LINKÖPING 4–5/10 1995

CPT '95

International Symposium on Cone Penetration Testing

Volume 1 • National Reports

Swedish Geotechnical Society

Report 3:95

SVENSKA GEOTEKNISKA FÖRENINGEN
SWEDISH GEOTECHNICAL SOCIETY

SGF Report 3:95

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Preface

In 1974, the first European Symposium on Penetration Testing, ESOPT was held in Stockholm, Sweden. The main objective of the symposium was to document the use of penetration testing, to outline areas of further research, to stimulate standardisation and to provide guidelines for future developments. After the second European Symposium, ESOPT-II in 1982, the ISSMFE Technical Committee on Penetration Testing decided to continue these successful speciality conferences on an international basis. Hence, in 1988 the first International Symposium on Penetration Testing, ISOPT-1 was held in Florida.

Since ESOPT was held more than 20 years ago, the role of geotechnical engineering has changed significantly. New geotechnical areas have been developed, such as geotechnical off-shore, earthquake and environmental engineering. The cone penetration test, developed by Dutch engineers more than 60 years ago, has become the most widely used geotechnical field investigation method.

During the past two decades, the CPT has emerged from a simple, mechanical field investigation tool into a reliable electronic multi-purpose testing methods. New types of cone penetrometers have been developed. A variety of sensors can now be incorporated in the cone, such as vibration and acoustic sensors, tilt meters, resistivity sensors, to mention just a few.

The *International Symposium on Cone Penetration Testing, CPT'95* is jointly organized by the Nordic geotechnical societies. The theme of the symposium is the solution of geotechnical problems by cone penetration tests. Particular emphasis is placed on the exchange of practical experience and the application of research results. The aim of the symposium is to enhance the exchange of knowledge between researchers and practitioners from countries all over the world and to facilitate the interaction between experienced and younger engineers.

The technical programme comprises Theme Lectures by eminent international experts in the area of penetration testing, presentations of state of practice in Technical Reports and selected papers, a Poster Session, a Technical Exhibition and a Field Demonstration. Information regarding the symposium programme, as well as lists of all papers and respective abstracts were available on Internet.

The symposium would not have been possible without the dedicated work and competence of the many authors which have submitted papers. The hard work and enthusiasm of many individuals and the support of many organizations and companies provided the basis for the planning and successful implementation of CPT'95.

Linköping, October 1995
Organizing Committee of CPT'95

K. Rainer Massarsch	*Bengt Rydell*	*Marius Tremblay*
Chairman	Vice Chairman	Secretary

Readers guide to Proceedings

In order to provide a sound basis for discussions and interaction between symposium participants, National Reports have been prepared by countries from all over the world which document the state of practice of cone penetration testing. Technical Reports have been prepared, covering the three sessions of the symposium. Theme Lectures were given on specific areas of CPT applications.

The symposium is documented in Proceedings consisting of three volumes:

Vol 1: National Reports

Vol 2: Technical Papers
- Session 1: Equipment and Testing
- Session 2: Interpretation of Test Results
- Session 3: Solution of Practical Problems

Vol 3: Theme Lectures
Technical Reports
Key Note Addresses
Summary Reports of Poster Session etc
List of participants

Vol 3 will also include Technical Papers received after deadline of submission. At the beginning of each volume there is a list of contents and at the end of each volume there is an author index. In Vol 1 the National Reports are listed alphabetically by country. The Technical Papers in Vol 2 are listed for each session alphabetically according to the first author.

The proceedings are published by the Swedish Geotechnical Society in the SGF Report series.

Review of papers

The abstracts submitted were reviewed by the Advisory Committee. Accepted papers were placed in the most appropriate session by the Organizing Committee, as some papers cover more than one theme.

References to proceedings

When using material from these Proceedings full credit shall be given to the symposium and the author(s).

Contents

National Reports on CPT

AUSTRALIAN NATIONAL REPORT ON CONE PENETRATION TESTING

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SYNOPSIS

Cone penetration testing has been used for about 20 years in geotechnical engineering in Australia to determine the strata sequence in alluvial and aeolian soils. The testing is performed throughout the whole of the continent but usually only in the heavily populated areas mainly along the east coast.

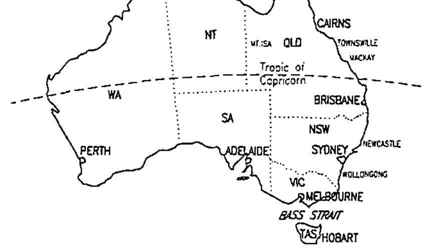
The standards used for testing are essentially similar throughout the country and reflect common practices in Europe. Conventional friction cone penetration tests are performed by many consulting companies with a few also engaged in piezocone testing. Only one commercial organisation is involved in resistivity and seismic cone testing. Interpretation methods have relied extensively on European experience and have been confirmed by local testing.

This paper contains a brief resume of cone penetration testing in Australia. It is not intended, nor indeed practical, to provide an exhaustive summary because of the vast size of the continent, the diversity of organisations involved in penetration testing and the space limitation of the paper. Nevertheless, it is hoped that the paper will give some insight into penetration testing in Australia, the way in which the test results are utilised, the research topics being studied and the areas of future research.

1. GEOLOGICAL AND GEOTECHNICAL CONDITIONS OF REGION

Australia is a continent which covers an area of 7,682,300 square kilometres, stretching from 10° to nearly 45° South latitude and 115° to 153° East longitude (Ref 1). Consequently, it is well nigh impossible to adequately describe the geological and geotechnical conditions of the region in the space available. Nevertheless it is worthwhile to show the main population centres where cone penetration testing is systematically performed for geotechnical investigation so that the readers are aware of the vastly different environments in which this testing is applicable.

The main population centres in Australia extend from Cairns in the north-east along the east coast to Brisbane, Sydney and Melbourne and along the southern portion of the continent to Adelaide and eventually Perth. Many areas between these centres are sparsely populated with only limited development so the need for testing is severely restricted. Mostly cone penetration testing is conducted in and around the major population centres along the coastline and river systems where deep alluvial and aeolian soil deposits are prevalent.



In addition to onshore cone penetration testing of alluvial soils for infrastructure and development projects, two offshore areas have seen a large number of tests performed in calcareous sediments for oil and gas field structures. These are in the Bass Strait oil fields off the south-eastern coast of Australia and on the north-west shelf where major gas deposits have been exploited.

2. INVESTIGATION METHODS

Geotechnical investigation methods in Australia can largely be divided into two types, namely drilling using *in situ*-dynamic penetration testing (i.e. standard penetration testing utilising automatic trip hammers) and quasi-static cone penetration testing. The equipment and procedures used for both SPT and CPT are described fully in Australian Standard 1289 published in 1977 (Ref 2). The current methods of performing SPT and CPT are little changed since 1977 but the CPT equipment has advanced substantially, particularly with regard to data recording and display. The pen type chart recorder used to plot the data for CPT's are long gone, replaced by digital data acquisition systems employing laptop computers and real time strata interpretation software.

Other *in situ* tests such as pressuremeters, Marchetti dilatometers, plate bearing, vane shear and dynamic cone are sometimes performed but these are used less frequently than the two principal test types and are normally restricted to special circumstances when standard methods are unsuitable or where design parameters are more easily obtained using alternative equipment.

Friction cone penetration testing has been used extensively for about 20 years in Australia but it is only in the last five years that a number of commercial organisations and research institutes have systematically performed piezocone testing. Moreover, seismic and conductivity cone testing have been provided by only one company in Sydney as standard geotechnical and/or environmental investigation tools.

3. TYPE OF CPT EQUIPMENT

3.1 Equipment

Electric friction cone penetrometers are the most common type of CPT equipment used in Australia. They are normally 10 cm^2 and 60 degree cones, with 150 cm^2 friction sleeves. All operate with power and communication cables and many of them are locally designed and manufactured.

More specialised CPT equipment is operated by a few consulting engineering companies and universities, and include piezocones, seismic cones, electrical resistivity cones, and water and gas sampling cones.

Piezocones are operated by several organisations, with all probes using conventional 10 cm^2 and 60 degree cone tips. Porous elements are located on the cone face or immediately behind the cone shoulder. A cordless data system is used by one company.

Seismic cones are used by only one organisation, and is generally used in association with earthquake or dynamic foundation assessments.

Resistivity cones use the standard 10 cm^2 cone configuration, with conductor elements separated by ceramic insulators. Four element systems are used to overcome polarisation effects, with the central two elements separated by 50 mm.

Groundwater sampling probes comprise the Hydropunch system and the BAT sampler. A modified Hydropunch type system is also used in the sampling of gases in the unsaturated zone above the water table, using a portable gas chromatograph to analyse gas content.

Use of inclinometers with CPT equipment to check verticality and to help prevent loss of probes is becoming increasingly more common as the complexity and cost of replacement probes increases.

Pushing rods are normally manufactured in Australia to typical European standard dimensions, though the quality of the steel used in manufacture may be lower than that specified by European standards. The use of slightly increased rod wall thickness is sometimes used to increase rod strength, though this may present difficulties in threading cables through them.

In most instances the power to the cone is provided by a communication cable which is threaded through the push rod although downhole signal processing is becoming more common. The signal is transmitted to the surface where it is normally decoded and processed by a laptop computer using locally written data acquisition software. Results are mostly plotted in the office from data stored by the computer logging system but some organisations still use analog printers to produce the results as tests are being performed. These analog printers are very unreliable in the Australian environment (heat, humidity and dust) and are fast disappearing from normal usage.

Friction cone penetrometers are calibrated either by the manufacturer, in the laboratory or in the field before and after each project. The most commonly adopted calibration procedure is to accept the manufacturer's calibration and assume that very little drift will occur unless the strain gauges are overloaded. Regular checks, however, are systematically performed by some organisations to ensure the integrity of the data.

3.2 Test Procedure

Although there is an Australian Standard on cone penetration testing, its use is not mandatory so the test procedures are not standardised nationally. Nevertheless there tends to be more similarities between the procedures adopted by both research institutions and commercial organisations than differences.

The standard method of pushing cone penetrometers has been to use purpose designed rigs similar to those employed in Europe. Hydraulic rams are located in the centre of a ballasted truck to provide a normal thrust of 15 tonnes and, in special circumstances for large projects, 22 tonnes (see Plate 1).

Some organisations have used conventional drilling rigs to push the cone penetrometer but this has limited the thrust to less than about 6 tonnes. Research is presently underway to reduce the friction on the cone rods using bentonite lubrication when utilising conventional drilling rigs but there are still severe limitations with such methods because of the low relative thrust when compared to specially designed test rigs.

Cones are zeroed before the test and pushed into the ground at a rate of 2 cm/s, with additional rods added every metre. Where drill rigs are used to push cones, the penetration rate may vary by up to 0.5 cm/s from the normal rate (depending on the uniformity of the soil deposit being tested) and push rods may be added in increments of two or even three metre lengths where there is little penetration resistance.

Measurements are typically performed automatically at 20 mm or 25 mm depth increments.

Probe tolerances for wearing components are not standardised and individual organisations use different tolerances depending on the intended use of the test information.

Testing is terminated where either the required depth is reached, or where the total penetration resistance exceeds the pushing capacity of the pushing rig. The use of various friction breakers is common and multiple reversal of the probe may often be used to reduce side friction and increase penetration depth. Multiple reversal may result in poor friction readings due to soil clogging the sealing rings at each end of the friction sleeve.

Piezocone probe test procedures vary between organisations. Porous elements typically consist of sintered brass, and saturation is usually performed in the laboratory prior to the test by either boiling the elements in water or by the use of 24 hour vacuum treatment in glycerine. Loss of saturation in the upper (unsaturated) soil is commonly prevented by predrilling a hole to the water table, or by filling a predrill hole with water prior to testing. Dissipation tests are common, with the duration of testing depending on the site conditions and the purpose of the test. Tests longer than about one hour are uncommon. Pore pressure measurements are normally made at the same depth increments as for cone and sleeve resistance.

For seismic cones, tests are usually made at regular intervals (usually 1 m) as the cone is pushed progressively into the ground. The shear wave is generated at the surface immediately above the cone by striking horizontally a steel or wooden block under the supporting jacks of the test rig.

Calibration of resistivity cones is achieved by either immersion in standard solutions or by the use of resistivity bridges. Readings are performed at 20 mm depth increments.

3.3 Presentation of Results

Presentation of results from normal CPT data include depth, cone resistance (MPa), sleeve friction (kPa), friction ratio (%) and comments related to interpretation. Additional field data, such as reduced level, groundwater level, and penetration rate, may sometimes be presented also. Site and project details are normally recorded on the presentation sheet, and sometimes probe identification and calibration, and operator details are included.

For piezocone tests, normalised and corrected parameters are frequently presented in addition to the measured parameters, and may comprise q_t, Q_T, F_R and B_q. Interpreted soil parameters, such as undrained shear strength, may be presented for particular sites where their inclusion can be justified (e.g. inclusion of s_u, where representative N_{kT} values have been assessed).

For seismic cones, the shear wave velocity (m/s) and interpreted dynamic shear modulus (MPa) are included in addition to the normal CPT parameters. For conductivity probes the conductivity (mS/m) are included in addition to the normal parameters.

Results are normally presented on standard A4 pages, with typical depth scales of 1:100 (1cm per 1 m penetration). Sometimes more detailed depth scales are used but the presentation of between 16 m and 20 m per page is typical. Some organisations present several interpreted parameters on separate sheets for each test depth increment.

Data presentation standards vary greatly but the trend is to high quality output in the office using laser printers. Examples of the standard being adopted by most commercial organisations is shown in the Figures 1 and 2 below.

4. INTERPRETATION OF CPT DATA

CPT data is used primarily for geotechnical engineering purposes, and interpretation generally includes:

- Soil stratigraphy using various published charts based mainly on friction ratio.
- Estimation of basic engineering properties such as shear strength, relative density, friction angle and moduli (although it is recognised that estimates of modulus are quite approximate).

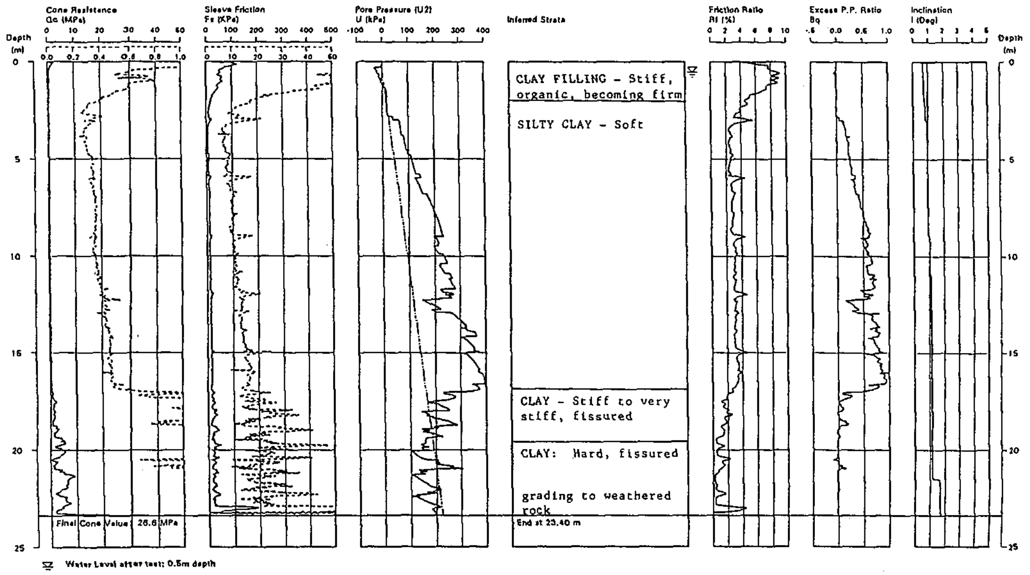


Figure 1 - Piezocone Penetration Test

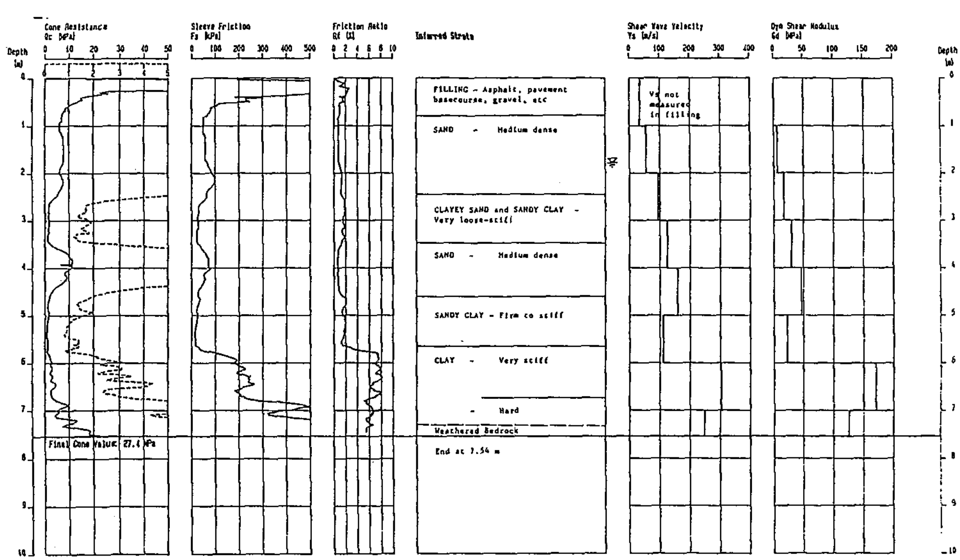


Figure 2 - Seismic Cone Penetration Test

- In the case of piezocone, additional properties estimated include over-consolidation ratio (OCR), consolidation characteristics, permeability and dilatancy. Published charts which relate pore pressure parameters (e.g. B_q) to soil type are used to aid soil type identification.
- Conversion of q_c to SPT N values is not common practice in Australia, with engineers preferring to work directly with q_c.
- Other parameters which are sometimes estimated from standard CPT include OCR, soil density, G_{max} and liquefaction potential, although the existing empirical correlations tend to be very approximate.

Interpretation charts and theory used to estimate soil properties from friction cone results are largely based on work published overseas. There is as yet little published data on local correlations, however the available material suggests that the established relationships are generally applicable to Australian soils. Some local trends may become apparent following further research and/or publication of data held by those commercial and government organisations involved in CPT.

Seismic cone tests are used to estimate low strain soil moduli (mainly shear modulus) for use in dynamic analyses, including earthquake and liquefaction assessments. There is also a trend to extend seismic cone data to estimate large strain moduli and Poisson's ratio from measurements of both V_s and V_p and then relating these to strain functions.

Resistivity (conductivity) cone tests have been used to complement environmental investigations since 1990. To date, interpretation of the bulk resistivity data has been mainly confined to broadly mapping contaminant plumes to enable better targeting of monitoring/sampling bores. Some work has also been done in assessing saline intrusion in coastal areas. Work being done overseas to relate multi-electrode resistivity measurements to soil properties such as porosity, density and dilatancy has not yet been tried extensively in Australia.

In one research project (Ref 3) an attempt was made to relate bulk conductivity to porosity in a coastal sand aquifer to help evaluate interstitial groundwater flow velocities. Despite the apparent success of this technique, it is not used systematically in investigation work at the moment.

Conductivity cone testing has largely been performed using a two electrode cone using alternating currents at a constant frequency. Some research work has been carried out at the University of New South Wales (Ref 6) using a four electrode cone operating at varying frequencies. To the authors knowledge the results have not yet been published but they may offer an insight into the real nature of groundwater contamination if different constituents in the groundwater are found to respond to variations in the applied current frequency.

5. USE OF CPT IN GEOTECHNICAL DESIGN

The main users of electrical cone penetrometer testing equipment in Australia concentrate on obtaining the following parameters for use in geotechnical design:

- Soil Type - from friction ratio
- Shear Strength - from cone resistance
- Modulus - from cone resistance

The shear strength and modulus values are obtained using empirical and experience based correlations, and hence are subject to varying interpretation. Where the values of shear strength and modulus are critical to design, confirmation testing is carried out using either *in situ* vane shear, pressuremeter and plate load tests and/or laboratory testing including triaxial and oedometer tests.

Once the above parameters have been determined, the results are used for the design of both high level and piled foundations (both bearing capacity and settlement). The most common Australian practice is to design directly from q_c values without converting to SPT N values as is commonly done elsewhere throughout the world. In particular the q_c profile is increasingly being used for the design of driven piles using computer aided "profile averaging" techniques for q_c values above and below the target founding level.

Soil strength is usually determined directly from cone resistance value. There is a wide variety of relationships for clay in use but one commonly used:

$$Su = (q_c - \sigma_v^{'}) / N_k$$

where q_c = cone resistance
Su = undrained cohesion
$\sigma_v^{'}$ = effective vertical stress
N_k = 12-15

At this point it should be noted that bearing capacity is rarely the primary design consideration in Australia. Settlement is normally a more critical factor so correlation of cone parameters with modulus has been more widely studied. Overseas published correlations between modulus (or soil compressibility) and cone resistance indicate a wide variation and this experience is also true in Australia. For sands, the relationship most commonly used is $E = (1.5 \text{ to } 5)q_c$ with the value of the multiplier being 2-3 in most circumstances. For clay, $E = (3 \text{ to } 7)q_c$ is frequently used but lower values have been noted in one instance in Sydney when measured and predicted settlements were compared. It is unclear, though, how much of the measured settlements over the 20 years of recordings are due to creep in the soft clay alluvial deposits.

One government organisation (Ref 4) has published a relationship between cone resistance and California bearing ratio (CBR) for pavement design [CBR(%) = 4.5 q_c (MPa)] although it is unclear whether this represents a lower bound estimate for design or is an average value. Moreover, estimated moduli from this relationship [based on E(MPa) = 10 CBR (%)] give values very much higher than conventional estimates probably due to small total strains and high loading rates which occur in pavement testing. Clearly the relationship is applicable to pavement design only.

The piezocone is being used by a minority of cone operators and reports indicate that these operators are gaining confidence in the use of the results to determine consolidation rates, dilatancy, and permeability; in addition to soil type. It is noted that retrieval of these design parameters from the piezocone would appear to be very experience and operator dependent. The piezocone requires more careful use and result interpretation than the standard electrical friction cone to obtain reliable design parameters for use in determination of rate of consolidation and seepage characteristics.

Other "fringe" activities of some Australian cone operators is the use of the seismic cone to obtain shear modulus for design of footings subject to dynamic loading, and the use of the resistivity or conductivity cone to map contaminant plumes.

In summary, the results of cone penetrometer tests are widely used in geotechnical design by experience operators in Australia - predominantly in the determination of bearing capacity and settlement of both high level and piled foundations.

6. CORRELATION OF CPT WITH OTHER METHODS

Correlations between cone penetration test results and other test methods are only infrequently attempted in Australia because of the limited scope of most geotechnical investigations. In one known case cone tests were performed beside a test bore in which Marchetti dilatometer tests were performed from just beneath the surface to depths of around 20 m. Within medium dense sand layers (q_c = 5 to 15 MPa) the relationship between constrained moduli (calculated in accordance with Marchetti's recommendation) and cone value was generally in the range E(MPa) = 3 to 20 q_c. This upper limit was much higher than expected although most results were in the range of E = 3 to 10 q_c which is consistent with data published overseas.

Dilatometer tests were also carried out in soft clay layers on the same site but estimates of shear strength (0 kPa) and modulus (0.1 to 0.4 MPa) are clearly too low to get meaningful results at the bottom of a borehole even though extreme care was taken to minimise disturbance.

Jones (Ref 5) studied correlations between various parameters and q_c in Newcastle. Typical results are shown in Figures 3, 4 and 5 and indicate correlations which are similar to those predicted from overseas experience.

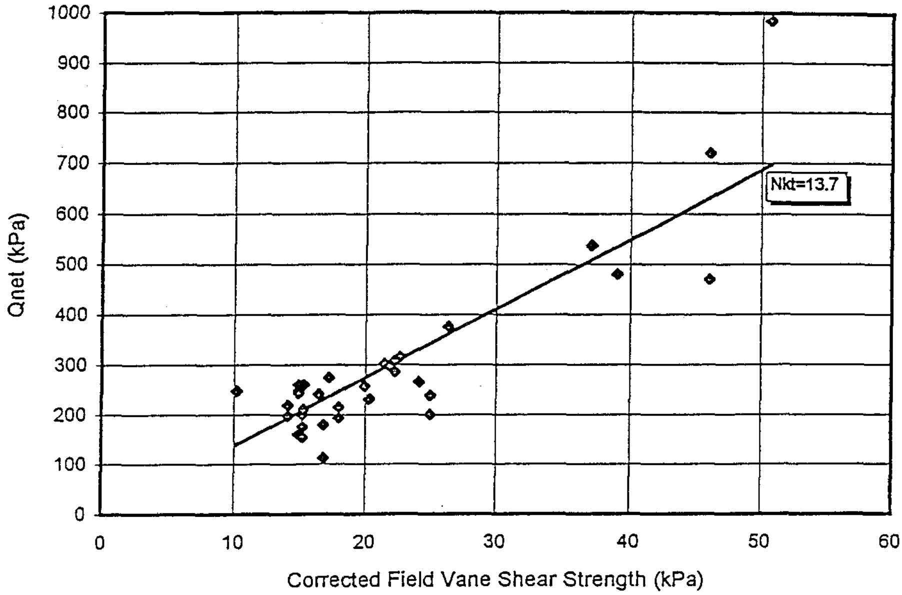


Figure 3 - Net Cone Resistance vs Field Vane Shear Strength (after Jones)

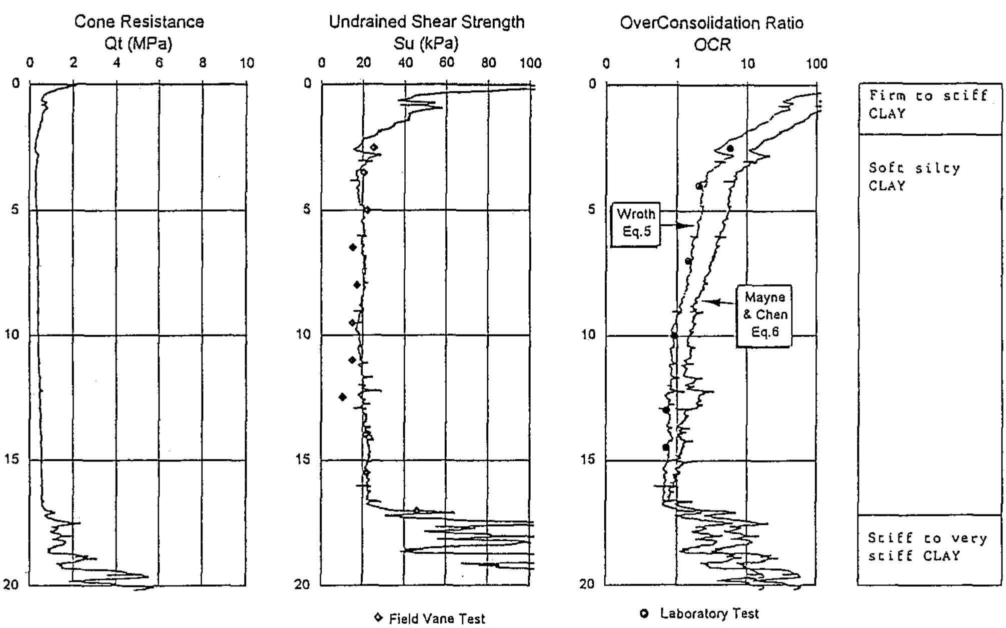


Figure 4 - Comparison of Shear Strength and OCR Results (after Jones)

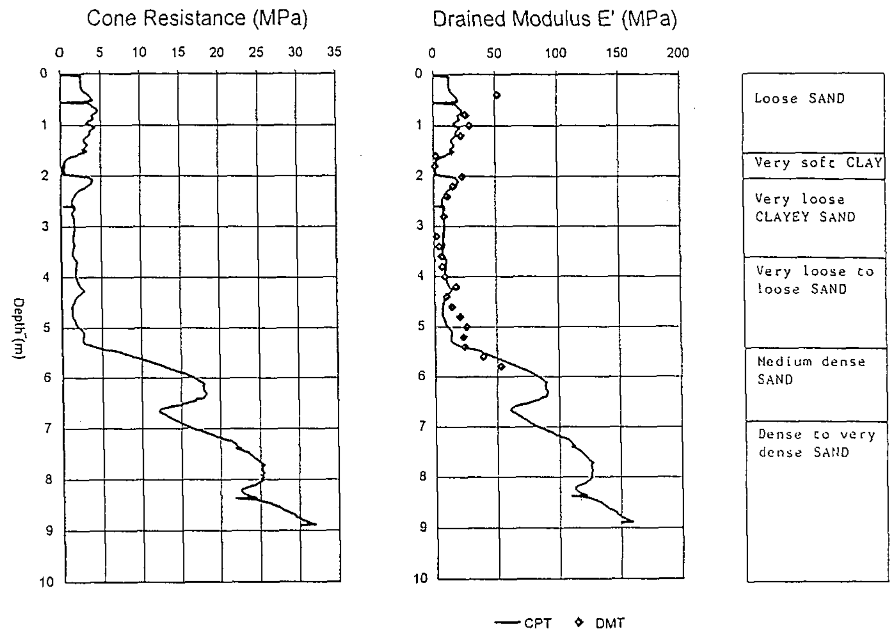


Figure 5 - Drained Modulus at Williamtown (after Jones)

7. RESEARCH

CPT research is essentially the domain of the Universities, and largely concentrates on calcareous (or otherwise compressible) soils and soft alluvial soils, with test chamber investigations being prominent.

At Monash University (Clayton Campus, A Parkin chief investigator), a 1.2 m dia by 1.8 m high double-wall test chamber has been adapted to allow the simultaneous installation of CPT and driven model piles in uncemented calcareous silty sands. Interest has focussed on the static and cyclic performance of driven/grouted pipe piles and the application of CPT data to the driving behaviour of open-ended pipes. Of necessity, procedures have had to be developed for the processing of saturated and highly silty materials, and for accommodating volume changes not normally encountered with such equipment.

A 1 m by 1 m chamber (with airbag loading) is operational at the University of Newcastle (H-S Yu, chief investigator), where the prevailing interest is in pressuremeter interpretation, and in particular in the effects of guard cell length and operational procedure, using silica sands. This work may, at a future time, be extended to calcareous sands in co-operation with Monash University in Melbourne. Extensive theoretical development is being pursued via cavity expansion theory.

At the working end of the scale, the reliability of CPT interpretations of engineering parameters is being assessed in another study at the University of Newcastle (H-S Yu and S R Jones). Standard CPT, piezocone and seismic cone data is being systematically gathered from diverse sites and compared with other in-situ (eg dilatometer and shear vane) and laboratory test data, including calibration chamber tests. It is expected that these results will ultimately be integrated into a computer package.

What has become known as the CRB chamber, measuring 0.9 m dia by 1.2 m high and the prototype of various double-wall chambers around the world, is presently located at the University of Auckland, New Zealand. Interest there lies in CPT interpretation in compressible volcanic soils, but investigations have not advanced beyond some preliminary studies at this stage (L Wesley, chief investigator).

A long-standing interest in calcareous soils at the University of Western Australia (Perth) has fostered a project to study the role of compressibility in the relationship between q_u from the CPT and the triaxial UCS. This project uses calcareous sands with varying (from nil) degrees of artificial cementation and varying confining pressure. Samples are prepared in a small calibration chamber and in a geotechnical centrifuge and probed with a model CPT (M Fahey, chief investigator).

CPT applications to soft clays are also being pursued at UWA (M Fahey), where the piezocone dissipation test is being studied as a means of deriving c_r in-situ. A cone measuring horizontal total stress and pore-pressure at matching points on the shaft has been specially developed for this work and results are compared with values from a self-boring pressuremeter. Other programs in soft clays are being undertaken at James Cook University (Townsville) using as a seismic piezocone in association with the Queensland Department of Transport. Procedures have been developed for seismic studies (of shear wave velocity), but generally the emphasis has been on the evaluation of properties of soft coastal soils and mine tailings (J Eckersley, chief investigator).

8. FUTURE TRENDS

Anticipated developments in CPT usage in Australia may be broadly divided into the following areas:

- increased application to environmental investigations.
- The use of increasingly sophisticated software to process, interpret and present CPT data.
- Development of specialist research cones for use in field and/or laboratory based studies.

A marked increase in the demand for thorough yet cost-effective environmental/ geotechnical investigations has occurred in recent years. Elsewhere this has led to the inclusions of more sophisticated instruments in the cone, such as ultra violet fluorescence to detect hydrocarbon contaminants. This technology is now commercially available in USA/Europe, and is likely to find application in Australia in the near future.

A multi-electrode resistivity cone, which includes temperature, pH, Eh and dissolved oxygen sensors, may also be commercially available in the next 1 - 2 years. A further development being considered is a nuclear density/moisture cone, however the market potential appears limited at this stage.

The use of both purpose written software and spreadsheet packages to process and interpret CPT data is currently widespread. These are being extended to provide more interpretative functions such as soil type, soil properties, and integration with contouring or graphical packages to produce contours and cross-sections to aid visualisation of various site characteristics. These interpretative functions need to be backed up by appropriate (preferably local) empirical correlations and/or theory in order to validate the output. This work is largely being carried out by university based researchers, as described in the previous section. Increased use of multi-parameter cones is foreseen in these research organisations such as seismic piezocones and dual or triple element piezocones. Such instruments will take longer to find a place in commercial geotechnical work due to the higher capital and operating costs.

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Penetration testing in Austria

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SYNOPSIS: Due to the lithology in Austria not more than approximatly a quarter of the area can be considered as suitable for dynamic penetration testing. Cone penetration testing can successfully performed in very limited areas of postglacial deposits. Thus, cone penetration testing is in Austria only occasionally carried out. Dynamic penetration testing (dynamic sounding and Standard Penetration Testing) is, on the other hand, carried out in tertiary and quaternary deposits on routine basis.

1. GEOLOGICAL BACKGROUND

1.1 General remarks to the geomorphology

Austria covers an area of about 84.000 km^2. The major part of the Austrian territory is dominated by the mountainous character of the Eastern Alps. They represent the eastward continuation of the Swiss Alps and are orientated in a generally east-west direction. At their eastern end, near Vienna, the Eastern Alps swing northeastwards and pass over into the Carparthians. The highest regions of western and central parts of the Eastern Alps with summits exceeding an altitude of 3000 m, are glaciated. Towards the east the height of mountains is gradually decreasing.

The northern and northwestern parts of the country are occupied by the southern margin of the Bohemian Massif revealing a well wooded, moderate mountainous landscape with elevations up to 1400 m.

Between the smooth northern outliers of the Alps in the south and the Bohemian Massif are the wide low-lands of the Molasse Basin. Further prominent low-lands being of importance for the national economy are the Vienna Basin in the east and the Styrian Basin (around Graz on the border to the Pannonian Basin) in the south-easternmost part of Austria.

1.2 Lithology of the principal geomorphic divisions and its possibility for penetration testing

- *The Eastern Alps*

They are built up by a wide range of hard rocks from limestone to marble, from clay- and siltstone to phyllite and micaschist, granite, gneiss and amphibolite. With the exception of the valley fillings, which are described below, and weathered marls there is obviously no need for soil mechanical penetration testing in the Eastern Alps.

- *Tertiary Basins*

The most prominent basins are the above mentioned Molasse Basin, Vienna Basin and the Styrian Basin.

Shales with partly thin sand intercalations, clayey marls, sand, gravels and small amounts of lignite are the main compounds of the basins lithology. Limestone is only present in the Vienna and Styrian Basin. Due to uplift and erosion, highly consolidated and partly indurated shales and marls outcrop in these regions.

Generally good quality undisturbed samples can be taken from these strata or good quality in situ tests can be performed. So far only occasionaly, static and dynamic penetration tests have been carried out.

- *Bohemian Massif*

Here only the intensivly weathered zones of the cristalline rocks (predominantly granite and gneiss) and some small tertiary Basins offer a limited possibility of using dynamic penetration testing.

- *Quaternary Deposits*

Quaternary deposits are the most important foundation soils for traffic lines (railways, highways) and buildings in Austria.

During the last glacial period (four main glacial epochs are documented) the Alps were covered by big ice streams covering a large part of the landscape.

The voluminous valley glaciers reached far into the foreland which is evidenced by large end moraines and other glacial phenomena. As a result, deposits related to the glacial activities like till and glaciofluvial sediments (layers of silt, sand and gravel) are common within alpine valleys (e.g. Inn, Enns and Traun valley). After the withdrawal of the glaciers the overdeepened valleys and Basins have been filled up with gravels and unconsolidated clayey silt (e.g. the Salzach valley around Salzburg). In some cases the last stage of the infillment of glacial lakes was peat (e.g. east of Salzburg, Rhine valley).

The oversteepening of alpine slopes due to glacial erosion caused big postglacial mass-movements (rockfalls, rockslides and sagging). These deposits and - on a smaller scale - the big alluvial fans of some Alpine creeks caused impeded drainage of the upstream valley floors and the formation of shallow lakes. Unconsolidated silty, clayey sediments and peat are the typical foundation soils in these regions (eg. upper Gail valley, middle Enns valley).

In the non-glaciated periglacial foreland extensive accumulation terraces (built up by gravels and sand) were deposited by rivers which can be considered as the precursors of the tributaries of the Danube. Well developed steps of terraces can be observed in the Molasse Basin and the Vienna Basin along the course of the Danube and her major tributaries. During the cold periods a considerable loess and loess loam cover (up to ten meters or more) was deposited on the top of the older terraces and the surrounding hills.

Considering the nature of the soils it can be concluded that no more than 25 % of the total avaialble area in Austria is suitable for penetration testing and is confined primarily to the quaternary deposits.

2. PENETRATION TEST METHODS AND EQUIPMENT USED IN AUSTRIA

Due to the complex geological and geomorphological situation of Austria in only rather limited areas of zones with soft, unconsolidated quarternary deposits especially of postglacial age cone penetration testing (CPT) could be used successfully. However, to the authors knowledge from literature and from the information of a questionaire sent out recently, cone penetration testing is not performed in Austria at all.

Dynamic penetration tests according to the Austrian Standard (ÖNORM B 4419, Teil 1) are, on the other hand, carried out on routine basis. From the different methods described in the ÖNORM the heavy sounding method (SRS) is used more widely. In the Standard the equipment (shape and dimensions of the rods and the sounding points) and some factors influencing the results are described. No information is given how to interprete the results or how correlations to densities, consistencies or material parameters can be obtained.

The Standartd Penteration Test (SPT) is used in Austria whenever dynamic soundings give no reliable results and undisturbed samples of sufficient quality can not be taken, i.e. in tertiary overconsolidated fissured soils. Basically, the interpretation of the results of the Standard Penetration Tests follows the classical methods used in the USA, or more recently published methods as summarized by e.g. Biedermann (1979).

3. TEST PROCEDURE, APPLICATION AND INTERPRETATION OF TEST RESULTS OF DYNAMIC PENETRATION TESTING

The tests procedure of the dynamic penteration tests (SRS, SPT) are described elsewhere, DIN 4094, ÖNORM B 4419, Teil 1. The results of the sounding tests are plotted as penetration resistance (number of blows) for 10 cm penetration versus depth.

Dynamic penetration testing is employed primarily to evaluate the density of granular materials, the consistency of fine grained soils or the homogeneity of a specific construction site. Material parameters, such as stiffness moduli or strength parameters are seldom evaluated from the results of dynamic sounding tests. Only in areas, where material parameters are well known from experience (such as the quaternary gravel deposits of the river Danube) the penetration resistance is correlated to e.g. the stiffness modulus (Gstöttner, 1987). Calculations of settlements and bearing capacity of piles based on results of dynamic penetrations tests or Standard Penetration Tests are used according to methods outlined in the State of the Art Report by Biedermann, (1979).

Under conditions where good quality samples can be taken, material parameters, determined from laboratory tests on undisturbed samples or from in situ tests (e.g. pressuremeter tests) are preferably used for geotechnical analyses.

3.1 Examples of application

In the following two typical examples are given how dynamic penetration tests are applied in Austria mostly and how the results are interpreted.

3.1.1 Compaction control in trenches

Design parameters are chosen from experience or determined from laboratory tests for the structural design of embedded pipes. The design parameters depend to a large extend on the relative desity of the backfill material. Consequently, the soil (either as dug material or imported material) is generally placed in layers and each layer is compacted by adequate means. To check the compaction in trenches, dynamic penetration tests with the light (10 kg hammer) and the heavy (50 kg hammer) dynamic sounding equipment are carried out (cf. fig. 1). The periodic change of the penetration resistance is typical for well compacted layers of about 50 cm thickness. With the light sounding equipment the differences in densities are more pronounced, as might be expected. The ratio of the results of the light and heavy sounding equipment is typical for well graded sandy gravel and can be described with reasonable accuracy by the well known work equations.

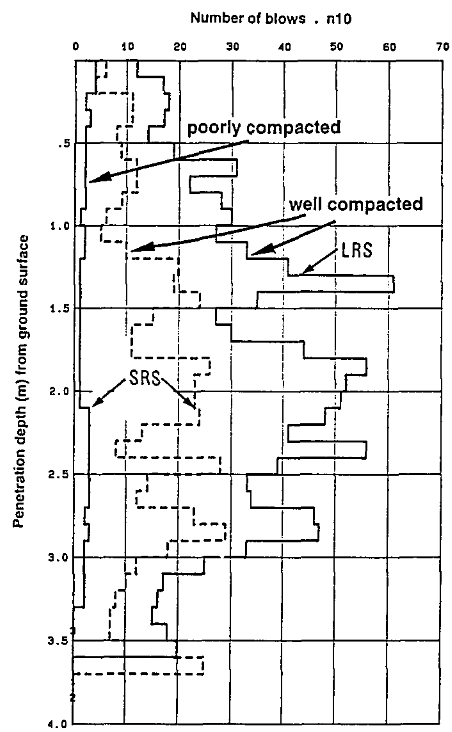

Fig. 1. Results of dynamic sounding tests (LRS, SRS) of well and poor compacted backfill of a trench

The results of a poorly compacted trench from the same site is also shown in fig. 1. From the penetration resistance it can be concluded that only the surface of the backfill has been compacted. The remainder of the trench was just filled up without any compaction. Considerable settlements in the order of up to 10 cm will develop with time in that part of the trench.

From a considerable number of tests in the Vienna area with typical back fill material of well graded sandy gravel it was found that the minimum number of blows per 10 cm penetration with the heavy sounding equipment should not be less than 6 over the entire depth of the trench. The settlement will then not exceed acceptable values.

3.1.2 Evaluation of soil improvement

The foundation soil in the Vienna area are the quaternary deposits of the river Danube and their tributaries. They consist primarily of inhomogeneous sandy gravel overlayed by loamy silts. The grain size distribution and density of the gravel can vary cosiderably in all directions. Dynamic penetration tests are widely used to determine the necessity and the method for soil improvement. The inhomogeneity with respect to density of the site is generally investigated by dynamic soundings with the heavy sounding equipment (SRS). The most effective soil improvement method e.g. vibro compaction, vibro replacement or stone columns can thus be evaluated.

Gstöttner (1987) has shown that, based on the results of dynamic sounding tests, complex multistoriage hospital buildings could be founded on strip footings on inhomogeneous soils after soil improvement of carefully selected areas.

4. SUMMARY

Due to the complex geological sintuation in Austria cone penetration testing is not carried out. On the other hand, dynamic penetration testing (dynamic sounding acc. to DIN and ÖNORM) is carried out on routine basis. The heavy sounding method (SRS) is most widely used to evaluate the density of coarse grained soils or the consistency of fine grained soils and to investigate the homogeneity of a specific construction site. Dynamic soundings are extensively used to check the compaction in trenches, to evaluate ground improvement methods and to check soil improvements.

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SYNOPSIS:
The CPT has been used extensively in Belgium since the first years that followed World war II, so the test procedures, the interpretation methods and the design methods are well established and are summarized in the present report.

1. GEOLOGY

The country is rather flat with a continuous transition from a plain at the North Sea and the Dutch border to the highlands of the Ardennes, the highest point being situated at Botrange (640 m above sea level).

The geology of the Tertiary and Quaternary formations in Belgium is characterized by an approximately South-East North-West oriented epirogenetic axis (Silence, 1992), which follows the valleys of the rivers Haine, Sambre, Meuse and Vesdre (Figure 1), divides Belgium to approximately two equal parts.

In the North part, the stratigraphy was governed by fluctuations in the coastal line. Consequently the bedrock is covered by alternating Tertiary clay, sand and (occasionally) gravel sediments, with thicknesses up to hundreds of meters. The Quaternary Pleistocene formations have been heavily influenced by the glacial periods, giving rise to the formation of marine, coastal, river, lake or wind deposits of sand, clay, peat and silt (loëss). Holocene erosion and river sedimentation, as well as human activities, have further influenced the actual subsurface. *In the South* of the epirogenetic axis, the bedrock is often found at rather shallow depths, overlain by colluvium layers consisting of weathered rock and river sediments.

As a result of the geological history, one can find in the North a wide variety in stratigraphy, with complicated and heterogeneous soil layer patterns. It is not therefore surprising that the North of Belgium (like the Netherlands) has to face serious foundation problems, requiring particular foundations such as piling or ground improvement. The soil layers interferring in the foundation design most frequently allow for the execution of CPT. In the South, soil investigation as well as foundation design may be based on CPT as well, although during the last decade other tests, e.g. pressuremeter tests, have known an increasing use.

So, in Belgian practice, calculations for foundation design are by far based on CPT. During the last decades, very extensive research work by geotechnical experts, universities, research institutes and contractors, with the support of the authorities and control organizations, has resulted in a wide experience and know-how on the use of the CPT to the design of piles and their behaviour, and also to some extent on most common soil improvement methods (such as soil compaction or stone columns).

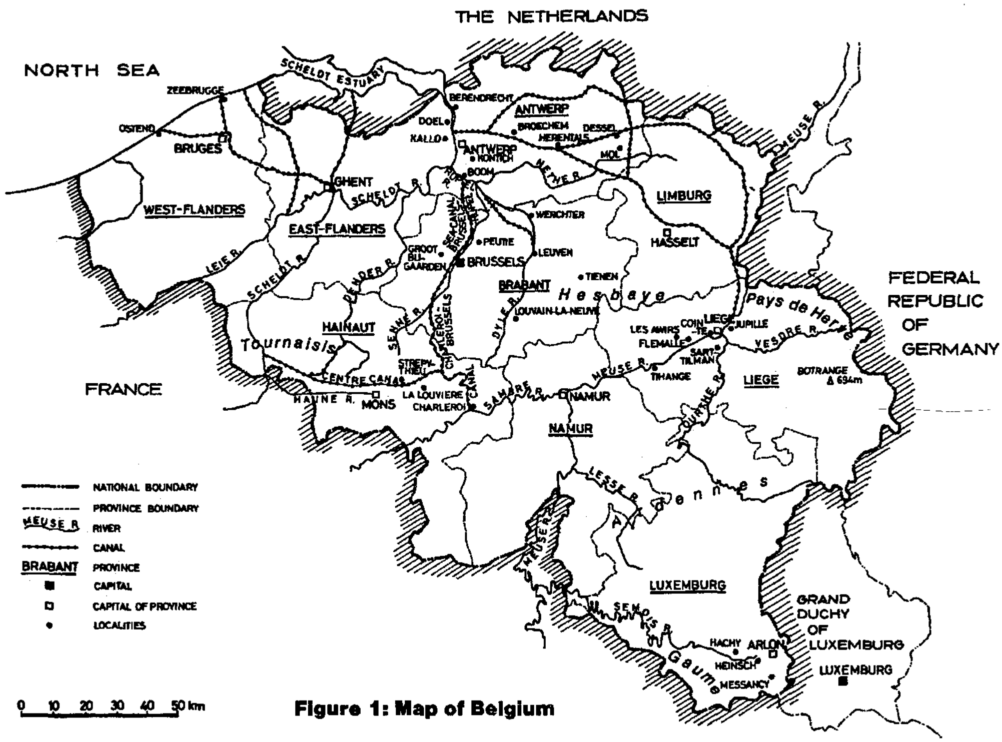

Figure 1: Map of Belgium

2. INVESTIGATION METHODS

The use of the CPT was developed in Belgium by De Beer (1945) with the elaboration of the mechanical simple cone with closing nut M4 and by Verdeyen (1945) with the use of the Dutch mantel cone M1 (Figure 2). The CPT remained the most commonly used technique for *in situ* geotechnical testing in Belgium. De Beer (1948) did pionering work in developing his own interpretation and calculation methods based on a bilinear intrinsic curve while Verdeyen (1948) promoted the rational interpretation according to the classical theory of plastic limit equilibrium. De Beer's research work played a major role in the introduction and general acceptance of the CPT in Belgium from the early days this equipment was developed

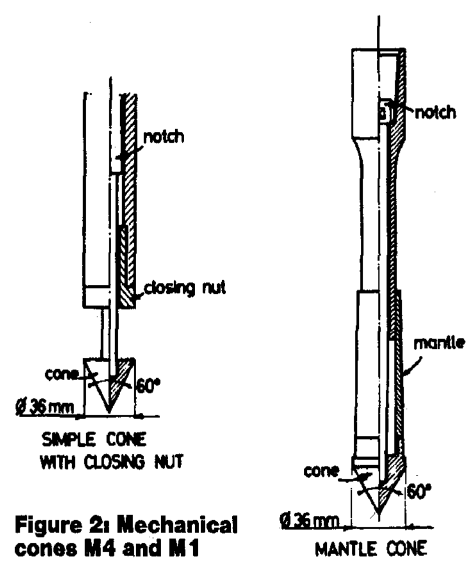

Figure 2: Mechanical cones M4 and M1

Dynamic penetration testing is also performed: the cone penetration testing is used mostly in the field of road construction; the SPT has been used a lot during the sixties for the design of industrial plants.

More recently, the growing need for rational stress-strain analysis is at the origin of the fast increasing use of pressuremeter techniques.

3. TYPE OF CPT - EQUIPMENT USED IN BELGIUM

3.1. NATIONAL CODES AND/OR STANDARDS

Recently, recommendations (available at CPT'95) were issued describing the CPT equipment, the way tests should be performed and how the data should be presented. Companies following these recommendations will be able to get a certification which is needed to work for the regional or federal government services. Also private companies will use these recommendations as a guideline for their site investigation tests. At present the possibility is considered to transform these recommendations to a standard for CPT testing in Belgium.

The above recommendations are mainly based upon the recommendations for CPT testing of the ISSMFE (Report of the ISSMFE TC16).

3.2. EQUIPMENT

For economical and technical reasons, the mechanical cone is still more often used than the electrical cone. This is especially the case in certain parts of Belgium where soil conditions are too severe (presence of rock, sandstones, ...) for the highly sensitive electrical cones. Because of this fact, both the mechanical cones and the electrical cones are accepted in the recommendations.

The mechanical CPT can be performed with the mantle cone (M1), the friction sleeve cone (M2) and the simple cone (M4).

As electrical cones, both the standard electrical cone (CPTE) equipped for measuring cone resistance and local friction (and preferably inclination of the cone) and the piëzocone (CPTU) can be used.

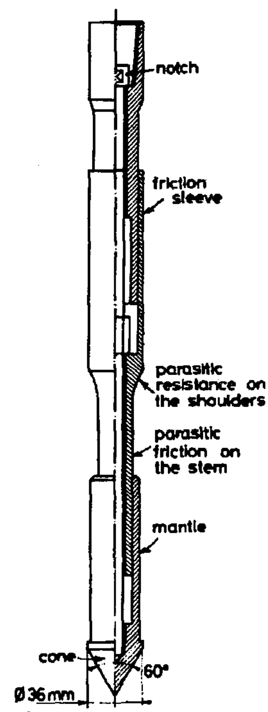

Figure3: Mechanical cone M2

3.3. TEST PROCEDURES

One condition which has to be fulfilled concerning the execution technique when performing the mechanical CPT test is that the different parts of the cone must have a significant displacement relative to each other.

For CPTU tests, no recommendations are given for performing dissipation tests, although for completeness it is mentioned that such a test can be performed.

The measuring equipment of both the mechanical cone and the electrical cone has to be calibrated by an officially accredited laboratory every 6 months. When extensively used, a more frequent calibration is advised: after 250 tests or after 3000 m of sampling.

Calibration of the electrical cone consists of calibrating the cone as used in the field with analog and digital equipment. A small apparatus for checking calibration of electrical cones with measuring equipment on the site is advised for regular use. A logbook, which should always be present in the CPT truck, serves as a possible means for control of this requirement. The logbook has to contain data and information about the use of the equipment and the calibrations.

3.4. CORRECTION AND PRESENTATION OF TEST RESULTS

For mechanical cone tests, a correction for the weight of the tubes and inner rods is recommended but not mandatory.

For electrical cone tests, a correction of the q_c value for the porewater pressure acting on the back side of the CPTU tip and a correction of the depth for the inclination of the tubes are advised but not mandatory.

For all CPT procedures, when corrections are included, they must clearly be stated in the report.

It is generally known that the different cone types can give different results. Because of the large influence of soil type and stress condition, no general correction formula is available for these differences. The importance of possible errors must be recognised and it is mentioned in the recommendations. In this respect, it is generally accepted that the CPTE-cone serves as a 'reference test'.

The presentation of the test results follows the instructions of the TC16 report.

4. INTERPRETATION OF TEST RESULTS

4.1. SOIL PARAMETERS AND OTHER DATA

Soil parameters derived from CPT-results are :

- friction angles $\varphi'_{De\ Beer}$ and $\varphi_{De\ Beer}$ or φ'_d;
- deformation modules;
  - stiffness index C as derived from oedometer tests, used for settlement calculations in Belgium and the Netherlands and related to the compression index C_c by

$$C = 2.3\ (1 + e_0)\ /C_c$$

  - E-modulus;
  - G-modulus;
- undrained shear strength c_u.

4.1.1. FRICTION ANGLE

De Beer's method for assessing the friction angles φ and φ' (commonly taken as 30°) of the bilinear intrinsic curve of his foundations design methods was used for a long time in Belgium and has been described by Sanglerat (1972).

For limit equilibrium and retaining wall calculations in granular soils, φ'_d-values are derived according to Meyerhof, resp. Caquot & Kerisel from equation $N'_q = q_c/\sigma'_{v0}$.

Besides these two methods, different correlations between φ'_d and q_c for granular soils are used for practical applications (Durgunoglu and Mitchell 1975) , (Robertson and Campanella 1983).

Tables presented in the Dutch standard NEN 6740 and in the German Standard DIN 1055 Teil 2 give for different q_c-levels and/or soil types representative values for different soil parameters, including φ' and c'-values.

4.1.2. DEFORMATION MODULES

C-values are derived from CPT-results according to the theory developped by Buisman (1940)

$$C = \alpha\ q_c/s'_{v0}$$

With the Buisman value of $\alpha = 1.5$ a conservative estimate of C is derived from CPT tests with the same order of magnitude as from consolidation tests. For tertiary overconsolidated soils the derived C-value is then multiplied by 3 for clay, and 10 for sands. In today's practice the α factor is differentiated for each soil type as proposed by Sanglerat (1972).

The constrained modulus M is obtained by a similar formula

$$M = E_0 = \alpha\, q_c$$

with the same values for the α factor. M is also calculated starting from tables from other authors like Mitchell and Gardner (1975) and Lunne and Christofferson (1983).
Relationships between the drained Young modulus (E_{25}, E_{50}) and q_c for NC-sands and between dynamic shear modulus G_{max} for NC sands and q_c are used, after Robertson and Campanella (1983). A table giving normalised E-values for different soil types and corresponding q_c-levels is found in the Dutch report on sheetpiles CUR 166 (1993).

4.1.3. UNDRAINED SHEAR STRENGTH c_U.

Derived from the classical theory bearing capacity of a pile, the in situ undrained shear strength c_u is provided by

$$c_u = \frac{q_c - \sigma'_{v0}}{15}$$

N'_c for $\varphi = 0$ is theoretically equal to 0 but one has to take into account the cone type and the soil plasticity. In most cases $N'_c = 15$ and the influence of soil plasticity is taken into account when design values are put forward (e.g. use of Bjerrum correction factor for slope stability calculations). Most often the equation is simplified to

$$c_u = \frac{q_c}{15}$$

For stiff overconsolidated clays c_u-values are derived according to the following expression, which is a function of the sensitivity S_t of the clay.

$$c_u = \frac{q_c}{10 \times S_t}$$

5. USE OF CPT IN GEOTECHNICAL DESIGN

5.1. BEARING CAPACITY OF SHALLOW FOUNDATIONS

In Belgium, the calculation of the bearing capacity of a centrally and verticaly loaded shallow foundation based on CPT results uses the classical formula

$$q_u(\mathrm{kN/m^2}) = s_q \cdot d_q \cdot p_t \cdot N_q$$
$$+ s_c \cdot d_c \cdot c \cdot N_c + s_\gamma \cdot d_\gamma \cdot \gamma \cdot B \cdot N_\gamma$$

The three terms represent the contribution of surface loading p_t (subscript q), cohesion c (subscript c) and soil unit weight γB (subscript γ), respectively. They are multiplied by dimensionless bearing capacity factors (symbol N), shape factors (symbol s) and depth factors (symbol d). B and L are the dimensions of the footing in meters. The values of p_t and c are expressed in kN/m², γ is expressed in kN/m³. The factors N_φ, N_q and N_c are the same for the different methods:

$$N_\varphi = \mathrm{tg}^2\left(\frac{\pi}{4} + \frac{\varphi}{2}\right)$$

$$N_q = N_\varphi \cdot e^{\pi \cdot \mathrm{tg}\varphi}$$

$$N_c = \left(N_q - 1\right) \cdot \mathrm{cotg}\,\varphi$$

The factor N_γ and the shape and depth factors depend on the method used. The most frequently used factors are those of Meyerhof (1951)

$$N_\gamma = \left(N_q - 1\right) \cdot \mathrm{tg}(1.4 \cdot \varphi)$$

$$s_q = 1 + 0.1 \cdot \frac{B}{L} \cdot N_\varphi$$

$$s_c = 1 + 0.2 \cdot \frac{B}{L} \cdot N_\varphi$$

$$s_\gamma = s_q$$

$$d_q = 1 + 0.1 \cdot \frac{D}{B} \cdot \sqrt{N_\varphi}$$

$$d_c = 1 + 0.2 \cdot \frac{D}{B} \cdot \sqrt{N_\varphi}$$

$$d_\gamma = d_q$$

but those of Vesic and of the DIN 4017 and NEN 6744 are also used. In case of an eccentric and/or inclined load the corrections of Meyerhof (1951) are used:

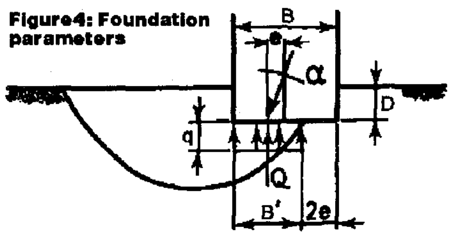

Figure4: Foundation parameters

$$B' = B - 2\,e_B$$

$$L' = L - 2\,e_L$$

$$i_q = \left(1 - \frac{\alpha}{90^\circ}\right)^2$$

$$i_c = i_q$$

$$i_\gamma = \left(1 - \frac{\alpha}{\varphi}\right)^2$$

The calculated bearing capacity should be divided by a safety factor to obtain the design value. In case of normal loading conditions, a safety factor of 2.5 is used with De Beer's while other methods need a safety factor of 3 to 3.5.

5.2. PILE BEARING CAPACITY

5.2.1. END BEARING CAPACITY

The "De Beer" method, is without any doubt, the most widely used method in Belgium for the calculation of the end bearing capacity of single piles. Its theoretical background has been elaborated and published some 25 years ago (De Beer, 1971). The method and later modifications have also been reported in ESOPT and ISOPT proceedings by Van Impe among others.

The De Beer method is based on a multi-step consideration of scale effects, when passing from a soft to a hard soil layer. This application of the scale effect is performed in 4 steps, designated by the terms homogeneous values, descending or downward values, upward valus and mixed or blended values.
The final mixed values $q_{u,b}^{(m)}$ are the basis values for the further end bearing calculation of the pile. They have to be reduced, according to the way of pile installation and according to the soil type.

An example of calculation is shown in Figure 5, whereby the variation of $q_{u,b}^{(m)}$, as calculated according to the basic De Beer method ($q_{u,b}^{(m)}{}_{DB}$) is given, together with the measured unit cone resistance q_c.

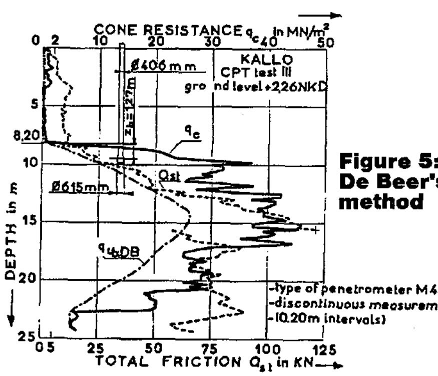

Figure 5: De Beer's method

The pile ultimate end bearing $Q_{u,b}$ is further defined as :

$$Q_{u,b} = q_{u,b} \times A_b = \alpha_b \times \varepsilon_b \times q_{u,b}^{(m)} \times A_b$$

with : α_b = a coefficient taking into account the way of installation of the pile;

ε_b = a parameter referring to the scale effect by the fissuring of the soil;

A_b = the nominal pile base section.

The ε_b parameter has been introduced to take into account the scale effect of the size of the failure mechanism of a pile base relative to the failure mechanism of the sounding cone. From an extensive research program in the tertiary overconsolidated Boom clay (De Beer, e.a, 1977), it was found that the following is approximately true:

$$0.476 \le \varepsilon_b \approx 1 - 0.01(D_b/d - 1)$$

with : D_b resp. d = diameter of pile base resp. of sounding cone.

The α_b coefficient has empirically been deduced from static pile load tests in various research projects. An overview of values, commonly used in the Belgian design practice, is given in table 1.

It should be emphasized that the above mentionned empirical α_b coefficients have been deduced :

(1) on basis of correlations with CPT's executed with the mechanical cone of type M4 or the electrical E1 cone;

(2) refers to the conventional rupture load for displacement piles, corresponding to s_b/D_b = 10 %, and to the critical rupture load for bored piles, corresponding to s_b/D_b = 5 %.

Pile type	Table 1 : α_b coefficient for	
	sand	stiff O.C. clay
Cast in situ driven displacement pile with low dry concrete (Franki expanded base)	1.15	1.0
Cast in situ driven displacement pile with bottom plate and plastic concrete	0.8-1.0 (1)	0.8
Driven precast concrete pile	1.0	0.85
Cast in situ screwed displacement pile with double soil displacement (Atlas type)	1.0	1.0
Bored piles (large diameter and CFA)	0.33-0.8(2)	0.8

(1) : function of diameter of the bottom plate relative to diameter of driving tube;

(2) : for bored piles in soft cohesive soil and in sand De Beer-Van Impe (1977) proposed to determine $\alpha_{b,z}$ out of CPT results by:

$$\alpha_{b,z} = 0.8 - \left(\alpha_{d,o} - \alpha_d\right) \times \left[\frac{q_{u,b(z)}^{(m)} - q_{u,b(\min)}^{(m)}}{q_{u,b(\max)}^{(m)} - q_{u,b(\min)}^{(m)}} \right]$$

5.2.2. ULTIMATE SHAFT FRICTION

Estimation of the ultimate shaft friction is based on one of the following CPT values :

1. the total skin friction $F_{s,z}$ from the CPT;
2. the cone resistance q_c;
3. the local skin friction $f_{s,z}$ from the CPT.

Ultimate shaft friction deduced from $F_{s,z}$

The easiest way for evaluating the total pile shaft resistance $Q_{u,s}$ remains related to the (cumulative) measurement of the total skin friction $F_{s,z}$ from the CPT :

$$Q_{u,s} = \xi_f \times F_{s,z} \times \frac{D_s}{d}$$

with : ξ_f = a parameter depending on soil and pile type;

D_s resp. d = diameter of the pile shaft resp. of the sounding rod.

The ξ_f coefficient should be seen as the product of mainly three factors, α_s, β_s and ε_s.The ξ_f parameter is to a large extend influenced by the method of installation of the pile (factor α_s) , which defines the densification or eventual loosening of the soil in the vicinity of the pile, as well as the stress state around the pile after installation. Also the nature of the lateral surface of the pile (steel, rough or flat concrete, ...) has a consistent influence on the shaft friction (factor β_S). Finally, scale effects related to the soil structure (such as the fissuring of the soil) may also be an influencing factor on the overall skin friction (factor ε_S).

Table 2 gives indications on the design values for the overall factor ξ_f, deduced from various research work.

Pile type	Table 2: ξ_f coefficient for	
	sand	stiff O.C. clay
Cast in situ driven displacement pile with rammed dry concrete (old Franki type)	1.75	1.15
Cast in situ driven displacement pile with vibrated plastic concrete	0.8-1.0	0.65
Driven prefabricated concrete pile	1.0	0.85
Cast in situ screwed displacement pile with double soil displacement (Atlas type)	1.25	1.25
Rammed or screwed steel pipe piles	-	0,45-0,55
Bored piles (large diameter and CFA)	-	-

Altough the F_S approach is very simple, it should not however be overlooked that the assumed linear correlation between total friction on the steel "jacked" CPT sounding rods and the total friction on the pile shaft should be corrected on some occasions. Such cases include, for example, the following :

* deviation of the sounding rods or the use of unsmooth rods (by the presence of e.g. the ball-clamp used for the extraction of the rods) often leads to higher (unsafe) CPT friction values;
* it is often experienced in particular sand layers, even being medium dense to dense, that the increase of F_S is very moderate and even zero; this might be explained by some buckling of the sounding rods, leading to certain widening of the penetration hole, to some re-arrangement of the grain structure, or to very low horizontal contact stresses between the rods and the surrounding soil; these phenomena occur in particular in dry, somewhat cemented sands;
* rod buckling and relaxation also are reflected in the CPT values by a somewhat delayed drop in the F_S value after penetrating through a dense sand layer.

Ultimate shaft friction deduced from q_c

The pile shaft friction can also be evaluated on a semi-empirical correlation between the cone resistance values q_c and the ultimate unit shaft friction $q_{u,s}$:

$$Q^{+}_{u,s} = \pi\, D_s \times \sum H_i \times \eta_p \times q_c$$

$$q_{u,s} = \eta_p \times q_c$$

Values for η_p have been reported by Van Impe (1986, 1989) and De Cock (1993).

Ultimate shaft friction deduced from $f_{s,z}$

A third method relates the pile shaft friction to the unit skin friction on CPT tubes, by :

$$q_{u,s} = \alpha_s \times f_{s,CPT}$$

Again, α_S depends largely on the installation method of the pile and the consequent stress state around the pile, but also (Carpentier e.a., 1985) :

* on the nature of the soil layers, and for cohesive soils on the rigidity of the soil;
* on the nature of the skin of the shaft : steel or concrete;
* for concrete, on the way the shaft is fabricated: precast shaft, shaft constructed in situ with tamped dry concrete, shaft constructed in situ of vibrated wet concrete.

Although the $q_{u,s}$ approach based on $f_{s,CPT}$ should be less subject to parasitic phenoma than the case of total $F_{s,CPT}$ values, only little experimental data are available in Belgium, so that actually, this method is not widely used.

The unit skin friction $f_{s,CPT}$ may be directly measured in the CPT using mechanical or electrical friction sleeves, or may be derived from the q_c values. In the latter case, the following rules are often used in Belgium.

For cohesionless quartz sands (Carpentier & al. 1985) :

$$f_{s,CPT} = \frac{q_c}{200} \quad for \quad q_c \geq 20\ MPa$$

$$f_{s,CPT} = \frac{q_c}{150} \quad for \quad q_c \leq 10\ MPa$$

For intermediate values of q_c, the value of $f_{s,CPT}$ is calculated by linear interpolation between the values q_c:200 and q_c:150.

For cohesive soils with a small rigidity index :

$$f_{s,CPT} = \frac{q_c}{15}$$

For stiff clays :

$$f_{s,CPT} = \frac{q_c}{36.6}$$

6. COMPARISON WITH OTHER METHODS

There seems to be no direct comparison possible with the pressuremeter methods.

7. MAJOR AREAS FOR RESEARCH ACTIVITIES

* Atlas piles in stiff clay : performance, stress measurements
* Steel piles
* PCS and Omega pile
* Static and cyclic model pile tests in sand (Kanai)
* Prediction of pile settlement (Verbrugge)
* Different behaviour of bored and driven piles

8. FUTURE TRENDS AND NEW DEVELOPMENT

In general, the mechanical CPT will more and more be replaced by the electrical cone when soil conditions or improved equipment allow the safe use of the CPTE. Related to the electrical measurement and digital storage of the measurements, automatic and faster presentation and analysis of tests results will become a rule. An important condition for digital data collection and exchange is the use of a uniform data format. In Belgium, a Geotechnical Data Interchange Format (GDIF) was presented already several years ago (Nuyens, 1989). This format will become of utmost importance for flexible data exchange between site investigation companies and design offices in the future. The GDIF format is suggested in the CPT recommendations mentioned above. Furthermore, the GDIF will allow a fast and reliable means to transfer data for research and mathematical analyses. Thus GDIF will play a key role in future research, performance evaluation and methodology improvement.

New types of cones such as the seismic cone, the resistivity cone, the environmental cone, ... are not (yet) commonly used in Belgium. Although some companies are considering buying such equipment, the opportunity of such an investment for the moment remains unclear. The use of this equipment up to now is mainly restricted to (university) research centres or the regional governmental geotechnical departments. With the growing importance of site investigation techniques for quick and reliable screening of possible polluted sites, some of these cones will certainly gain importance and will more generally be used in Belgium. Certainly more research will be needed to fully implement these techniques in Belgian geotechnical practice, in Belgium but also abroad.

With respect to the environmental site investigation, test methods to detect groundwater pollution are - although very limited - already in use. The CPT probes to withdraw water samples at different soil levels become more important. A possible area of research will be the reliability of the results considering the possibility of cross contamination which can occur because of the polluted water remaining in the filter element of the water suction device.

At the laboratory of Soil Mechanics of Ghent University, a research project is running on the use of the acoustic emission technique with penetration testing. In a first stage of the project, the parameters influencing the AE signal were investigated in the laboratory. Therefore two special test set-ups were designed and a AE needle apparatus and a prototype AE cone were built. It is planned to prepare a prototype AE cone for field testing. The AE signal generated in the soil during penetration of the cone is considered as additional information with the classical CPT data which will also be measured. As a result of the first stage of the project is was concluded the AE cone is a highly sensitive apparatus, measuring events being generated at grainsize level. The AE signal helps in identifying more clearly soil type, grain size, (micro)layering, density, shear resistance, ... The main advantage of this research equipment seems to be its possibilities in defining different soil structures and ageing effects in non cohesive material. More details about this research are given in the paper (Mengé and Van Impe, 1995) also presented at this conference.

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Cone Penetration Testing in Brazil - National Report

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SYNOPSIS: This report attempts to summarize the Brazilian geotechnical experience on cone penetration testing including design methods , engineering practice and research.

In despite of having been introduced in Brazil since the middle of the fifties, the cone penetration test did not play a major role in the site investigation programme and have been used primarily as an alternative in situ test to estimate the bearing capacity of piles.

In recent years this has progressively been changed, mainly as a consequence of systematical research programmes established in major Universities which, by providing reliable electric cone and data acquisition and processing systems, has contributed to improve the Brazilian practice.

1. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION

Brazil is a continental country that has a land surface of about 8.5 million square kilometers; this vast area encompasses a great variety of geological environments and a wide variation of climate (see Figure 1 for reference). An appreciation of the importance of geology on the different soil formations is beyond the scope of this report, so the aim is to summarize some of the geotecnical features found in the country.

The variety of natural deposits encountered is almost limitless, ranging from sedimentary sands, clays, silts and tills at the highly populated coastline to extensive in situ weathered soil profile (the so-called residual , tropical, lateritic and saprolitic soils). The in situ weathering processes cause development of a sequence of horizons within a soil profile (horizons A, B and C, according to Vargas (1971) and Mitchell (1993)), their thickness ranging from less than a meter to more than 100 meters. Such soil profiles are formed in place in response of the combining action of physical, chemical and biological weathering process on the local parent rock (basalt, granite, and gneiss).

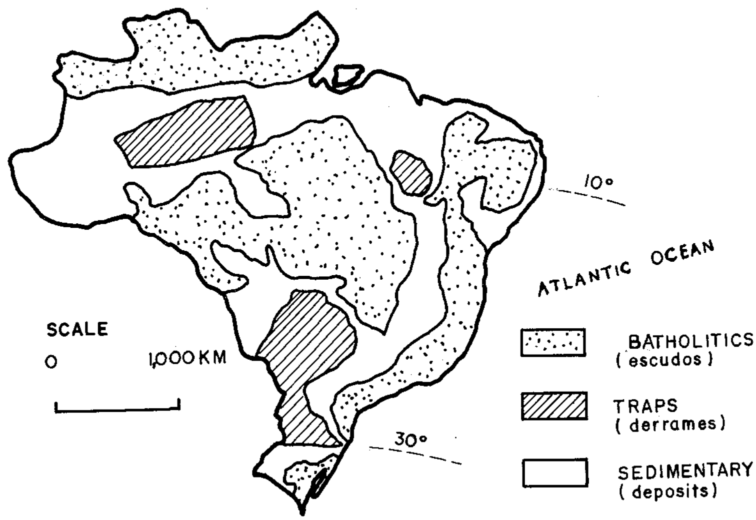

Figure.1. Main geological features occurring in Brazil

2. TYPE OF PENETRATION TESTING AND OTHER INVESTIGATION METHODS

Soil investigations and determination of geotechnical parameters in the Brazilian practice are dominated by the use of standard penetration test (SPT). SPT based correlations are preferred as a design method for the vast majority of foundation problems. Consulting contractors and private companies rely on SPT results as the main source of soil data , whereas State Companies are the only group where the SPT covers less than 80% of global use (Milititsky and Consoli, 1989).

Other in situ testing techniques and laboratory testing are not widely used in this country, except for major design projects in which a more sophisticated analyses of soil-structure interaction is required, such as : offshore platforms, earth dams, subways, heavy industrial plants, large bridge, power plants, etc. or in case of particularly adverse subsoil conditions. Commercially available geotechnical investigation methods in Brazil also includes: laboratory tests (oedometer, direct shear and triaxial) and field tests (mechanical cone, vane, plate loading , Menard pressuremeter and Marchetti dilatometer). In special and more complex investigation programmes use have been made of instrumented laboratory triaxial tests and field tests including electric penetrometer, piezocone and self boring pressuremeter.

Penetration tests for geo-environmental purposes with electric resistivity / conductivity measurements or contaminated soil and water sample have not been yet incorporated in the Brazilian engineering practice.

3. TYPE OF CPT - EQUIPMENT

One of the first technical publication presenting details of a cone penetrometer device and equipment, used in Brazil, is dated of 1970. However, there are others evidences, registered as internal or contract reports, which indicate that many of cone penetration device, equipment and test procedures developed in Netherlands, were also introduced in Brazil since the middle of the fifties. These, consisted basically of mechanical penetration with both cone and push rods being pressed down by a simple manually operated system.

In this mentioned reference, Aoki and De Brito (1970), describe a Dutch cone penetrometer, manufactured by N.V. Gouda Maschinenfabrieck (patent nº 101.924), bought the by Brazilian branch of the Franki Pile CO. This equipment, with a maximum thrust capacity of 175 KN (17,5 metric ton), has a hydraulic jacking system for pushing down and pulling out the penetrometer and push rods. Two types of mechanical penetrometers were used. The mantle cone, originally developed by the Delft Soil Mechanics Laboratory (Vermeiden, 1948) and the friction sleeve cone penetrometer, as designed by Begemann (1957). Both cones with a maximum point resistance of 70 KN; a base area of 10 cm^2 and an apex angle of 60°. The friction sleeve locates above the conical tip and of the same diameter has a standard area of 150 cm^2. These mechanical penetrometers use a double rod system (inner solid rods with 15 mm diameter and push thick-walled rods with 35,7 mm-ED/16mm-ID) respectively, to extent the tip and to advance the penetrometer to the required test depth, incrementally operated using a telescoping action. Two Bourdon pressure gauges, using different scales, were used to measure the penetration resistance at the surface. Following the pioneer step taken by Franki Pile, some others Brazilian site investigation and foundations specialist contractors, have also acquired cone penetration equipment, whose, in most cases, were manufactured in this country. The Dutch cone test, became commercially available in Brazil and was used, basically, as an alternative in situ test to evaluate the bearing capacity of pile (pile design). Maybe, as a consequence of this specific use, no systematic research was carried out, aiming to new developments of : equipment and device; test procedure or method for evaluating soil properties.

This situation remained as such until the middle of the seventies, which, motivated primarily by the offshore oil activities and secondary by the inland pile design requirement for tall buildings, there were an increase of interest in the use of cone penetration test.

For the offshore site investigation, which required a more sophisticated and expensive new equipment, that was not available in Brazil, the Brazilian State Oil Company - Petrobrás, aiming to promote the transference of technology, has led contract involving both a national and a foreign contractors.

Bogossian and McEntee (1978) report results of the first electrical cone test used for offshore site investigation in Brazil, under the direct association of Geomecanica and Wimpey Companies. Electrical cone penetration tests, were carried out in water depths of 140 meters, being remotely controlled from the deck of M.V. Wimpey Sealab Vessel. The basic jacking system consisted of a small diameter hydraulic cylinder with a 3 meters stroke piston, that was lowered in the drill pipe and latched mechanically in the bottom section, which from its weight provided the necessary reaction for the test. For these series of tests, a conventional electrical cone, with point and friction sleeve measurements was used. The recording system consisted of : chart recorders, data logger and printer. It is also indicated that depths ranging from 110 to 150 meters below sea bed were reached.

Other offshore site investigation programmes, using the same association policy, were carried out, involving the use of downhole penetrometers, with both jacking system based on the wireline system for water depths up to 500 meters and on the McClelland Dolphin system, which required the use of wireless penetrometers, for deeper water. In one of this offshore site investigation programme a seismic cone was used.

It is noteworthy the experience obtained by a totally national enterprise sponsored by Petrobrás, involving Geomecanica Company and the Pontifical Catholic University of Rio de Janeiro (PUC-RJ), which have carried out cone penetration tests in water depths of 210, 600 and 1120 meters. For these series of test, a non-conventional piezocone (end resistance and pore pressure measurements), with 20cm^2 base area and an apex angle of 60^0, developed at PUC-RJ (Rocha-Filho, 1986), was used. This penetrometer was fixed at the bottom section of the drill pipe, using a simple mechanical latching system provided by Geomecanica and the penetration process was entirely due to the selfweight of the drill pipe column. With this type of non-controlled rate of penetration process it was obtained, for these three sites consisting of soft clay, penetration depths of 92; 23.5 and 22.5 meters and has provided useful information in terms of geotechnical parameters for the well-head foundation design.

In this period of time and for inland site investigation, the novelty came with a heavy duty truck mounted rig, provided by a branch of the Dutch Van den Berg Company, offering commercial services including electric and the so called hydraulic penetrometers. Such hydraulic penetrometers use a fluid pressure transducers to register and to transmit to a recording unit the penetration resistance. This initiative step was latter incorporated by a national company, which has, since then, established with some success, indicating a clear demand for such specific service and also a need for a systematic research programme involving developments, improvements, standardization and application of the cone penetrometer test.

In Brazil, the first systematical research programme related to electric penetrometers, was established at PUC-RJ in the end of seventy's decade. This research programme has led to developments of conventional (in conformity with the ISSMFE standards)and non-conventional electric penetrometers, including data acquisition and processing system. Large ($20cm^2$ base area) and mini-piezocones ($2.0cm^2$ base area), respectively used for offshore deep water (Rocha-Filho and Sales, 1994) and for laboratory tests (Sales, 1988), have been developed. Penetration tests were carried out in several soft clay deposits (Rocha-Filho and Alencar, 1985 and Rocha-Filho, 1987) and in structured gneissic residual soil (Rocha-Filho and Carvalho,1988). In a specific project sponsored by Petrobras, a piezocone (see figure 2) and a data acquisition system were developed, to be used in waterdepths up to 2,000 meters. A high pressure calibration chamber was built to check the watertightness and to establish the end area ratio for correction of recorded cone resistance. Calibration procedures of the assembled penetrometer were carried out for confining water pressure ranging from 0 to 20MPa.

Consistent research activities have also being developed at the Graduate Programme of both Federal Universities of Rio de Janeiro (COPPE-UFRJ) and of Rio Grande do Sul (CPGEC-UFRGS), including development of equipment and extensive test programmes.

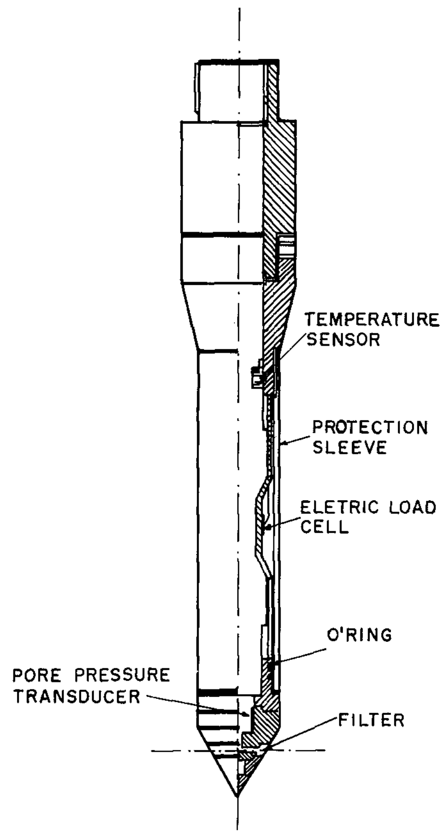

Figure.2. Piezocone for deep water off-shore investigation developed at PUC-RJ

Following the international trend, the interest in the use of electric cone penetration tests for site investigation in Brazil, has gradually increased. The availability of reliable electric cones and data acquisition system

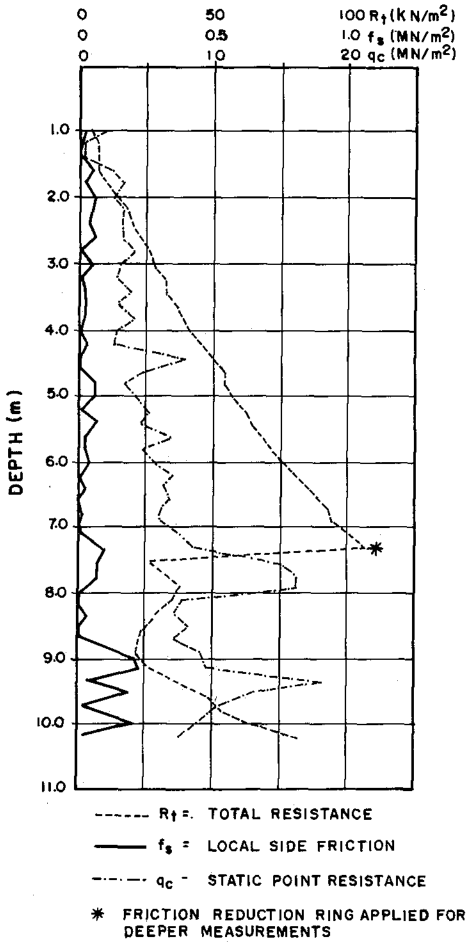

Figure.3. Typical presentation of mechanical CPT result (apud: ABEF, 1989)

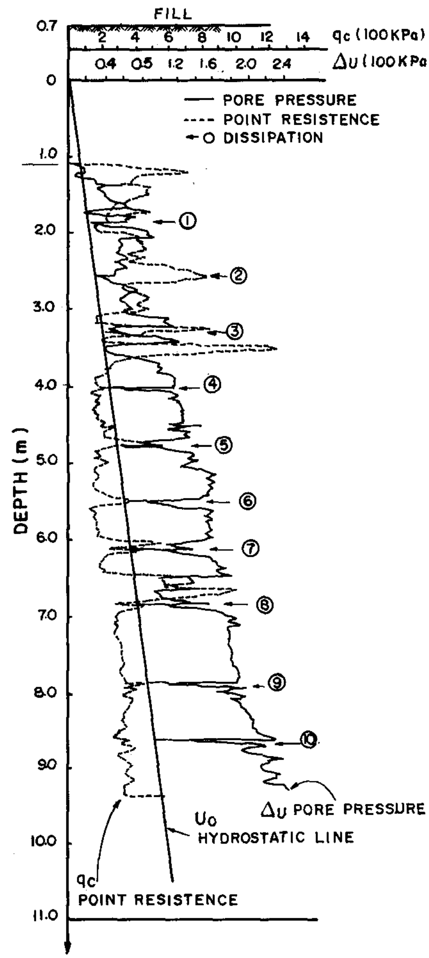

Figure.4. Typical presentation of piezocone test result

mainly provided by these researches programmes, is contributing to improve the Brazilian practice.

Typical presentation of test results are shown in Figures 3 and 4, respectively, obtained with a mechanical penetrometer and a piezocone. In the test results it is also indicated the type of penetrometer tip used and the calibration certificate for the reading unit, such as the Bourdon pressure gauge for the mechanical penetrometer. In case of piezocone tests carried out in clayey and silty deposits, where significant pore pressure are generated during penetration, correction is applied to the recorded point and friction sleeve resistance, in order to account the unequal end area effect. As well established, this correction are based on the net area ratio and on the total pore pressure value at a reference filter location.

The current edition of the Method of test for the CPT, designated as MB-3406, has been approved by the Brazilian Society for Technical Standards on June/1991. This Brazilian Standard covering: scope; definition; apparatus; test procedure including special technique and precautions and report of results, has been based on the ASTM Standard D 3441 (1986).

4. INTERPRETATION OF TEST RESULTS.

4.1 Soil classification and stratigraphy.

It is not common practice in Brazil to use the cone test result to classify the soil and to determine the stratigraphy. But in some site investigation programme using friction-cone penetrometer, attempt has been made to classify the sub-soil using the friction ratio (fs/qc%) and the empirical relationship initially proposed by Begemann (1965) and latter extended by Schmertmann (1978) or other more complex proposal based on the comprehensive work presented by Douglas and Olsen (1981). As expected this procedure have been applied with some success for uncemented sandy and silty soils but has been found to be of limited value when applied to structured residual soils. In residual soils, the structure is closely related to the weathering degree which, acting in a differential process, produces a highly non-homogeneous soil profile. For this type of soil, both cone and friction resistance values are strongly dependent on the weathering degree and less influenced by the granulometry, as the proposed empirical relationships based on the friction ratio.

Using a different approach, results of piezocone tests carried out in soft clays have being used for soil classification together with several classification charts based on the corrected total cone resistance (q_t) and on the pore pressure parameter ratio (B_q) such as proposed by Jones and Rust (1982); Senneset and Janbu (1984) and Robertson et al (1986). Values obtained in several soft clay deposits (References: Rocha-Filho and Alencar (1985) and Soares et al (1986) for the Sarapui deposit; Rocha-Filho (1987) for the Jacarepagua deposit; Coutinho et al (1993) for the Recife deposit; Soares et al (1994) for the Porto Alegre deposit and Arabe (1995) for the Quilombo deposit), have indicated that the classification chart proposed by Robertson et al (1986) provides a more consistent correlation with the type of soil tested. But, this local experience also indicates that some adjustment should be necessary on these classification charts.

4.2 Undrained Shear Strength

The undrained shear strength (S_u) of cohesive soils have being estimated using piezocone results and the classical bearing capacity equation expressed as :

$$S_u = \frac{(q_t - \sigma_{v0})}{N_{kt}} \qquad [1]$$

Where:

q_t is the total corrected cone resistance.

σ_{vo} is the total vertical overburden pressure.

N_{kt} is the cone bearing capacity factor.

Attempts have being made to determine the N_{kt} value for some soft clay deposits using reference value for S_u obtained from other types of field or laboratory tests. For the Sarapui soft clay deposit value of the N_{kt} ranging from 13.5 to 15.5 was determined by Rocha-Filho and Alencar (1985) using reference S_u values obtained from corrected field vane tests presented by Ortigao et al (1983) and results from laboratory triaxial tests reported by Costa-Filho et al (1977). For this same site, Soares et al (1986) and Danziger (1990) using results from a new series of piezocone tests have confirmed the N_{kt} value within this range. Rocha-Filho (1987) has found an average value of 14.5 for the Jacarepagua site, using reference value for S_u obtained from field vane tests reported by Bogossian et al (1982). Using the same procedure, Soares et al (1994) have obtained an average N_{kt} value of 15 for the Porto Alegre soft clay deposit. Coutinho et al (1993) have determined values of N_{kt} ranging from 10.3 to 15 for the Recife soft clay deposit , adopting the reference value for S_u from laboratory triaxial tests. Arabe (1995), also using field vane for reference values of S_u has found for the Quilombo soft clay deposit an average N_{kt} value of 15. This value was confirmed using the theoretical equation proposed by Houlsby and Teh (1988) and soil parameters obtained from laboratory triaxial tests. Adoption of N_{kt} value about 14 has provided good estimate of the undrained shear strength of marine clay deposits from piezocone tests carried out in deep water-offshore (Rocha-Filho and Sales, 1994) and in shallow water-harbour (Brugger et al, 1994). Hence, based on this experimental investigation it can be concluded that, for the Brazilian soft clay deposits, it can be recommended an average N_{kt} value of 14 within the range of ± 20%.

4.3 Stress History

The correlation between the over consolidation ratio (OCR) and the pore pressure parameter ratio (B_q), as indicated by Wroth (1984), have been used for the evaluation of the stress history of most of this soft clay deposits. Based on this proposal, values of OCR ranging from 1 to 2 have been found, indicating normally to lightly over-consolidated clays, which are in agreement with the geological formation of most quaternary soft clay deposits that occur along the Brazilian cost.

4.4 Flow Characteristics (Coefficient of Consolidation)

Approximate values of the coefficient of consolidation in the horizontal direction (C_h) were obtained, using results of dissipation pore pressure recorded in piezocone tests for some of the Brazilian soft clay deposits. For the Sarapui site, values of C_h ranging from 2.6 to $10.2x10^{-3}$ cm^2/s, were obtained by Rocha-Filho and Alencar (1985) using the method proposed by Torstensson (1977), based on 50% dissipation test. Such values are higher than values of the C_v (in the vertical direction), ranging from $2.5x10^{-4}cm^2/s$ to $3.0x10^{-3}cm^2/s$ obtained from oedometer tests by Costa-Filho et al (1978). Coutinho et al (1993) reported C_h values, for the Recife soft clay, of 8.8 to $16x10^3cm^2/s$, based on Houlsby and Teh (1988) proposal, which are 2 to 4 times higher than values obtained using laboratory oedometer tests, ranging from 2 to $6x10^{-3}cm^2/s$ (Coutinho, 1980). In a more recent work, Arabe (1995) obtained for the Quilombo soft clay deposit, C_h values of $8.3x10^{-4}cm^2/s$ using the method of Baligh and Levadoux (1980) and $4x10^{-4}cm^2/s$ using the method developed by Houlsby and Teh (1988).

5. USE OF CPT IN GEOTECHNICAL DESIGN

Geotechnical *in situ* testing methods have traditionally been interpreted in Brazil by direct approaches, in which field values are directly related to the performance of foundation without the need to evaluate the respective soil parameter. Considering the simplification and approximation inherent adopted to account for scale effects and data fitting analysis, respectively, these empirical proposals can be of rather restrict use. The need is stressed for relationships to be based on experience and local practice, solely in this case such approach can be of some practical value.

The number and quality of the case records upon which the Brazilian practice has been established is very limited, particularly in weathered structured soils. Velloso (1982) has observed that friction sleeve resistance from mechanical cones in submerged residual soils can be up to 70% higher than in submerged sedimentary deposits. Prezzi (1990) reported a successful experience on predicting bearing capacity of small diameter piles load-tested on a structured soil site, in which standard design rules interpretation of cone tests were evaluated (Heijnen, 1974; Schmertmann, 1978). Rocha-Filho and Carvalho (1988) reported cases in residual soils in which the shaft resistance for displacement piles are higher than for bored piles at least by a factor of 2. This cumulative experience can be used to emphasize the complexities in simulating soil-structure interaction in structured residual soils and consequent difficulties to incorporate such aspects to foundation analysis and design.

Attempts have also been made to estimate bearing capacity of piles from cone data based on purely empirical relationships (Aoki and Velloso, 1975). The authors acknowledge that in Brazil CPT tests are not as largely used as SPT tests. Therefore, despite of being conceptually developed for CPT testing interpretation, the method became known as a direct correlation between SPT blow count numbers and bearing capacity.

For design of shallow foundations, allowable bearing capacity is generally governed by settlement considerations. Direct methods to estimate settlements of shallow foundations based on statistical analysis are a current practice in Brazil (Burland et al, 1977; Burland and Burbidge, 1985); these methods were established in the domain of cohesionless drained materials and very few publications about foundation performance on structured soils have been issued up to now to justify their general acceptance. Cudmani et al (1994) carried out a research programme that comprises cone, SPT, pressuremeter, and laboratory tests, as well as plate loading tests on steel plates and real scale load tests on concrete footing. The authors suggested that Burland's correlation (Burland et al, 1977) can provide useful information on natural cemented soil conditions. Further developments from a number of well recorded cases linking field performance to penetration test data are clearly required to support this practice.

6. COMPARISONS AND CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

Limited budgetary and time constrains often result in poor subsurface investigation and difficulties in assessment of required geotechnical design parameters. The use of combined *in situ* testing techniques can improve design methods considerably, specially in subsoil conditions which are atypical to routine geotechnical practice such as unsaturated deposits.

With the increasing use of CPT, some attempts have been made to established correlations between tip cone resistance and SPT blow count N value. Presented q_c/N ratios provide a guideline to convert the measured q_c to the equivalent N value, which can be used as input for empirical SPT based correlations. Significant scatter is always observed around average q_c/N ratios despite application for SPT energy correction. The equivalent N value is therefore less accurate then a measured N value, increasing the inaccuracy of predictions based on available relationships developed empirically from SPT previous experience. It is the author's opinion that this practice should be used with considerable caution.

Cross-information of different *in situ* testing results requires an understanding of the relative merits of each technique adopted for site investigation and careful combination of the most relevant information provided by each test. In this area, a contribution that the Brazilian geotechnical community is now giving is related to interpretation of *in situ* testing in structured unsaturated soils. Some important features related to the topic are illustrated in typical soil profiles presented in Figure 5, in which independent measurements of tip cone resistance q_c, pressuremeter limit pressure ψ_l and SPT blowcount numbers N are plotted against depth. For the site, it can be seen that all three types of measurement provide consistent results, but that the continuous cone penetration provides the most detailed information on the soil profile. As for the gneissic residual soil, the detailed determination of the cone subsoil stratigraphy highlights several local characteristics that are not identified by the SPT profile, as indicated by Rocha-Filho and Carvalho (1988). The potential of the cone penetrometer to identify spatial variability in saturated soils is already established, and it appears that a similar potential exists also for the inherent vertical and spatial heterogeneity of the soil properties observed in structured unsaturated deposits.

The interpretation of a penetrometer profile in terms strength parameters largely relies on empirical correlation and it can be anticipated that, given the greater complexity of unsaturated soil behaviour, and the difficulties of expressing the constitutive relationships, the penetrometer is unlikely to be a satisfactory instrument for assessing soil parameters in such soils. The complete load-displacement response of the soil measured *in situ,* as given by pressuremeter and plates loading tests, appear to be the best possible complement to cone profiles, leading to strength and stiffness estimates (Schnaid et al, 1995).

7. MAJOR AREAS FOR RESEARCH ACTIVITIES

Although significant developments have taken place in Brazil as a result of systematic research established in main Universities, there is still a need for continuing development of reliable electric cones and data acquisition system.

Application of CPT in structured residual soils become the most important item for research activities in Brazil. This, mainly associated with determination of soil stratigraphy and direct application of results in piles design and assessment of settlement of shallow foundation. In residual soil, the potential for use of combined static penetration devices, such as the cone-pressuremeter and the seismic cone is very attractive for, both, application to geotechnical design and evaluation of fundamental soil properties.

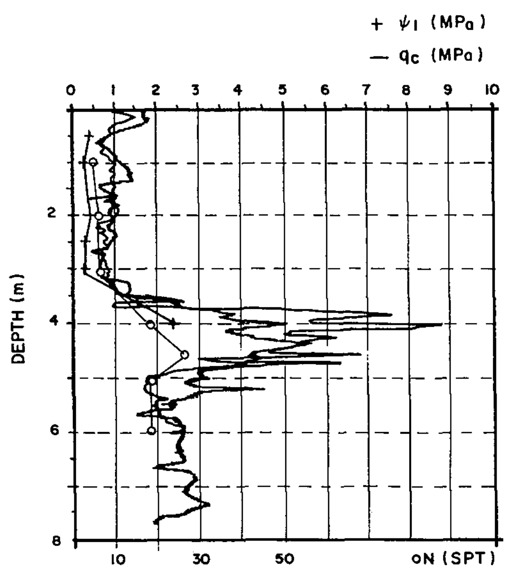

Figure.5. Eletric CPT, SPT and Pressuremeter results

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CPT National Report - CANADA

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SYNOPSIS: This report provides a general but brief description of the state of practice for cone penetration testing (CPT) in Canada. The comments contained in this report may not represent the views of all practitioners, however, the comments are based on 15 years of Canadian experience on over one thousand CPT site investigations.

1. INTRODUCTION

Canada is one of the largest countries in the world extending over 5,000 miles from coast to coast. Both the geography and geology of the country are extremely varied encompassing coastal plains and vast mountain ranges. Only a relatively small portion of this vast country has geological conditions suitable for CPT based site investigations. The country is also sparsely populated with less than 28 million inhabitants.

Cone penetration testing is generally limited to softer soils, such as, clays, organic silts, peat, silts, sands and clay tills and clay shales. In Canada, most of these types of soils are generally found on our coastal margins, river deltas, alluvial and lake deposits and in our mine tailings disposal areas.

At the present time western Canada and specifically the province of British Columbia (B.C.) carry out the most CPT based site investigations. The CPT is starting to show signs of increased application in other parts of Canada within the last 5 years. There are currently approximately six companies offering CPT related services in Canada with all but one of these located in the Vancouver region of British Columbia. Three Universities in Canada have cone penetration testing equipment; the University of British Columbia, Carleton and Laval.

2. SITE INVESTIGATION TECHNIQUES

The Standard Penetration Test (SPT) is the most dominant in-situ test for site investigations in Canada. In general, the CPT is used about 10% of the time in penetrable soils. In some regions this number may increase somewhat. The limited use of the CPT is largely due to a lack of education and experience by practicing engineers in the area of CPT technology.

Cone penetration testing in Canada is carried out primarily by specialist site investigation contractors such as; ConeTec Investigations Ltd. and Hughes In-Situ Ltd. These companies have highly skilled professional engineering staff and design, build and operate all of their own CPT equipment. Several geotechnical consultants and two drilling contractors also own and operate CPT systems. However, generally the consultants and drilling contractors have experienced problems of quality control and equipment maintenance and have often not continued to operate the equipment.

Most site investigations in Canada are supervised by experienced geotechnical engineers supplied by the geotechnical consultant. Borehole logs and field reports are prepared by the geotechnical consultant through their field supervision. CPT based site investigations also tend to be supervised by the geotechnical consultant.

3. TYPES OF CONE PENETRATION TEST EQUIPMENT

Mechanical cones are used very little in Canada. The most common CPT equipment is the electric cone penetration test with pore pressure measurement (CPTU). The most common location to measure the pore pressure is between the cone tip and the friction sleeve (u_2). Most cones allow the location of the pore pressure sensor to be moved to the mid-point on the face of the cone (u_1) during a site investigation if stiff soils are encountered. The second most common penetration test is the seismic cone penetration test with pore pressure measurement (SCPTU). There are also a variety special penetrometers which involve the addition of specific modules behind the standard piezo cone, such as:

- resistivity modules
- chemical sampling modules
- ground water and soil vapor sampling modules
- pH modules
- moisture content modules

Most cones also have slope sensors to record the change in inclination during a sounding.

There are no specific equipment calibration standards in Canada. The calibration of the equipment is carried out by the specialist contractors who generally maintain detailed records of the equipment to ensure good quality control. Detailed records are maintained of the zero load readings before and after each sounding.

The cone data are routinely corrected for:

- zero load shift
- temperature
- non-linearity in calibration
- unequal end area effects, and, more recently,
- overburden stresses.

The most commonly collected CPT parameters are:

- q_c penetration resistance
- f_s sleeve friction
- u penetration and equilibrium water pressure
- i inclination

Seismic wave velocity measurements and specifically shear wave velocity (V_S) is routinely recorded during CPT based investigations on the west coast of Canada and on large projects where static or dynamic liquefaction of sands is a concern. Shear wave velocity is also routinely collected during site investigations involving vibrating machine foundations. Bulk resistivity using the resistivity cone penetration test (RCPTU) is another parameter which is recorded during environmental site assessments usually in conjunction with the collection of penetration type groundwater sampling systems.

CPT data are usually collected and presented in real time during site investigation through the use of computers, monitors, plotters and printers. Some operators now present CPT results in color. Not only are the basic cone parameters (q_c, f_s, u) displayed and plotted in real time, but some operators also compute and display the soil classification and selected engineering design parameters in real time. These selected design parameters can then be copied directly into specialized geotechnical design software and such things as pile capacity can be estimated on site during the field investigation.

At the present time, Canada does not have its own CPT standard. Generally when a standard is required, either the ASTM or the ISSMFE standards are employed and in general these are found to be adequate. Geotechnical practice in Canada relies very little on standards. There is no design code for geotechnical design.

With the rapid advance of computer technology, the presentation of CPT data begins in the field in real time, during testing. In the field color monitors can display q_c, f_s, u, and friction ratio (R_f) as well as resistivity and pH data versus depth. The displayed data on the monitors can be sent to color printers and a hard copy generated at any point during a pause in the penetration process. CPT operators generally supply the final CPT plots along with a computer disk containing the CPT data to the geotechnical consultant.

4. INTERPRETATION OF CPT DATA

Soil type and stratigraphy are interpreted primarily using charts developed by Robertson, (1988 and 1990). These charts have been used across Canada in a wide variety of soil conditions and have generally proven to interpret soil behavior type reliably. Site specific adjustments to these charts are easily made using interpretation software packages (e.g. Greig, 1995). However, adjustments are not often required in the Canadian experience.

Most of the basic geotechnical design parameters can be estimated based on CPT data either directly or on the basis of many well established empirical corrections (e.g. Robertson and Campanella, 1983). Modification to these various correlations are made based on local experience. The most common parameters estimated from CPT data are:

- relative density of sands (Dr)
- undrained shear strength of clays (s_u)
- friction angle of sands (f')
- equivalent SPT N value
- soil modulus (E and G).

Unlike in the United States, penetration testing in Canada is not widely used in geo-environmental site characterization. When the CPT is used for geo-environmental applications the data collected is employed to determine such things as:

- soil stratigraphy
- equilibrium groundwater level
- hydraulic flow gradients
- soil permeability and bulk resistivity
- soil and ground water pH

5. GEOTECHNICAL DESIGN USING THE CPT

Many geotechnical design software packages based on CPT parameters have emerged over the last 10-15 years. The most common geotechnical designs carried out using CPT data are:

- pile capacity (e.g. LCPC Method)
- liquefaction and earthquake design (especially on the West Coast)
- settlement analysis for shallow and deep foundations.

It is expected that design software based on CPT data will continue to develop at a rapid pace.

From an environmental perspective the CPT is also used in the design of:

- groundwater recovery systems
- bio-remediation systems
- groundwater containment and cutoff.

In Canada the application of CPT design for environmental systems is just emerging.

The CPT is still used in Canada for comparison purposes with other in-situ tests. The most common comparison is with the SPT N value. Determining the SPT N value on the basis of CPT q_c is done using correlations developed by Robertson et al. (1983) and Jefferies and Davies (1990).

6. FUTURE TRENDS

It is anticipated that there will be a steady increase in the diversity of CPT equipment and the application of CPT technology in Canada. From the equipment perspective it is anticipated that the following developments will continue:

- heavier CPT pushing units with 30 and 40 ton (300 to 400 kN) push capacity for investigations in stiff ground
- track mounted CPT units for sites with difficult access
- horizontal deployment systems for tunneling applications
- pneumatically powered CPT units for investigations of underground mine waste backfill
- angled deployment systems for the investigation of tanks and reservoirs
- improved low cost CPT deployment systems for the near shore and deep offshore site investigations.

Downhole CPT tools and sensors are developing rapidly. Some of the new CPT tools being developed in Canada are:

- nitrate modules
- dielectric modules
- redox modules
- ultra violet induced florescent modules

These new CPT sensors have a strong environmental.

In Canada the largest potential for growth in the application of CPT technology appears to be in the geo-environmental and mine tailings area. Continued growth will also be seen in the eastern parts of Canada.

The seismic cone penetration test (SCPTU) has been widely used in western Canada for more than 10 years. This test is perhaps the most exciting of any in-situ test for geotechnical applications. With the advancement of our understanding of shear wave velocity (V_S) and the many potential applications of V_S, the increased use of the SCPTU appears to be a trend of the future.

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The Application of Piezocone Tests in China

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SYNOPSIS:The experiences of piezocone tests in China were introduced in this paper. There were five sections included: (1) Machnical and operation. (2) To examine the classification chart proposed by author and to discuss the reliability of the soil profile. (3) To determine coefficient of consolidation C_h and to compare it with the test data measured by laboratory and field observation. (4) The method of settlement analysis using CPTU tests parameters. (5) The possibility for interpreting stress history. Five examples were used to illustrate the reliability of these results.

1. INTRODUCTION

In the early eighties the pore pressure sensor was incorporated in the electrical cone penetrometer which named "Piezocone" (CPTU). As a fast and economical method, it is widely used by many countries.

In China, in the early eighties, the CPTU equipment and testing method were investigated. Nowadays, some experiences were obtained about the equipment, the soil classification, the measurement of coefficient of consolidation, the interpretion of stress history and the method of settlement analysis using the CPTU parameters.

The above-mentioned contents would be presented in this paper. Five examples were selected to illustrate the reliability of these results.

Finally, a simple technique applying CPTU results to interpret stress history would be discussed.

2. EQUIPMENT AND OPERATION

2.1. Mechnical design of the equipments

In order to obtain good CPTU data, the following general requirements should be:

(1) The measured quantities q_c, f_s, u would be independent from each other.

(2) The pore pressure measuring system is designed for a rapid response time.

For these reasons, the following measures have been taken:

(1) Each part, including transducers and filter element, is rigid and has low compressibility.

(2) Silicon strain gauge transducers and special hardened strainless steel filter element were used.

(3) For maintaining saturation of the filter, silicon oil was used in the pore pressure measuring system.

(4) A 3mm thick filter element was located immediately behind the tip.The characteristic properties of probe manufactured in China are summarized in Table.1.

2.2 Operation

The standard penetration rate is 2 cm/sec for all tests.

It is the most important for CPTU tests to maintain the pore pressure measuring system saturated. Both the maximum pore pressures and dissipation times could be affected by air entrapment.General practice is to carefully saturate the filter elements in the laboratory by placing them in a high vaccum with the saturating oil for several hours.

Table.1. Characteristic properties of probe

	Cone	Friction sleeve	Pore pressure
Range(f.s) (MPa)	6, 10, 30	0.3	2.0
Comb. linearity and repeatibility (%)	<1	<1	<1
Meas. system	str. gauges	str. gauges	silicon, str. gauges
Bridge resist. (ohm)	390	390	1000
Excitation (v)	8	8	8
Output (mv/v)	1.5	1.5	9.5
Temp. range (° C)	5~45	5~45	5~45
Diameter (mm)	36.0	36.0	
Sens. area (cm^2)	10	150	
Sens. area for pressure (%)	84	0	100

To obtain the corrected cone resistance q_t, a correction has been applied to account for the design of the cone using the following relationship:

$$q_t = q_c + u(1-a) \qquad (1)$$

in which, net area ratio a is equal 0.84.

3. SOIL CLASSIFICATION CHART

3.1. Background

Four soil classification charts using CPTU data have been proposed in the recent literatures, Senneset and Janbu (1982), Jones and Rust(1983), Robertson and Campanella (1988), Zhang and Greeuw (1990).

Experience shows that there would be some troubles to use Senneset and Jones charts, Robertson chart is better than the preceding two charts, but some scatter exists as well. A new chart is proposed by the authors.

We introduce a reference parameter σ_e, which is defined as the equivalent pressure σ_e, i.e.

$$\sigma_e = 10\gamma h \qquad (2)$$

where h--the depth of soil considered, m
γ--9.81KN/M^3

We use the reference parameter σ_e to normalized the cone resistence.

For Δu, we use a parameter B_p which defined as:

$$B_p = \Delta u/(q_t - \sigma_e) \qquad (3)$$

More than 50 sets of CPTU data all over the world were used to plot the relationship between $\log(q_t/\sigma_e)$ and B_p. Different kinds of soils, such as sand, silty soils and clays, were found to be located into different parts of the chart, as shown in Fig.1. Based on this chart, a new parameter N_h, has been established to distinguish the different kinds of soils.

The parameter is defined as:

$$N_h = 500B_p/\log(q_t/\sigma_e) \qquad (4)$$

Three regions can be classified as:

for clay	$220 < N_h < \infty$
for silty soils	$3.3 < N_h < 220$
for sand	$0 < N_h < 3.3$

It is clear that all parameters used in this chart can be obtained from a CPTU test. N_h value can be computed for any depth and a N_h profile can be plotted continuously. From this figure the soil profile can be estimated immediately.

3.2 Zhang and Greeuw chart was checked in China

Zhang and Greeuw chart was checked in Case 1.

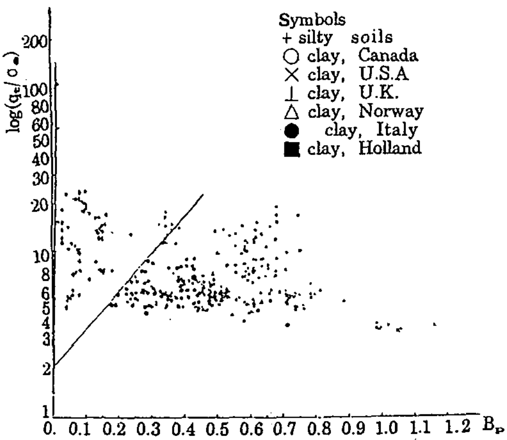

Fig.1 Zhang and Greeuw Classification Chart for the soils in different regions.

It is a small town located near Shanghai. To investigate the improvement methods for highway from Shanghai to Nanjing. 13 plans were performed within a distance of about 2 km. The soil properties of all. layers are summarized in Table.2.

20 sets of CPTU test were conducted in this site, and using these results a same classification chart was obtained as before, see Fig.2.

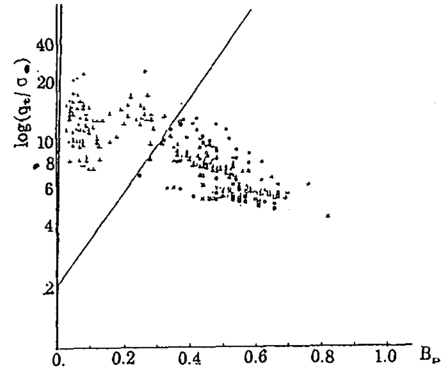

Fig.2 Zhang and Greeuw Chart in China.

3.3 Soil profile

Case 1 is selected for illustrating the reliability of the soil profile obtained from N_h profile. The CPTU tests, vane tests and boring samples were performed close to each other.

The typical results were shown in Fig.3. From this figure we can find a good correspondence between the N_h profile, soil profile of boring samples and vane strength profile.

4. DETERMINATION OF COEFFICIENT OF CONSOLIDATION C_h

4.1 Background

The coefficient of consolidation C_h is a key parameter for settlement analysis and improvement design

Table.2. The soil properties at case 1

SOIL	ω (%)	γ (KN/m³)	L.L.	P.I.	Cv (cm²/sec)	Cc	Pc (kPa)	C′ (kPa)	φ′ (°)	Af
Silty clay	27	19	> ω	14	1.5×10^{-3}	0.22	130			
Soft clay	40~60	17~19	> ω	12~19	$(3\sim6)\times10^{-4}$	0.3~0.7	<100	0	30	0.7~1.2
Sandy clay	23~31	20	> ω	10~15	$(1\sim9)\times10^{-3}$	0.15~0.2	200	5~10	31	0.35
Sand,Silt	30	19				0.11	340			

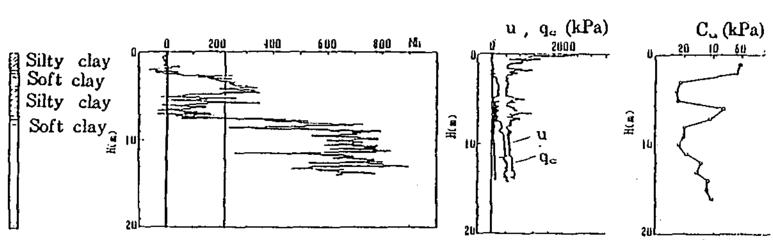

Fig.3 The Results of Boring Samples, CPTU Tests and Vane Tests.

of soft soils.Usually, the coefficient of consolidation were obtained from laboratory tests using boring samples. The C_h value may be made several orders of magnitude underestimation in field values. Another method is that the C_h value can be deduced from the pore pressure dissipation curve measured in-situ under loading. The results measured in-situ might be reliable than the laboratory test.However it could not provide the design parameters for the project.

The field C_h value can be obtained from CPTU dissipation test for design. So, it is very important to be clear about the interrelation between the CPTU dissipation tests, laboratory tests and field measured results.

4.2 Evaluation of C_h

The basic principle of a dissipation test is that after the arrest of steady penetration, excess pore pressure generated during cone penetration immediately starts to dissipate.By monitoring the rate of dissipation, the coefficient of consolidation may be obtained.To compare CPTU dissipation test with the theoretical dissipation curves, the excess pore presssure records have been normalized by calculating the degree of excess pore pressure $\bar{u}$, as:

$$\bar{u} = (u_t - u_0)/(u_i - u_0) \qquad (5)$$

where, u_t, u_i, and u_0 are the current, initial and static pore pressure respectively.

The measured and normalized curves can be over laid and matched as closely as possible with the theoretical curve, such as Randolph and Wroth curve. Generally speaking, the beginning of the measured data does not match very good and the end of the theoretical curves is beneath the measured curve.Therefore the sections of u=0.4-0.8 must be mainly considered. For a given decay of excess pore pressure, the real time t and the corresponding time factor T can be determined. The coefficient of consolidation can be calculated at any disssipation level using the measured time, t, and the corresponding time factor, T, as following:

$$Ch = R^2 T/t\,(200/I_r)^{1/2} \qquad (6)$$

in which, R---the radius of the cone,

T--time factor

t--measured time to reach this degree of consolidation

200--reference rigidity index

I_r---rigidity index value of the soil

4.3. Case histories

Five sites were selected to illustrate the application of CPTU dissipation test.

Case 1

The soil condition has been discribed in 3.2. The cofficient of consolidation were determined by three mehods; CPTU dissipation tests, laboratory tests on undisurbed samples and deduced from the pore pessure dissipation curve under loading (measured in field).

The results are summarized in Table 3.

From this table we can find that the values measured in field are about ten times higher than the values of laboratory tests,the C_h values of CPTU dissipation test are very close, and its values are 2 ~ 3 times higher than the values of measured in field.

Case 2

This site is located in Nanjing. The original ground conditions at this site are summarized as follows; the artifical soil ranges from 0 to 5 meters in depth, 5 ~ 10 meters is silty clay stratum, underlying stratum is sandy silt.

Table.3.The Ch values for three methods in soft clay ($\cdot 10^{-2} cm^2/sec$)

①	SECTION	1	2	7	8	9	10
②	CPTU tests	1.57	2.90	3.20	3.30	3.30	3.00
③	Lab tests	0.028	0.032	0.33	0.20	0.23	0.22
④	Measured in field	0.95	0.40	1.12	1.18	1.60	1.30
⑤	②/④	1.65	7.25	2.86	1.75	2.10	2.30

The results of laboratory tests on undisturbed samples show that the C_h value of silty clay is 3×10^{-4} cm^2/sec. The Ch value of CPTU dissipation test is $2.83 \times 10^{-2} cm^2/sec$, this value is higher than the laboratory value by two orders of magnitude.

Case 3

It is located nearby the bank of Yangtse River. In this site the main layer from the level of 2 meters to 25 meters is silty clay. The water content is higher than 50%, void ratio is 1.1 ~ 1.5, it belongs to high compressibility clay.

The C_v value of laboratory tests varied from $1 \times 10^{-3} cm^2/sec$ to $7 \times 10^{-3} cm^2/sec$, and the C_h values of CPTU tests are very close, the average value is $1.25 \times 10^{-2} cm^2/sec$, The difference of these two kinds methods is one order of magnitude.

Case 4

This site is located in Wuhan. The main layer ranged from 3 meters to 11 meters in depth is sandy clay, the void ratio equal 1.5, and water content is higher than 50%.

Case 5

It is located in Guonzhou harbor. The main layer ranged from surface to 9 meters in depth is silty clay, the void ratio is 2.5 ~ 3.0, and the water content is much higher than the liquid limit, near 100%.

In cases 4 and 5, three methods were performed; CPTU dissipation tests, laboratory tests and measured in field from settelement ~ time curve. The results are summarized in Table 4.

Table.4.The C_h values for cases 4 and 5 ($\times 10^{-2} cm^2/sec$)

CASE	Case 4	Case 5
CPTU tests	0.451	0.346
Lab tests	/	0.047
Measured in field	1.200	1.970

It must point out, in these two cases, the Japanese equipment were used, and the filter element is located on the cone. In table 4, we can find that the C_h values of CPTU tests is lower than the laboratory tests by one order in magnitude, and the C_h values of CPTU test are 2 ~ 5 times lower than the measured in the field.

5. THE METHOD OF SETTLEMENT ANLYSIS USING CPTU TESTS

5.1. Basic concept

The soil deformation properties can be expressed by tangent modulus, it is defined as:

$$M = dP'/d\varepsilon \tag{7}$$

where dP'--the increment of effective stress

$d\varepsilon$ --the increment of strain

The modulus reflects the ability of deformation resistance of the soil.

CPTU tests measures the total cone resistance q_c, and the total overburden pressure γh is known for a given depth, so, the net cone resistance $q_n(=q_c-\gamma h)$ can be decided.

From the Norwegian experiences the relationship exists between q_n and M as follow:

$$M = m \times q_n \tag{8}$$

where m is a experimental factor.

Based on the concept of M, for an initial soil thickness of dh, stress increases from insitu stress P_0' to $P_0' + \Delta P$, (in which ΔP is the applied stress) and the normal strain varies $d\varepsilon$, so the settlement would be:

$$ds = d\varepsilon \times dh = (dP' \times dh)/M \tag{9}$$

The amount of settlement S would be:

$$S = \frac{1}{m}\sum_{i=1}^{n}\frac{\Delta P_i\,\Delta h_i}{(q_c-\gamma h)_i} \tag{10}$$

in which, ΔP_i--stress increment for each layer, KPa

Δh_i--thickness for each layer, m

$(q_c-\gamma h)_i$--net cone resistance for each layer, kPa

We can evaluate the experimental factor m for given example, if the values ΔP_i、Δh_i、$(q_c-\gamma h)_i$ are given and the settlement are measured. Then we use the m value to calculate the settlement of another site. For checking the reliability of this method, we compare it with measured value.

Table.5 The values of settlement calculation

	①	②	③	④
Section	m value	Calculated S_∞, cm	Measured S_∞,cm	Error (③-②)/②, %
2	6.0	27.5	23.4	17.5
3	6.75	112.4	120.0	6.3

5.2. Examples

Four examples were selected in Case 1,in which sections 1 and 2 belong to lower stress level conditions ($P_0'+\Delta P<P_c$), sections 3 and 4 belong to higher stress level conditions ($P_0'+\Delta P>P_c$). Pc value is apparent preconsolidation pressure.

As mentioned above,we can calculate the m values for sections 1 and 4 examples using q_n, Δh_i, ΔP_i and measureed settlement S_∞, the m values are 6.0 and 6.75 respectively.

We can use m value to calculate the settlement of sections 2 and 3 , the results are shown in Table 5.

From the results we can find that the difference between calculated and measured settlement are not too large, within 18% only.It is good enough for settlement calculation.

6. THE POSSIBILITY FOR INTERPRETING STRESS HISTORY

In Case1, the soil compression curves behave a yield point.The yield stress is named apparent preconsolidation pressure P_c, the ratio of P_c to effective overburden pressure P_0', P_c/P_0' ,is named apparent overconsolidation ratio,OCR. A good relationship has been established between OCR and depth, as shown in Fig. 4. It can be seen that OCR=6.8 in the surface crust, and varied from 3.0 at 2 meters to 1.0 at 9 meters.

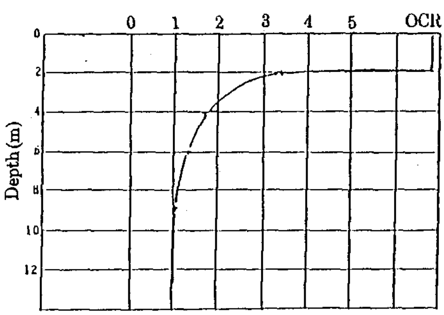

Fig.4 The Distribution Curve of OCR in Depth.

The ratio of excess pore pressure Δu to corrected cone resistance qt, can be obtained from CPTU test. We plot the $\Delta u/q_t$ profile, as shown in Fig.5.

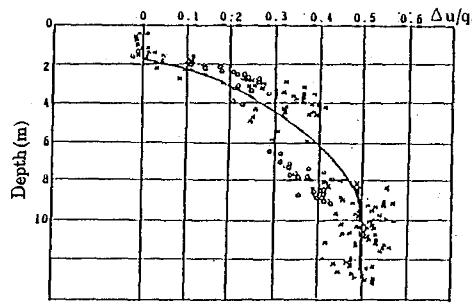

Fig.5 The Distribution Curve of $\Delta u/q_t$ in Depth.

Although some scatter exists in this figure, but the tende-ncy of variation with depth is obviously. Within the range of 2 meters $\Delta u/q_t=0$, then increase with depth until 9 meters, a constant value 0.5 is obtained in the deeper zone.

Comparing these two figures we can find that the boundary of the depth are very similar. The author suggest that the method may be a useful alternative to other current techniques for predicting stress history.

7. CONCLUSIONS

1. The CPTU test is becoming increasingly more popular for site investigation and geotechnical design in China.

2. Zhang and Greeuw classification chart have been examined and can be used in China.N_h profile of CPTU tests are corresponding to soil profile of boring samples and vane strength profiles.

3.The values of coefficient of consolidation C_h determined from CPTU dissipation tests are very closely in the same layer, and the C_h value from CPTU is higher than laboratory tests by 1 ~ 2 orders in magnitude. Comparing the value of CPTU tests and field measured under loading, one can find that the values of CPTU dissipation tests are 2 ~ 3 times higher than the values of field measured. But in another case, a result has been obtained the Ch values of CPTU tests are 2 ~ 5 times lower than the measured in the field.

4. The method of settlement analysis using the net cone resistance is good enough.

5.The $\Delta u/qt$ profile can be used to judge the stress history, this method may be a useful alternative to other current techniques for predicting stress history.

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CPT in Denmark - National Report

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SYNOPSIS: Although the soils in Denmark may not seem ideal for penetration testing, CPT has been applied with a remarkable increase within the last decade. Consequently, the modern, mobile equipment is now applied onshore. Offshore testing is performed with a fully computerized underwater rig.

The daily use of CPT is backed up with substantial national experience from consultancy and supplemented with results from research projects with objectives dedicated to CPT.

1. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION

Denmark proper is lowland area, on average no more than 30 m above sea level.

Although the whole of Denmark is relatively low-lying, a remarkable scenic boundary can be traced from Nissum Fjord on the west coast of Jutland eastward toward Viborg, thence swinging sharply south down the spine of the peninsula toward Aabenraa and the German city Flensburg. This boundary represents the extreme limit reached by the Scandinavian and Baltic ice sheets during the most recent glaciation which terminated about 10,000 years ago.

The ice front is clearly marked in the contrast between the flat west Jutland region composed of sands and gravels strewn by the meltwater that poured west from the shrinking ice sheet, and the fertile loam plains and hills of eastern and northern Denmark which become markedly sandier toward the ice front. In the northwest, around the Limfjorden area, there are numerous landscapes of flat sand-and-gravel tracts created by marine deposits.

On the island of Bornholm the contrasting solid rock reveals close affinities with that of southern Sweden. Exposure of Precambrian granites cover extensive areas of the northern half of the island which is overlain to the south by Cambrian sandstone and shales.

A typical soil profile from Denmark exclusive of Bornholm consists of a top soil of different thickness and soil types underlain by clay till and limestone.

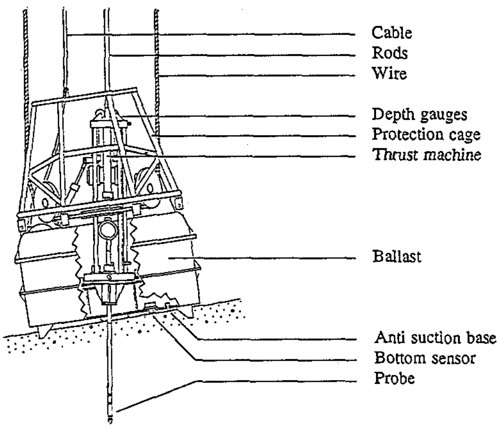
2. PENETRATION TESTING AND OTHER INVESTIGATION METHODS IN DENMARK

The most frequently used investigation method in Denmark is to collect soil samples from boreholes. The reason for this is

- The Danish soils exhibit significant and often unexpected variations (in type and parameters) within short distances both in horizontal and vertical directions. These variations are difficult to identify by methods where no soil samples are produced.
- The soil in Denmark is generally stiff and often contains gravel and boulders which means that it is difficult to penetrate and the equipment is often damaged and probes are lost.
- For smaller projects only one rig type can be applied for economical reasons. As results from penetration tests should be compared and calibrated with unequivocal determinations of soil stratigraphy and properties of specific soil layers a drilling rig is considered optimal.

However, penetration testing in general and CPT in particular are used increasingly in Denmark. Particularly for larger projects or in situations where a specific soil signature can be clearly identified. CPT is here an important alternative accompanying method as a large

Figure 1: The Danish offshore CPT-rig SCOPE



area can be investigated in a relatively short time.

A field with increased use of penetration testing is offshore investigations of the sea bottom of the Danish inland waters where in particular CPT can be performed conveniently by a remotely controlled rig placed directly at the sea floor. CPT has here proved to be a valuable alternative to the more costly conventional drilling procedure performed from a jack-up platform.

Another important field is environmental sounding where the extent on a possible contamination in many cases can be determined much more precisely and efficiently by CPT than by drilling.

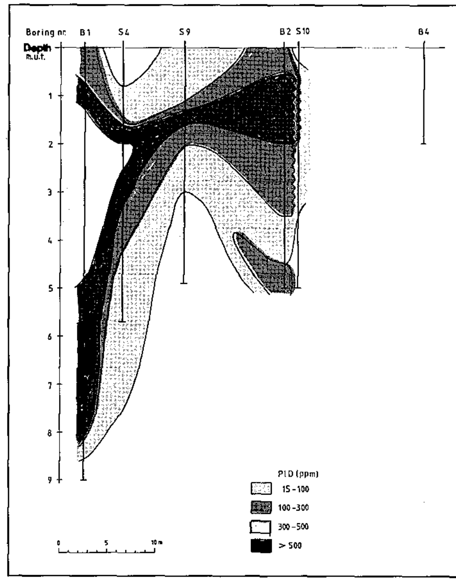
3. CPT EQUIPMENT USED IN DENMARK

3.1 Equipment

Onshore CPT investigations are mostly performed by special, ballasted trucks containing the thrust machine and the necessary data acquisition equipment. This set up is highly mobile and almost all operations in connection with the test can be made from the inside of the truck.

Report-ready enclosures are produced immediately after the test, often in a high laser printer quality.

Figure 2: Cross Section showing VOC measurements by PID (ppm)



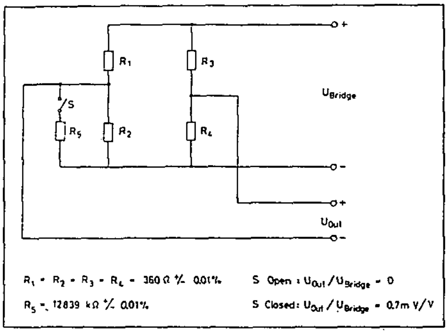
Several trucks made by this concept owned by different companies cover the current Danish market for CPT investigations.

The test procedures are as prescribed by ISSMFE (cf.: Swedish Geotechnical Society, 1989). No Danish code or standard deal in detail with CPT.

For offshore applications the Danish Geotechnical Institute has in 1992 developed a fully computerized rig to perform all kinds of CPTs directly from the sea floor. The rig, SCOPE (Seabed COne PEnetremeter), consists of a cylindrical bottom chamber (Figure 1) containing ballast. The ballast can be adjusted for the specific job to obtain the optimum between easy handling and sufficient counterweight. A hydraulic push-puller press with a capacity of 200 kN ensures a completely smooth penetration by a constant velocity. Furthermore, SCOPE is equipped with an erector system to secure vertical penetration even when the seabed is inclined up to 10° with the horizontal. A detailed description of the rig is offered by Denver and Riis (1992).

SCOPE has proved its versatility on several

Figure 3: Shunt Calibration Circuit



jobs and has now been modified to operate on water depths up to 400 m.

During the last 4 years the Danish consulting company Kampsax Geodan has developed CPT tools and procedures for environmental investigations.

These tools make it possible to carry out continuous soil gas sampling/analyses with a CPT-rig. The concept is to pump soil gas from the formations while penetrating to the analysing instruments at the rig. Here the presence and concentrations of volatile components as volatile organic carbons (VOC) are determined by a PID (photo ionisation detector), methane, CO_2, etc.

Continuously, CO_2, O_2, CH, and VOC contents are measured together with flow, temperature and vacuum. By indication of VOC, soil gas samples are furthermore analyzed by a portable gas chromatograph to determine and quantify the VOC.

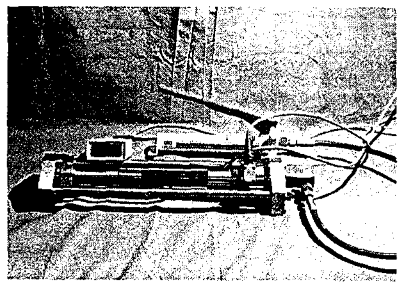
Besides soil gas sampling, equipment has been developed for soil and ground water sampling at specific depths.

A special stainless steel tube device ensures that high quality soil samples may be collected without risking to loose volatile compounds or cross contamination.

If necessary the knowledge of the formations may be supplemented by carrying out continuous electrical and gamma logs while penetrating.

Thus a highly detailed 3D knowledge of

Figure 4: Field check facility for CPT cone and sleeve (hydraulic jack, load cell and tip mounted in a stiff frame)



contamination and formations may be obtained in a rational and cost effective way by combining the above mentioned tools (cf. Figure 2, where VOC contents measured by PID are shown).

The methods are well suited for investigations at old dump sites (tracing methane and other gases), industrial sites (pollution with VOC etc.), filling stations (fuel and oil) etc.

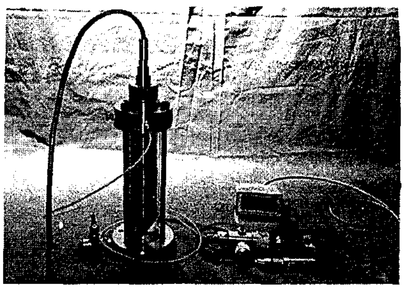
The equipment has been used on hundreds of polluted sites being more time and cost effective and giving a much higher quality of investigations for volatile compounds than ordinary borehole investigations.

3.2 Calibration

The concept for calibration followed by the Danish Geotechnical Institute has previously been reported in detail (Denver and Wille, 1991). A brief description is offered in the following:

- All sensors are calibrated in the laboratory against a set of house normals (denoted GI-Normals). These are themselves calibrated to national or international standards to ensure and maintain the expected traceable accuracy. The integrity of this system is provided by different means described further by Denver and Wille (1991).
- The proper functioning of the electric system except the probe (i.e. inclusive of the cable) is controlled in the field by a shunt calibra

Figure 5: Field check facility for pore pressure measurements (hand pump reference transducer, indicator connected to the piezocone mounted in a cell



tion facility incorporated in the amplifier. The check is performed by connecting the probe end of the cable to the amplifier cabinet. The unbalance caused by the shunt circuit (shown in Figure 3) is logged and displayed together with the corresponding value obtained during calibration of the probe. A minor deviation can be tolerated as the results will be adjusted. Larger deviations mean malfunction of the circuit, and are not accepted, and the entire equipment will be overhauled in the workshop. Zero values are registered and logged in the field in connection with the test.

- A condition check of the probe can be performed in the field as the sensors can be excited in a controlled way by small check units (Figures 4-5).

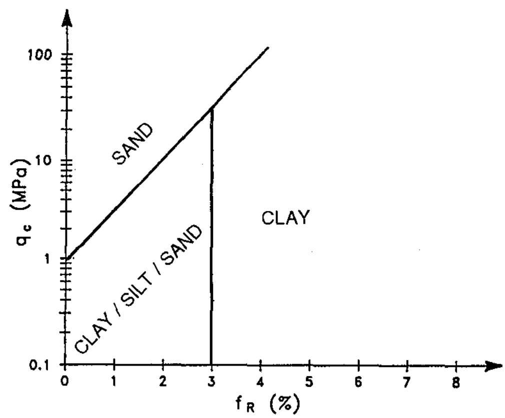
The philosophy is that a traceable calibration is maintained in the field by (i) thorough calibration routines performed in the laboratory against GI-Normals, (ii) integrity of electrical circuits and amplifiers controlled by shunt calibration facilities, and (iii) check procedures made in the field to verify the probe.

4. INTERPRETATION OF TEST RESULTS

4.1 Classification Charts

The Danish classification charts have mainly been based on the point resistance and the

Figure 6: Danish soil type diagram prepared by Jørgensen & Denver (1992)



sleeve friction. It is experienced in Denmark as in most countries that a classification chart should only be regarded as a rough guideline in the determination of the soil types. Locally large deviations might be registered. This fact has inspired Jørgensen and Denver (1992) to propose a simple chart where only the main soil types, namely sand and clay are considered (Figure 6). Furthermore, the authors have demonstrated a high degree of accuracy to determine these soil types. But it should be added that

- the data used to verify the chart are those which have been used to design the chart, and
- a "grey zone" where the prediction ends up with an indeterminate mixture of clay/silt/sand, has been included.

The basic idea behind this simple chart was that it could be used without a previous calibration on a new site.

Later, Luke (1994) has proposed a more detailed classification chart (Figure 7) based on a substantial amount of additional data. This chart offers much more specified information on the different soil types.

However, a comparison between the two charts reveals no significant contradiction.

Figure 7: Danish soil type diagram prepared by Luke (1994)

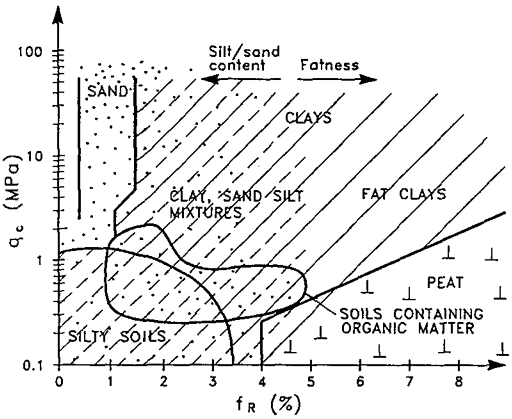
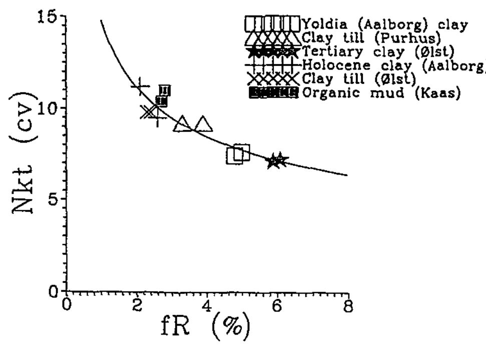


Figure 8: N_{kt} plotted against f_R showing the best fitting line (from Luke, 1994)



4.2 Undrained Shear Strength

In Denmark it is costumary to use a cone factor

$$N_{kl} = \frac{q_c - \sigma_{vo}}{c_{vane}} = 10$$

where c_{vane} is the undrained shear strength measured by the field vane and σ_{vo} is the overburden pressure (Denver, 1988; Kammer Mortensen et al., 1991; Jørgensen and Denver, 1992). Results from different Danish clay types are in agreement with this value of the cone factor.

Special attention should be paid to the investigation by Kammer Mortensen et al. where hundreds of corresponding values of q_c and c_{vane} sustain the above-mentioned finding for the glacial Storebælt clay till. The spatial variation of the parameters has here been taken into account by application of interpolation routines. However, a large scatter was observed (cf. Figure 9), and it was found that the correction for the overburden pressure did not reduce this scatter significantly. The cone factor $N_{kl} = 10$ actually corresponds to a modified formula where σ_{vo} is neglected.

Later, Luke (1994) proposed the above-mentioned value to express the undrained shear strength measured by n triaxial tests whereas

$$N_{kl} = \frac{q_t - \sigma_{vo}}{c_{vane}} = f_R^{-0.4} \cdot 15$$

for the field vane where f_R is the friction ratio (cf. Figure 8). More test results that sustain this expression are shown in the reference.

4.3 Modulus of elasticity

The modulus of elasticity for sand has occasionally been estimated by CPT by

$$E = 7\sqrt{\frac{q_c}{q_o}}$$

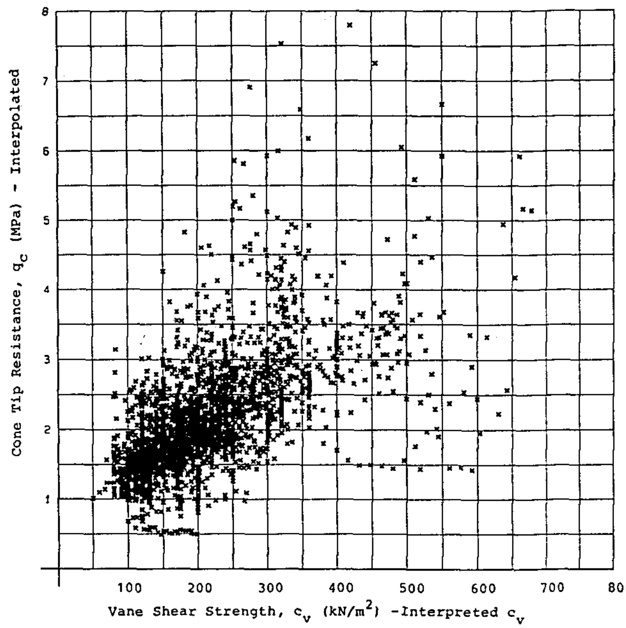
where q_o = 1 MPa. The modulus predicted corresponds to the one obtained from a pressuremeter test (E_p) or from the initial slope of the performance curve from a screw-plate test.

This value can be used in connection with a simple settlement calculation procedure proposed by Denver (1985).

5. USE OF CPT IN GEOTECHNICAL DESIGN

A complete soil model in probabilistic terms where the uncertainty is quantified has occasionally been established for large offshore foundations where a considerable number of test results from borings and sounding are available. Ditlevsen and Gluver (1991), Ditlevsen (1991) and Andersen et al. (1991) have derived methods to calculate the spatial

Figure 9: Tip resistance versus field vane strength (from Kammer Mortensen et al., 1991)



variation of undrained shear strength for clay till by means of results from CPT and shear vane tests for anchor block design for the suspension bridge across the eastern channel of Storebælt, Denmark.

For the same bridge the reliability of the pylon foundation has been analyzed by Madsen (1990) where the soil properties have been derived from CPT.

6. RESEARCH ACTIVITIES

A Ph.D. thesis made as a joint project between Akademiet for de Tekniske Videnskaber, Kampsax Geodan, and Aalborg University, has just been concluded (Luke, 1995). Many of the findings have been mentioned elsewhere in this report. The results are based on extensive in situ testing on six test sites supplemented with advanced laboratory tests on recovered samples. Furthermore, a test serie has been performed with CPT in a calibration chamber.

The Danish Geotechnical Institute joins presently an EU-funded project in the programme package BRITE-EURAM II together with British Research Establishment, Norwegian Geotechnical Institute, and Laboratoire Central des Ponts et Chaussée (France). The objective of the project is to produce semi-empirical foundation design procedures based on the results of three new in situ testing devices.

One of the sounding methods is a triple element piezo-cone where three pore-pressure sensors are situated in different heights on the probe. The concept of using empirical design algorithms (and not basic geotechnical parameters) will be verified by a testing programme. The soil is actually tested by different probes on a score of special selected sites where previous observations of foundation behaviour for structures (constructed on the sites) are available.

7. NEW DEVELOPMENT

A combination of Cone Penetration Testing (CPT) and Vibrocore sampling is achieved with the new made offshore Combi-rig designed by the Danish Geotechnical Institute.

The advantages of the system are found in significant time savings (specially in deep waters) and higher quality of data.

The necessity to carry out a vibrocoring can now be based on the results of the CPT data evaluation on board. Standard site investigations (without CPT) often involves coring at pre-defined intervals, usually resulting in great numbers of equal samples.

The Combi-rig can stay deployed between CPTs, and will only be required on board after coring activities. Coring could directly follow series of CPTs which obviously drastically speeds up the field work.

Correlation of the soil characteristics derived from CPT data and core samples will be ideal as they will always relate to exactly the same site. This in contrast to standard, non-combined CPT and coring, where the comparison depends on the accuracy of the underwater positioning (if any at all).

The Combi-rig can be deployed in water up to 1,000 m of depths. A modular design enables adjustment of rig height and ballast (5–20 tons) to enable optimal handling with respect to project requirements.

8. ACKNOWLEDGEMENTS

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National Report, Finland

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SYNOPSIS: Due to the geological conditions in Finland the CPT test has not yet gained a dominating role as a tool for subsoil investigations. The surficial deposits consist of soft, glacial and postglacial clays, glacial till covering the bedrock surface and some fluvial deposits in-between. For the clay deposits the CPT-test signal is not sensitive enough. The presence of stones in coarse grained soil layers forms a major problem to sounding performance. Improving the resolution of the signals and modifications towards a static-dynamic procedure are future topics for development work

1. BRIEF GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF THE REGION

The surficial deposits in Finland were mainly formed during the last glaciation or thereafter as a result of various geological processes. The material either derives from the bedrock (mineral soils such as till, gravel, sand and clay) and plant remains (organic deposits such as peat), or is made up of precipitates of compounds dissolved in water (lime gyttja). Moraine and till deposits cover over 50% of the total surficial deposits in Finland. 15% are formed of peat deposits, 13% of rocky terrain and 10 % of clay and fluvial deposits. The distribution of Quaternary deposits is uneven, depending on land forms, the elevation of an area above sea level, regional and and local characteristics of the activity of the continental ice sheet and its meltwaters, the rate of uplift and climatic factors.

Basal till is the predominant sediment type in Finland, and it often acts as a substratum for other sediments. Basal till covers the bedrock, being thickest in depressions. The average thickness of the sedimentary deposits in southern and western Finland is on the order of magnitude of about 10 meters. The thickness of he predominantly silty deposits met with in the interior of the country varies generally from about five to ten meters. Even in valleys, the thickness of a sedimentary deposit can vary considerably: a ridge of bedrock or a moraine can rise to the surface in the middle of a valley.

Because the densely populated areas are situated mostly in the low-lying coastal areas of southern and south-western Finland, the marine and lacustrine deposits, silt to clay, have great importance from the engineering point of view. In central Finland, which was submerged for a relative short time, the deposits are small and shallow. In southern and south-western Finland, as well as in Ostrobothnia, the deposits are coherent, extensive and up to 60 m thick.

Finland's clays are mainly either glacial or post-glacial. The clays are largely composed of the same minerals as the bedrock, that is feldspars, micas and quartz, but they also contain clay minerals produced from the bedrock minerals by chemical weathering. The clay mineral compositions of the glacial and post-glacial clays do not vary much from one region to another. However, the properties of the clays may differ substantially, as the clays are very different from each other in grain distribution, sulphur and carbon contents and pore water composition. In accordance with the evolutionary stages of the Baltic Sea, fine-grained sediments can be divided into the following four types, enlisted from the youngest to the oldest: Littorina, Ancylus, Yoldia and Baltic Ice Lake sediments. The two former types of sediments are nearly homogeneous in structure. Yoldia sediment is homogeneous or it is characterised by symmything varving. Sediments from The Baltic Ice Lake phase are distinctly varved and anisotropic in their properties.

Fine-grained deposits are normally consolidated for the most part. Overconsolidation occurs, however, in many places beneath the dry crust, the OCR being on the order of 2...4. The undrained shear strength beneath the dry crust is apt to vary inside the range of about 2...60, being typically between 10 and 20 kN/m^2 for NC clays. The shear strength of fine-grained sediments is dependent mainly on the humus content, clay content and water content. The humus content is highest in Littorina sediments, the mean being about 4 %. The modulus of compressibility varies beneath the dry crust in humus- and clay-rich NC Littorina, Ancylus and Yoldia sediments mostly from 10 to 40 kPa. The modulus values for silty soils may be several tens of kPa's.

The geologic stratigraphy is typically problematic as far as cone penetration testing is concerned. Under the fine-grained and soft sediments there is usually a very stiff layer of moraine. The layer between moraine and clay can be thin, consisting of silt, sand and gravel, or the layer may be absent. Very often the layers beneath the clay and silt layers contain stones and boulders, and the CPT point can easily be damaged there. Because the Finnish surficial deposits so often are tills or moraines, the use of CPT is restricted to peat, clay, silt and sand areas, which represent less than 30% of the total area of Finland.

2. TYPE OF PENETRATION TESTING AND OTHER INVESTIGATION METHODS USED IN FINLAND

The most commonly used penetration testing method is the Swedish weight sounding test used both in fine-grained and coarse-grained soils. The method is used for determining soil layers, as well as for obtaining preliminary data on the soil strength properties and, e.g., pile lengths.

The method used in coarse-grained soil is dynamic probing, applied mostly for determining pile lengths. The method and equipment used here are slightly different from those used in continental Europe.

In the 1990's, the Swedish weight sounding test method has been increasingly substituted by the so-called static-dynamic penetration test. The method is used both in fine-grained and coarse-grained soils for determining soil layers and soil strength properties. The static-dynamic penetration test applies a mechanical CPT method. The probe point is the detachable point of the dynamic probe (D = 45mm). In this method the rods (D = 32mm) are compressed with a speed which remains constant (1.2 m/min), probing machine capacity allowing, while the rods at the same time continuously rotated. When it is no longer possible to probe further by compressing, the same rods and point are used for dynamic probing, rotating the rods constantly. The method incorporates automatic test result recording.

Along with the static-dynamic penetration test, the so-called SIPT, or static impact penetration test, is used. The strikes of the continuously rotating dynamic probe are recorded on the basis of stress-wave measurements. The results make it possible to determine the so-called point resistance, which again can be used to determine strength properties of coarse-grained soil in layers where the CPT method cannot be used.

The fine-grained soil shear strength and sensitivity are determined by using the vane test. Various vane sizes and equipment are used which make the method applicable also to the investigation of stabilized soils.

By percussion drilling, rock surface and quality as well as filling earth layers can be studied. Owing to the automatic registration of test results (MWD), the percussion drilling method will be increasingly used in soil and bedrock testing. Vibrodrilling is still sometimes used, e.g. in the preliminary determination of bedrock surface.

The core samples obtained by core drilling are used to determine bedrock quality and strength properties.

Various kinds of tip samplers are used for taking both disturbed and undisturbed samples from cohesion soils as well as disturbed samples from friction soils.

In the past few years, geophysical research methods have increasingly been used also in geotechnical field investigations. The methods used include the ground penetration radar, the seismic surveys as well as soil resistivity surveys.

The pressuremeter and the dilatometer are also available in Finland, even if these methods are not yet in extensive use.

Groundwater surface level is generally determined by using the groundwater standpipe. Pore water pressure is determined with the help of electric piezometers.

Other measurement techniques are also used.

3. TYPE OF CPT EQUIPMENT USED IN FINLAND

Equipment

Currently, there are about ten different equipments in use in Finland. Most often, cableless points are used, the probing data being recorded simultaneously in the memory unit situated in the point itself.

As the CPT is principally used for determining cohesion soil properties, the most often used points are those measuring point resistance, sleeve friction and pore water pressure.

Light or medium-weight drilling machines are used. Depending on the machine, the maximum compressive force varies between 20 and 60 kN. However, due to the cobbles in the soil, maximum power can often not be used in order to prevent the point from breaking.

Test procedure

As regards the equipment and its condition, there are no generally accepted quality criteria established so far. In fact, the suppliers of the equipment are mainly responsible for, e.g., point calibration.

Correction and presentation of test results

Automatic registration equipment is used in field work for recording the results which are then transferred to the computer. Some probing data processing programmes are in use for controlling the correctness of the data.

The probing data is corrected as concerns, e.g., the errors of measurement originating from the rods being changed and the drilling being terminated. However, no pore water pressure corrections are made to point resistance values. The measurement level difference between point resistance and sleeve friction values is usually not corrected in result processing.

Depending on the software used, some soil parameters can be determined but sophisticated CPT interpretation programmes are not yet generally used.

National codes and/or standards

No Finnish instructions or norms have been published. In probing and result interpretation, international recommendations, and especially the Swedish Geotechnical Institute (SGI) instructions are followed.

4. INTERPRETATION OF TEST RESULTS

Soil classification and stratigraphy

Soil layers are mostly determined by the geotechnical engineer on the basis of his professional experience and visual interpretation of sounding resistances. Some organisations have their own in-house interpretation programmes based on directly measured resistance values which have not been corrected with the pore water pressure values.

Soil parameters and other data

The most commonly interpreted parameter is the undrained shear strength. The interpretation is usually made by determining the local conversion coefficient between the point resistance and unreduced shear strength determined with the vane test. The CPT's points used in Finland have proved to be too unprecise for the interpretation of other soft soil layer properties, as concerns, in particular, the recording accuracy of the data measured. In isolated cases, the deformation properties in coarse-grained soils have been determined on the basis of point resistance (α-method). Correspondingly, soil layer friction angle has occasionally been determined on the basis of the static penetration test. The method has also been used for controlling compactness.

Environmental data

Not used in Finland.

5. USE OF CPT IN GEOTECHNICAL DESIGN (DIRECT AND INDIRECT METHODS)

Direct CPT calculation methods have not been used in Finland, except in some isolated cases. The point bearing pile commonly used in Finland penetrates the dense moraine or bedrock. The CPT applied to the relatively light equipment used in Finland cannot penetrate these bearing layers.

6. COMPARISON AND CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

No systematic comparison has been made in Finland. The most extensive empirical material has been collected on the correlation of shear resistance determined by the vane test and the CPT point resistance (q_T - σ'_{vo}). The correlation has been found to be quite insignificant in zones where shear resistance is inferior to 20 kN/m^2. It has been suggested that such low correlation values could be explained by the fact that the probing equipment used is inapplicable for the measurement of small point resistance values. The measuring range of almost all CPT drills stops at q_c = 100 MPa.

7. MAJOR AREAS FOR RESEARCH ACTIVITIES

Due to the limited use and applicability of the method, very few studies on CPT have been made in Finland. Some isolated studies have, however, been made on whether the interpretation methods published in international literature can be applied to Finnish conditions (Rusanen, 1994). The Technical Research Centre of Finland has collected a data base of reference material. Currently, the material includes about 100 CPT measurements with relevant reference measurements made by various organisations all over Finland.

8. FUTURE TRENDS AND NEW DEVELOPMENTS

There are two kinds of development needs in Finland. On one hand, there is a clearly emerging need to implement much more sensitive CPTs with a higher measurement resolution, applicable to property measurements of clays with the shear resistance of c_u = 5...30 kPa. On the other hand, practical needs orientate the development towards a method where the measurement can, after compression, be continued by strikes - i.e., towards the static-dynamic penetration test. The research and development work on the latter method is continued by some Finnish organisations.

9. REFERENCES

Rusanen, J., Puristin- huokospainekairauksen tulkinta (Interpretation of Cone Penetration Testing). *M.Sc. master thesis. Helsinki Univ. of Tech., Soil Mechanics and Foundation Engineering*. Helsinki 1994. (In Finnish, Abstract in English). pp. 106 + app. 13.

Application of cone penetration test (CPT) in Germany

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SYNOPSIS: Cone penetration tests are used in Germany particularly in soils and weathered rock to identify and to determine certain mechanical soil characteristics, such as bedding density, angle of internal friction and the stiffness coefficient, which either cannot be determined in the laboratory or can only be determined in small numbers. Soils of this nature cover an area of about 30 % of Germany, which explains why this test method is frequently used in our country. DIN 4094 describes both the test equipment and the procedure for the determination of cone resistance, local friction resistance and friction ratio. Further parameters, such as pore-water pressure, temperature, electric conductivity and inclination are carried out by specialists but there are no special regulations on this point. On some types of soil, the cone penetration device can replace the vane shear test, enabling the empirical determination of undrained shear strength and the identification of the soil as based on the friction coefficient. Driveability and bearing capacity of piles can be estimated. Instead of using the 10 cm^2 cone, as specified in DIN 4094 or TC 16 ISSMFE, tests are also performed with 15 cm^2 cones.

1. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF THE REGION

In the northern part of Germany the holocene subsoil consists of marine and fluvial sediments with different organic material, often of low density or low consistency. Undisturbed soil samples are difficult to obtain, laboratory tests often give too unfavorable values. The thickness of these sediments is generell about four meters, but can increase to fifteen meters or more. Important is to find the boundary surface of the pleistocene layers which regionally can have been preloaded by ice and have then better conditions of compactness , consistency and undrained shear strength between 5 and 25 kPa.

In the southern region of Germany some fluvial and limnal sediments are problem subsoils with long term settlement behaviour.

The middle regions of Germany consist of paleozoic and mesozoic rocks which often are weathered to a depth of about ten meters or more. Sometimes they are covered by eolian sand or loess. A lot of tectonic events destroyed the former structure of the rocks. Rivers flowing in these old rocks are accompanied by pleistocene and younger terrace sand and gravel which provide a good subsoil, but can locally contain soft organic soils. Igneous and metamorphic rocks in the paleozoic mountains and some neozoic regions are not problematic for foundation except the volcanic tuff or tuffit which are sensitive to any load.

2. TYPE OF PENETRATION TESTING AND OTHER INVESTIGATION METHODS USED IN GERMANY

For site investigation in Germany besides borings and the cone penetration test CPT, dynamic probing (DP) is often used in non-cohesiv soils as DPL and DPH. In special cases e. g. some gravels with flat grain shape or weathered rock, DPM and DPSH are in use. Equipment and test procedure correspond to the report of

ISSMFE TC 16 appendix C. The interpretation of the test results are described for some sands and gravels in DIN 4094. The standard penetration test may be used with a sampler as described in TC 16 appendix B, but in Germany the test is made to ascertain the compactness of non-cohesive soils and then used with a cone of 20 cm^2. The hammer assembly, sounding rod and cone connected to cable are lowered as a unit to the bottom of the bore hole. This test procedure is not in accordance with TC 16 but described in DIN 4094.

The weight sounding test (see TC 16) is not in general use in Germany. In cohesive soils the vane shear test is often used to determin the undrained shear strength up to 100 kPa of lacustrine clay, tidal mud deposits or any silty and peaty muds. The most interesting values are between 5 and 25 kPa. The equipment and test procedure are described in DIN 4096.

The pressiometer is sometimes used in Germany in connection with the compaction of deposits especially vaste deposits, by dropping heavy weights.

In rocks, dilatometers are used whose equipment and test procedure are described in „technical test procedure for soils and rock, DGEG recommendation No 8" (Technische Prüfvorschrift für Boden und Fels DGEG Empfehlung Nr. 8). The deformation under different loads is measured and a modulus is calculated, which gives an indication on the expected settlement and loading capacity.

3. TYPE OF CPT - EQUIPMENT USED IN GERMANY

3.1 Equipment

By means of the CPT equipment a probe is pressed into the subsoil at a constant rate, during which the resistance of the tip, the local side friction and sometimes the pore water pressure are measured. The mechanical CP with double linkage may be used today, but electrical measuring elements are now the normal equipment. The manufacturing tolerances of the new tip diameter are 35.7 ± 0.3 mm (DIN 4094). The minimum worn diameter must not be less than 34.8 mm. The apex angle of 60° is in accordance with TC 16. The friction sleeve in accordance with DIN 4094 has a surface area of 150 cm^2. According to TC 16 the diameter of the friction sleeve must not be less than the actual diameter of the cone base. Usually the cone and friction resistance are measured. It is also usual to measure the pore water pressure with a piezo cone and the inclination but this is not mentioned in DIN 4094. The filter shall have the same diameter as the tip und a height not less than 3 mm, but this is not a rule in Germany. According to DIN 4094 the resistance may be measured at the tip or at the thrust machine by inner rods. This last method of mechanical instrumentation is subject to error because the mass of the inner rods. Therefore this method is not used anymore in very soft and poorly compacted layers. Now there are also other tips with a 15 cm^2 surface cone area of and a 225 cm^2 or 350 cm^2 friction sleeve. There is no preference today as particular size. Much experience has been collected with the different tips but this has not yet been published.

3.2 Test procedure including calibration

The test equipment is set up with the penetrometer vertical.The deviation from the vertical must be not more than 2 %. The penetration shall be kept at a constant rate of 2 ± 0.5 cm/s. For each penetration test the penetration rate is checked and recorded at least once. The depth of the tip during the test must be measured correctly,the thrushing machine shall not move during the test. If a mechanical cone penetrometer with double linkage is used, the internal rod, the friction sleeve and the conical tip have to be checked for freedom of movement. The electrical measuring elements: tip, cable and cable connections shall be checked for mechanical damage and watertightness. Each time the penetrometer is used, the calibration of the tip shall be checked. If a friction reducer is used it shall be placed at least 100 cm above the tip. Some

users don't agree with this demand by reason of the difficult handling.
Depending on the penetrometer the electrical measurement is continous with readings taken every 1, 2 or mostly 5 cm. The mechanical measurement is intermittent and readings are taken every 20 cm. For the electronic readings there should not be any shift in the zero point following the measurement. The measurement range is to be matched to the anticipated readings. The uncertainty shall not exceed 5 % of the reading or 1 % of the effectiv range of the penetrometer tip employed. Prior to each test, a check of diameters is made to ensure that they are not less than the permitted values. The penetrometer rods shall be straight. The deviation should not exceed 1 mm over 1 m rod length. The precision of the measuring instruments shall be checked at least once every 6 month, unless the manufacturers specify shorter intervals.

3.3 Correction and presentation of test results

Corrections of the measured values should not be required. In case of greater error than allowed, the equipment must be repaired. Usually in Germany the recommendation of the wellknown TC 16 is followed except the scale of the x-axis which depends on the tip load capacity. Measurements in very poorly compacted or soft soils require an adapted scale for the horizontal axis. The vertical depth scale is usually 1: 100 but can deviate in accordance with other graphs of the site investigation.

3.4 National codes

The national code DIN 4094 describes the penetration testing of soil by the Section Baugrund of the Normenausschuß Bauwesen (Building and Civil Engineering Standards Committee), DIN Deutsches Institut für Normung e.V. and the Deutsche Gesellschaft für Geotechnik formerly Deutsche Gesellschaft für Erd- und Grundbau (German Association for Earthworks and Foundation Engineering). It contains the scope and field of application of Dynamic Probing, Standard Penetration Test (with the falling weight in the borehole) and Cone Penetrometer Test. The equipment and its checking, the measurement, field record and reporting of results and some factors influencing the results of penetration testing are described. The appendix gives some aids to application and interpretation of the results e. g. bedding density, relative bedding density, stiffness coefficient dependent on stiffness modulus, angle of the internal friction of some defined soils.

Some other national codes are mentioned below which use the measured values from the CPT.

4. INTERPRETATION OF TEST RESULTS

4.1 Soil classification and stratigraphy

The criteria for soil identification used in Germany trace back to the experiences of Begemann (1965), who described soils according to the relationship : local friction / cone resistance, now called friction ratio.Sanglerat (1972) gives some modified values for soils depending on the cone resistance which are also used in Germany. This seems to be a practicable possibility to generate a realistic lithologic profile. In spite of this interpretation there are uncertainties which can only be cleared by a good knowledge of the expected local geology. The well known graph of Begemann is in German published e.g. in Grundbautaschenbuch (1990)and used for soil identification. In stratified layers the depth of the interfaces is often better determinable by CPT than by borings. Therefore drilling results in soft and poorly compacted soils are often examined and -if necessary- corrected. Thin layered soils often give a great range of measured values, so that the identification after Begemann (1965) is very difficult. Bloh and Harder (1990) propose to filter the measured values in some steps in order to get a simple graph of mean values where a break indicates an interface. For geotechnical design this way is very dangerous because weak and thin layers remain undiscovered and unconsidered.

4.2 Soil parameters

The supplement of DIN 4094 gives some soil parameters derived from the CPT values. It must be considered that it is not possible to list a set of applicable rules for evaluation so that it would possible to derive directly all the essential soil characteristics or load bearing properties from the result of CPT. Several relationships in DIN 4094 have been derived empirically from a large number of experimental results.They are transferable to other conditions provided that proof is established that the properties are the same in the various layers in which these results are applied. Stenzel and Melzer (1972) described the validity of these relationships and the criteria for the statistical edit. The parameters which are derived in this way should be on the „save side". All the interpretations need a correct soil identification and classification corresponding to DIN 4022 and DIN 18196.

4.2.1 Bedding density or relative bedding density

In Germany the bedding density is used to derive some soil parameters as loading capacity or the angle of internal friction.The relationship is also given as relative bedding density. The equations depending on the group symbol, the non uniformity factor and the ground water level. The example below (table 1) describes the relationship between the CPT and the relative bedding density of some soils.

Table 1. Relationship between cone resistance of CPT q_c and relative bedding density I_D for some soils above groundwater level

Equations	Sy	u
$I_D = -0.33 + 0.73 \log q_c$	SE	$u < 3$ (1)
$I_D = 0.25 + 0.31 \log q_c$	SW, GW	$u > 6$ (2)

Initale figure qc in MN/m²
Target figure I_D
Ranges of validity $3<q_c<30$
GW: well graded gravel.
SE: poorly graded sand
SW: well graded sand
Sy : Group symbol DIN 18196
u : non uniformity factor

4.2.2 Stiffness coefficient

The appendix to DIN 4094 describes the connection between CPT result and compressibility, in order to find a tension related stiffness modulus E_s. The stiffness coefficient ν and the stiffness exponent w according to the well known equation of Ohde are used

$$E_s = \nu * p_a \left(\frac{\sigma \ddot{u} + 0.5 * \Delta\sigma z}{p_a}\right)^w \qquad (3)$$

ν: stiffness coefficient
w: stiffness exponent 0.5 or 0.6
$\sigma_{\ddot{u}} = \gamma*(d+z)$: vertical normal tension at the base of the foundation d or at a depth z below the base of foundation in accordance with DIN 4019
$\Delta\sigma_z = i*\sigma_1$: increase in vertical tensions caused by building operations at the base of the foundation d or at a depth z derived from the graphs e.g. of Kany (1974)
p_a: atmospheric pressure

Table 2. Relationship between cone resistance $q_c$and stiffness coefficient ν

Equations	Sy	u	
$\nu = 167 \log q_c + 113$	SE	$u < 3$	(4)
$\nu = 463 \log q_c - 13$	SW	$u > 6$	(5)
$\nu = 15.2 \log q_c + 50$	TL, TM	$0.75 < I_C < 1.3$	(6)

Initial figure qc in MN/m²
Target figure ν
Ranges of validity:
$5 < q_c < 30$ for cohesionsless soils
$0.6 < q_c < 3.5$ for cohesive soils
above groundwater level
TL: inorganic clays of low plasticity
TM: inorganic clays of medium plasticity
I_C: consistency index
Sy : Group symbol DIN 18196

4.2.3 Undrained shear strength

The undrained shear strength c_u is commonly determined in Germany by laboratory test described in DIN 18137. There are a lot of efforts to derive the undained shear strength form CPT values. In EAU (1990) a relationship between

c_u and q_c is published for

$c_u = 1/14*q_c$ kN/m² for clay (7)
$c_u = 1/20*q_c$ kN/m² overconsolidated clay (8)
$c_u = 1/12*q_c$ kN/m² for soft clay (9).

Sanglerat(1972) published the equations for cohesive soils depending on the absolute value of CPT:

$c_u = 1/10*q_c$ for $q_c<0.5$ MN/m² (10)
$c_u = 1/18*q_c$ for $q_c>0.5$ MN/m² (11)

Some other relationships depending on classified soils as silt, organic silts, silty peat or mud are being prepared and shall be published soon. They will provide a greater certainty for the derived characteristic values.

4.2.4 Angle of the internal friction

Only for poorly graded sand the supplement of DIN 4094 gives a relationship between the angle of internal friction phi and CPT q_c:

$$\varphi = 13.5 \log q_c + 23 \quad (12)$$

in the ranges of validity $5<q_c<28$ and the non uniformity factor <3,where
φ is the target figure in degrees and
q_c is the initial figure in MN/m².

It is also common to estimate the angle of internal friction according to the bedding density or relative bedding density published e.g. in EAU(1990).

5 USE OF CPT IN GEOTECHNICAL DESIGN

The CPT graphs are used to collect all layers with equal or similar geotechnic behaviour. Characteristic or design values are commonly determined by laboratory tests which need undisturbed specimen and take a long time to carry out.Therefore the relative quick and low cost CPT test shall ascertain the geological profile and the stratigraphic range and supplement the low number of laboratory tests.

5.1 Pile capacity

The bearing capacity of piles or bearing resistivity may be evaluated from the CPT results. However it is common in Germany to correlate the results of static load tests with the CPT results. In DIN 4014 part 2 (bored piles; large bored piles;c onstruction procedure, design and permissible load) a sufficient bearing capacity is given if q_c is more than 10 - 15 Mpa (depending on the type of soil). DIN 4014 part1 (bored piles; conventional type) the bearing capacity of non-cohesive soils is sufficient if the bedding density is $D > 0.4$ for $U < 3$, $D > 0.55$ for $U > 3$ or $q_c > 10$ Mpa.

5.2 Settlement calculation

The parameters such as stiffness coefficient or drained Young's modulus can only be estimated from the CPT in a wide range on the basis of local experience. In Germany the oedometric modulus (one dimension consolidationtest) is commonly in use to calculate the settlement. Now there are some correlations between the oedometric modulus and CPT test under study and discussion. The results are not yet available. Although the oedometer test is very often used in Germany there are only recommendations but not a DIN standard.

5.3 Stability of slopes or dams

As mentioned above for the stability calculation (DIN 4019; subsoil; analysis of bearing capacity) the cohesion of undrained soil c_u and the angle of internal friction are obtained by the unconsolidated and undrained triaxial test (DIN 18 137). The undrained shear strength derived from the CPT results is most important for road constructions on peaty, silty and clayey soils without soil replacement. So CPT has proved to be a useful means of soil investigation on saturated silt and peats in different states of composition. However it is not possible to give only one equation to describe the relation between CPT and laboratory tests. DIN 1054 (subsoil; permissible loading; of subsoil) assumes $q_c > 7.5$ MPa for the application of the admissible consolidation of the subsoil.

6. COMPARISON AND CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

The above mentioned soil parmeters such as relative bedding density are also available from dynamic penetration testing and the standard penetration test which are also described in DIN 4094.In very soft soils where it is impossible to obtain really undisturbed samples, the use of the field vane test DIN 4996 is common to obtain the shear resistance in situ c_v and to derive the undrained shear strength. Bjerrum(1973) gives the relation

$$c_u = \mu * c_v. \tag{13}$$

c_u: undrained shear strength
cv: value of the field vane test
μ: correlation factor dpending on the plasticity index I_p

I_p	0	30	60	90	100
μ	1.0	0.8	0.65	0.58	0.50

In very soft soils as inorganic and organic silts and peat with $I_p > 90$ some experiences prove that the postulated diminution is not valid if the shear resistance is between 5 and 20 kPa. Some experiments with sand dams on soft soils(height 3.5 m, width 60*60 m^2) proved that the the shear strength c_v (DIN 4096 provided)is equal with the laboratory result of c_u (Moritz et al. 1983). The correlation of q_c and c_v was made by Faust (1990) for silty soils:

silty soil n = 485
$$q_c = 8.83\ c_v + 131.55 \quad R = 0.79 \tag{14}$$

silt,low sandy,clayey,organic n = 86
$$q_c = 9.40\ c_v + 124.97 \quad R = 0.85 \quad (14)$$

silt high sandy,organic n = 80
$$q_c = 6.17\ c_v + 170.17 \quad R = 0.73 \tag{15}$$

silt, low sandy, clayey, high organic n = 121
$$q_c = 8.36\ c_v + 116.62 \quad R = 0.80 \tag{16}$$

silt, sandy, clayey, organic n = 90
$$q_c = 8.78\ c_v + 134.83 \quad R = 0.79 \tag{17}$$

silt, sandy, organic n= 108
$$q_c = 8.44\ c_v + 141.91 \quad R = 0.82 \tag{18}$$

The result is that the CPT values may be used to fill up the missing links between the vane shear and the laboratory test. Taking, the natural water content into account better correlations can be expected. Other authors are working on the criteria for more correlations taking established experiments into account, because the CPT is an economic alternative to the other tests.

7. MAJOR AREAS FOR RESEARCH ACTIVITIES

The CPT is used in very soft soils e.g. for road constructions to receive soil identification and c_u in:
gravel and sand to ascertain the bedding density for calculation of settlements and bearing capacity, *weathered rocks* to estimate the settlement behaviour for shallow foundations and *karstified regions* to find hidden dolines.

8. FUTURE TRENDS

The CPT is a proven means to measure the cone resistance and the local friction. The experiences are made with the 10 cm^2 cone and the 150 cm^2friction sleeeve. Also in use are cones with 15 cm^2 and larger friction sleeves. But there are no correlations published today. The larger cones can be driven deeper into solid soils because stronger driving rods can be used.

Other measurements with such as pore water pressure, seismic logging, temperature measurement are possible. But it must be considered, that all the derived parameters mentioned above depend on a constant speed of the tip. Some other added measurements require a long time so that in that case the sounding is not longer continous.

New developments of the tip or test equipment are not expected. The trend is to get more security in the derivation of soil parameters and as far as possible to get better correlations for characteristic and design values.

9. REFERENCES

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Cone Penetrometer Testing in Hungary in the last two decades

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SYNOPSIS: Cone penetrometer is used for logging-, rheological-, geophysical- and in situ permeability-testing in Hungary. Research is done in the following fields: (1) elaboration and validation of stochastic relationships between CPT data and geotechnical data, (2) theoretical and experimental studies concerning the rheological processes that take place after penetration, (3) geological information assessment on the basis of CPT data. Future trends are related to the development of the rheological type and the geophysical type CPT tests.

1 GEOLOGICAL CONDITIONS

Hungary occupies the central part of the intermontane basin of the Alpine belt of Europe - the Carpathian Basin. Hungary's surface area can be divided into four major geographical units (Rónai, 1968, Fig. 1).

(1) The Hungarian Central Mountains are Mesozoic mountains trending SW-NE. The average elevation is about 300 m. The mountains are flanked by two plains - the Little Plain to the northwest and the Great Hungarian Plain to the southeast.

(2) The Little Plain has a basement of Palaeozoic sediments on the west and Mesozoic rocks on the east buried 2000 - 3000 m deep.

The surface is covered by Holocene and Pleistocene fluviatile sediments: *gravel and coarse sand* are exposed over vast areas.

(3) The Transdanubian Tableland is composed of Late Tertiary deposits that have not undergone any marked subsidence. It is covered by thin sheets of Quaternary (mainly) *loess* deposits.

(4) The Great Hungarian Plain is the largest Neogene Depression of the Carpathian Basin filled up with Quaternary deposits. It includes two hilly regions (Danube-Tisza Interfluve and the northeast part of the Great Plain). Between them an almost perfect plain region (the Tisza-Körös Plain) is situated where even 3-4 differences in relative height are rare.

The former regions are characterized by sandy-silty hills rising 30 to 60 m above the mean surface level of the Great Plain (100 m).

The Tisza-Körös Plain is covered by fluviatile, lacustrine (mainly) *plastic* deposits and by *infusion loess* up to the depth of interest. Alkaline soils can also be found in some areas.

It can be concluded that quaternary deposits are found on the major part of Hungary's surface area that can generally be tested in situ.

2 IN SITU INVESTIGATION METHODS

Various in situ testing methods have become widespread in Hungary in the last two decades:

- CPT
- CPT combined with geophysical methods
- CPT combined with hydrosounding
- vane test
- dynamic sounding
- pressuremeter test
- weight sounding
- SPT

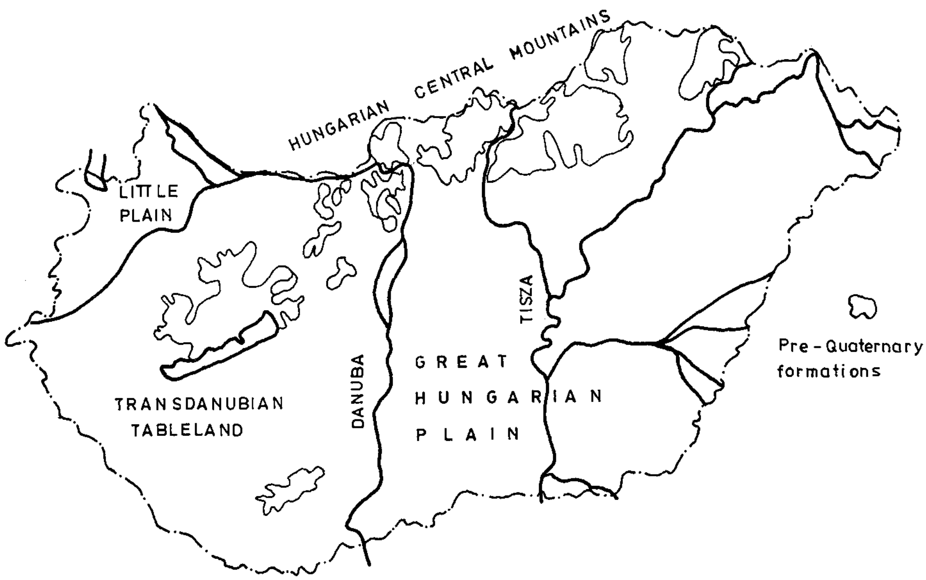

Figure 1 The four major geographical units of Hungary's surface area

3 TYPES OF CPT EQUIPMENT

3.1 S832 and Borro

Two types of electrical CPT equipments have been used in Hungary : S832 and Borro. Main features of them are presented as follows.

The general geometry of the penetrometer tip and of the cone is in accordance with the recommended standard (ICSMFE, 1977) in both cases. The friction sleeve is located immediately above the tip with surface area of A=350 cm^2 for the S832 system and A=250 cm^2 for the Borro system.

The *logging test* is performed by periodical pushing in the case of the S832 cone (the rod of the penetrometer being released after every one meter of stroke) and pushing continuously with the Borro cone.

The rate of penetration may vary between 0.0065 to 3.75 m/min for the S832 equipment and is equal to 1 m/min for the Borro equipment.

The maximum pushing force is 100 kN in the former and, 200 kN in the latter case.

Both machines are self-propelling, the S832 track is anchored before testing.

The measuring error is less for the S832 than the for Borro due to the more sophisticated measuring ranges.

A *rheological test* can be performed with the S832 equipment in such a way that the steady penetration is stopped at any depth, the rod is kept in clamped position, the local side friction and, the cone resistance are measured 'until they become constant'. The duration of the simple rheological test with S832 penetrometer is generally 2 to 4 minutes.

Results of both the continuous and the rheological tests are recorded in 'analogous' diagrams that are digitalized by the users.

The Hungarian regulation comprises manuals and design guidelines (Rév et al (1979), Juhász et al (1981), Tényi and Párdányi (1981).

3.2 EGS test

The so called Engineering Geophysical Sounding (EGS) method has been developed in the Eötvös Lóránd Geophysical Institute of Hungary since 1975. The traditional CPT is completed with the measurements of the

following variables:
- natural gamma intensity ($I_{r\gamma}$) using a KJ crystalline detector and a photomultiplier,
- gamma intensity (I_{γ}) with Cs^{137} isotope as a source,
- neutron intensity (I_n) with Am-Be isotope as a source.
- pore water pressure (u).

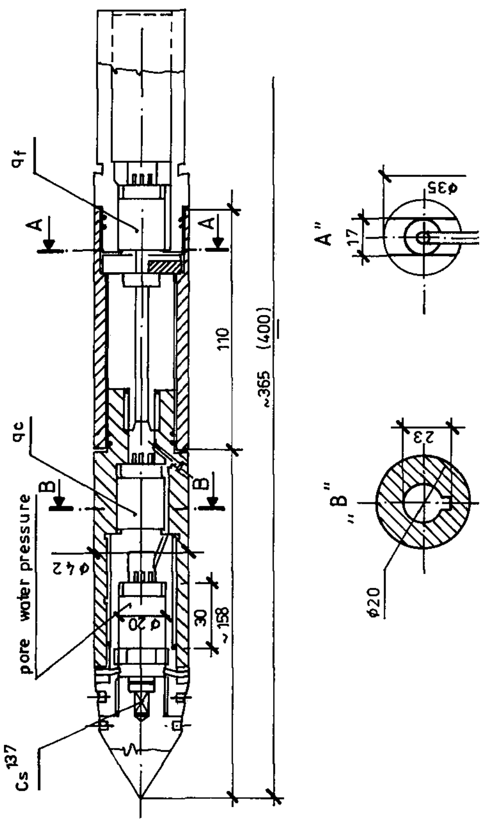

Figure 2 The EGS cone

The maximum pushing force is 200 KN. The machine can be self-propelling or portable.

The measuring cone (Fig. 2) is pressed down into the soil with a steel tube of 42 mm diameter.

Measurements are made in two phases. During the steady penetration the cone parameters are measured. Geophysical sensors are gradually lowered inside the tube once the steady penetration is stopped. Data are collected at every 5 to 10 cm of depths with a completely automated data management.

The metal cone can be substituted by a filter cone that allows both water adding and withdrawal. Soil sampler can also be attached.

3.3 Hydrosounding

The cone is pushed into the soil statically or dynamically. Water container is attached and pressurized water is added at predetermined depths while the penetration is stopped.

A cylindrical part above the cone (Fig. 3) is covered by a brass filter, the main dimensions are d=0.02 m, l=0.02 to 0.05 m. The conical part above the filter ensures a locking effect by means of the compaction of the soil.

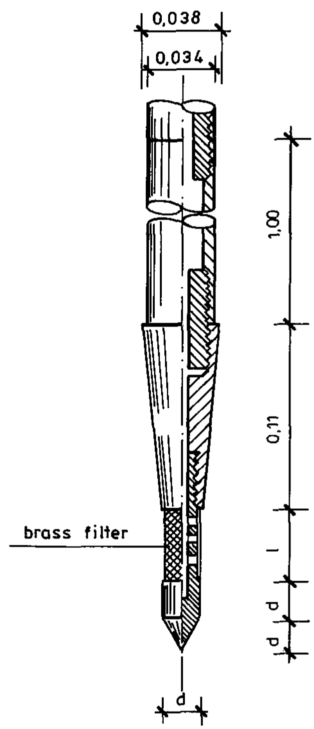

Figure 3 The hydrosounding cone

The cone resistance (alternatively the number of blows) is measured in the usual way during penetration. The water pressure and the water intake per time are recorded when the steady penetration is stopped and water is continuously supplied (Szvák, 1983).

4 INTERPRETATION

4.1 S832 and Borro

In the manuals (Juhász et al, 1981; Tényi and Párdányi, 1981) the soil classification systems of Begeman and Schmertman are mentioned, moreover, the stochastic relationships between the cone resistance (q_c) and

- the undrained strength (s_u),
- the relative consistency index (I_c),
- friction angle (φ) for granular soils,
- bulk density (ϱ) for granular soils and,
- oedometric modulus (E_{oed})

are suggested on the basis of the works of Voskovstchuk (1973) and Sanglerat (1979).

As an example, the relation q_c - I_c is visualized in Figure 4 where results of a research is also shown (see Chapter 7.6).

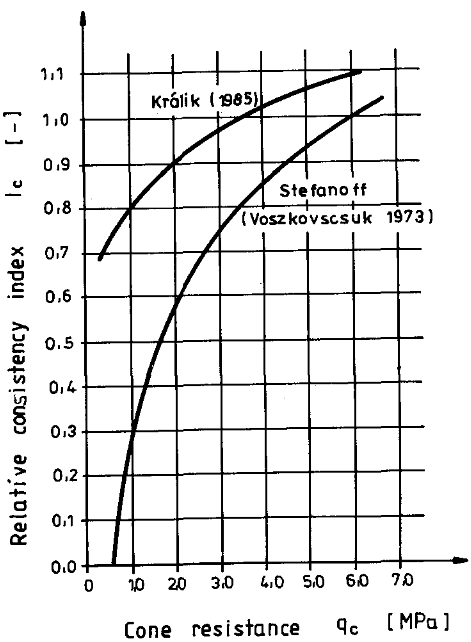

Figure 4 Empirical relation q_c - I_c

4.2 EGS

The theoretical background of the geophysical measurements can be summarized as follows.

Radioactive isotopes are frequently joined to clay minerals and, therefore, the natural gamma activity ($I_{r\gamma}$) can be related using a well-known formula to the clay content of the soil.

The attenuation of the gamma radiation (I_γ) or neutron radiation (I_n) is dependent on the bulk density (ϱ) and the water content (w) of the soil, respectively. These relations are determined with calibration beforehand.

The degree of saturation (S), the void ratio (e) and the dry bulk density (ϱ_d) are computable on the basis of the foregoing parameters (Fejes, 1994).

The diagram shown in Figure 5 is used for soil classification on the basis of the cone resistance (q_c) and natural gamma activity ($I_{r\gamma}$).

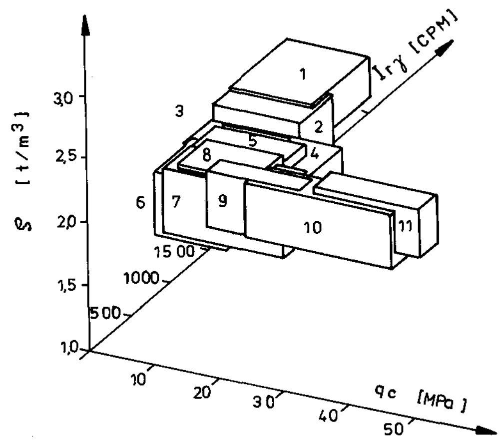

Figure 5 Classification for EGS. 1. clay, 2. lean (silty) clay, 3. (clayey) silt, 4. silty sand, 5. sandy silt, 6-7. fine sand, 8. sand, 9. coarse sand, 10. coarse gravelly sand, 11. (sandy) gravel

5 USE OF CPT IN GEOTECHNICAL DESIGN

The manuals suggest that the stochastic relation

$$P_t = A + B \cdot P_{CPT} \tag{1}$$

is reasonable to be established on the basis of local data for any specified geotechnical area by statistical method; where P_t and P_{CPT} are ultimate bearing capacity of piles from loading test and from simple rheological type CPT test data respectively; A, B are parameters; P_{CPT}

can be computed with the following formula:

$$P_{CPT}=q_c^{\prime}\cdot F+U\sum_{i=1}^{n}f_{si}^{\prime}\cdot l_i \tag{2}$$

where F is cross section area; U is periphery of pile; l_i is the length of the i-th pile segment, $q_c^{\prime}$ is the mean cone resistance (q_c) around the pile tip, $f_{si}^{\prime}$ is the reduced local side friction (f_{si}). The reduction can be assessed on the basis of a table in the function of the distance above the pile tip, the pile length and the measured value (Voskovtchuk, 1973).

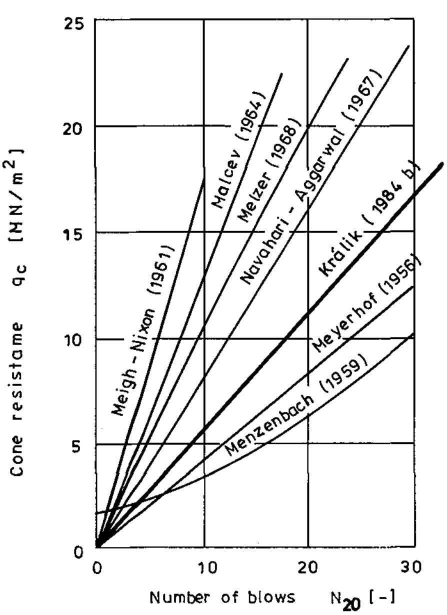

Figure 6 Relations between q_c and N_{20}

6 CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

Stochastic relations between the number of blows N_{20} in heavy dynamic sounding (50 kg, 50 cm) and the cone resistance q_c [MN/m^2] are suggested by Králik (1984b) for cohesionless soils :

$$q_c=1.095+0.476\cdot N_{20} \tag{3}$$

for silty sands :

$$q_c=0.790+0.515\cdot N_{20} \tag{4}$$

and, for clayey soils :

$$q_c=0.850+0.296\cdot N_{20} \tag{5}$$

on the basis of 470, 110 and 50 data pairs respectively. Relation (3) is compared with other expressions in Figure 6.

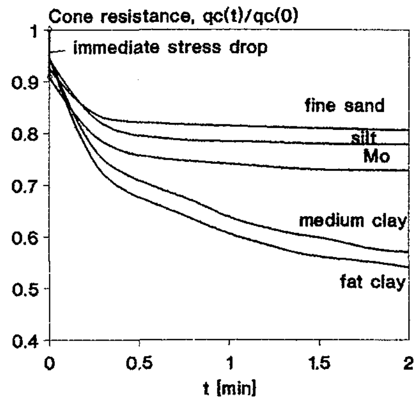

Figure 7 Measured q_c-time relations

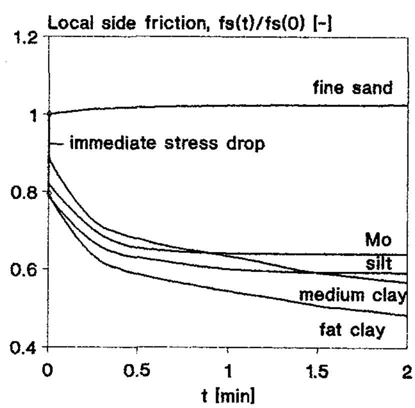

Figure 8 Measured f_s-time relations

7 MAJOR AREAS FOR RESEARCH

7.1 Soil profiling from rheological CPT data

Typical simple rheological type cone penetration test records (Fig. 7, 8) are characterized by an immediate stress drop and a subsequent time-dependent stress decrease except in the following case. Local side friction in sands slightly increases with time (without immediate stress drop in the case of coarser sands, Imre et al 1989; Imre, 1993).

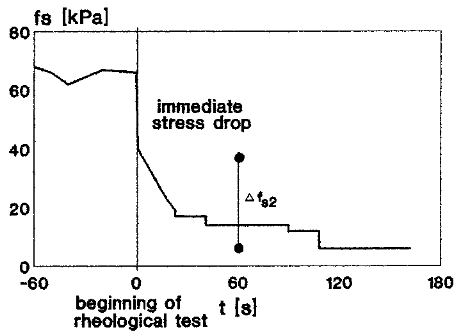

Figure 9 Cone parameter Δf_{s2}

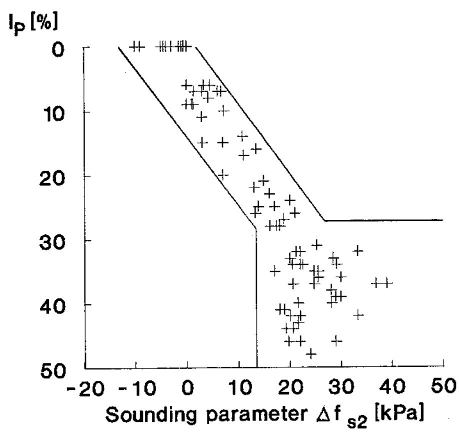

Figure 10 Δf_{s2}- I_P relation

Simple empirical parameters are used for the characterization of the rheological type cone penetration test records (Imre et al, 1989).

Only one parameter is mentioned herein (Fig. 9). The local side friction parameter Δf_{s2} is the following difference :

$$\Delta f_{s2} = f_s(t_i) - f_s(t_i + \Delta t); \tag{6}$$

where t_i is the moment when the immediate stress drop is ended and, Δt=120 s.

Factor analyses are successfully carried out on 103 samples composed of soil physical parameters and rheological type cone penetration test parameters (Imre et al, 1989).

Strong correlation is observed between sounding parameter Δf_{s2} and the plasticity index (I_P) if the latter is less than a certain value between 20 % and 30 % (Fig 10).

This relation is verified with the results of about 170 further rheological type CPT made in different parts of Hungary (Imre, 1993).

Deviation from the trend is encountered in the vicinity of layer boundaries and in layers with secondary structure (Imre, 1995a).

7.2 Pile bearing capacity

7.2.1 Bearing capacity of Franki piles

Two empirical relationships are suggested by Králik (1984f) between the bearing capacity of Franki piles determined with loading test (P_t, kN) and continuous CPT data:

$$lg\frac{P_t}{n_1} = a_1 \cdot lg\frac{q_c' \cdot F}{n_1} + b_1 \cdot lg\frac{U \cdot \sum_{i=1}^{n} l_i \cdot f_{si}}{n_1} + c_1 \tag{7}$$

and

$$lg\frac{P_t}{n_1} = a_3 \cdot lg\frac{F \cdot U \cdot A_t}{n_2} + b_3 \tag{8}$$

where q_c' is mean cone resistance around the pile tip [kN/m2]; F is cross section area of the pile [m^2], U is periphery of the pile [m], l_i is length of a pile segment [m], f_{si} local side friction [kN/m^2], A_t [kNm] is work done by the cone penetrometer; n_1=1 kN, n_2=1 kNm^4. The dimensionless coefficients a_1, a_3, b_1, b_3 and c_1 - shown in Tables 1, 2 - were elaborated from data concerning such 38, 13, 42 Franki piles where the tip was situated in sand, silty sand and clay, respectively.

7.2.2 Shaft yield load for bored piles

The first yield point of the load-settlement curve of a staged pile load test can be determined as follows (Imre, 1987). The time dependent settlements (the differences of the instantaneous settlement and the settlements at various elapsed times) are computed and plotted as function of the load at the stage. The upper contour of time dependent settlements contains two straight line segments. The point of the load test curve related to the end of the first line segment (Fig. 11) is the first yield point that is called shaft yield point.

Table 1 Parameters for formula (7)

Soil type	a_1	b_1	c_1
cohesionless	0.128	0.112	2.149
silty sand	0.095	0.083	2.412
plastic	0.137	0.101	1.978

Table 2 Parameters for formula (8)

Soil type	a_3	b_3
cohesionless	0.62	-0.23
silty sand	0.64	-0.33
plastic	0.64	-0.29

It is assumed that the load related to the shaft yield point is approximately equal to the shaft yield load. The following formula is suggested for the calculation of the shaft yield load on the basis of simple rheological CPT data in the case of plastic soils and continuous CPT data in the case of cohesionless soils (Imre, 1987) :

$$S_{yield} = U \cdot \sum_{k=1}^{n} l_k \cdot \min_{j=k}^{n} f_{sj} \qquad (9)$$

where l_i is the length of the i-th pile segment with local side friction f_{si} (i=1 for the uppermost element). Equation (9) expresses that the unit side resistance cannot be greater at any given depth than the smallest unit side resistance value under this point up to the depth of interest.

This suggestion is supported by 10 examples (Imre, 1987). It can be noted that similar reduction principle is included into the Hungarian National Pile Code and, some reduction for shaft resistance is suggested by Voskovtchuk (1973), too (see Chapter 5).

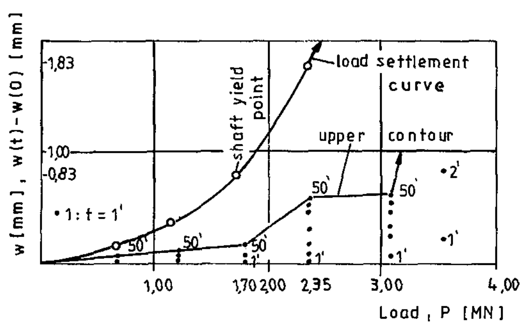

Figure 11 Construction of the shaft yield point from staged load test data

7.3 Modelling of rheological type CPT test

The rheological type cone penetration tests (dissipation tests, piezo-lateral test cell, simple rheological type test with S832 equipment) are very promising in the determination of the in situ coefficient of permeability (k). These tests can be used in nearly every type of soil and do not need water adding or withdrawal.

Existing models cannot successfully be applied for the evaluation of the major part of these tests. For example, the rheological type CPT test results are not evaluated in the lack of proper physical-mathematical model.

In an earlier research (Imre, 1991/1992) it is noticed that existing consolidation theories apply the assumption that the radial total stress is constant at the shaft-soil interface (coupled theories use this assumption as a boundary condition, uncoupled theories consider that the total stress state is constant). They predict a monotonous increase in the radial effective stress acting on the shaft with final value being significantly larger than the horizontal effective stress valid prior to penetration.

These statements are not verified by experimental results.

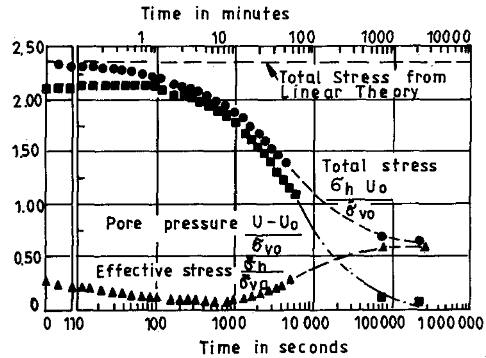

Figure 12 Piezo-lateral stress cell measurement (Baligh et al, 1985)

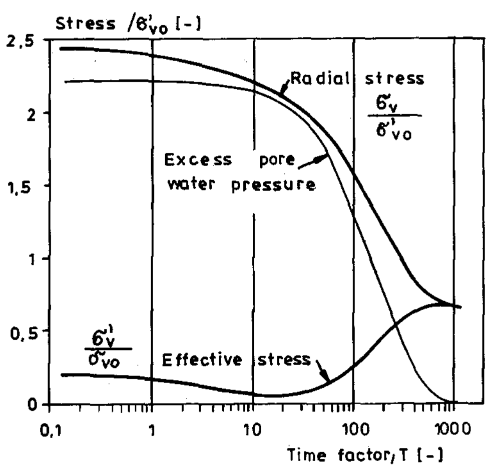

Figure 13 A simulation with the model

According to the result of the piezo-lateral stress cell measurement made in Boston Blue Clay (Fig. 12, Baligh et al, 1985), the radial total stress at the shaft decreases by 73% of its initial value until the pore water pressure dissipates. The radial effective stress decreases in the first minutes and, its final value is about equal to the horizontal effective stress valid prior to penetration. Simple rheological tests results reflect similar tendencies (Fig. 7, 8).

A one-dimensional model is elaborated (Imre, 1991/1992) where geometrical boundary condition is applied at the shaft-soil interface (constant radial displacement) instead of the static type used by the existing theories and, the relaxation is also taken into account.

According to the results of some simulations, the suggested model explains the large total stress decrease experienced with the piezo-lateral stress cell (Figs. 12, 13).

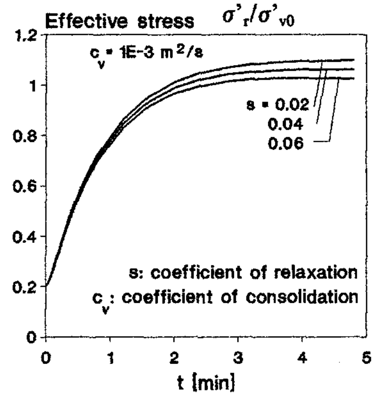

Figure 14 Simulated effective stress on the shaft ($c_v=10^{-3}$ m^2/s)

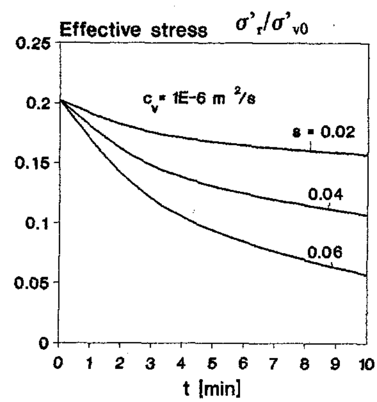

Figure 15 Simulated effective stress on the shaft ($c_v=10^{-6}$ m^2/s)

The model also explains the qualitative difference in the local side friction records of the simple rheological test for clayey and sandy soils (Fig. 8) since the time variation of the simulated radial effective normal stress on the shaft is an increase for great and a decrease for small coefficients of consolidation (Imre, 1990a, Figs. 14, 15).

Massarch and Broms (1977) suggested that hydraulic fracturing may occur during penetration in the vicinity of the shaft. This suggestion is also supported by the model.

The foregoing one-dimensional model is similar to the describing model of the oedometric relaxation test (Imre, 1990b, 1994, 1995a). It follows that the oedometric relaxation test can be used for studying the soil behaviour during and after penetration (e.g. the immediate stress decrease).

7.4 Geological information assessment of simple rheological CPT test data

An earlier geological layer description is modified on the basis of the statistical evaluation of simple rheological type cone penetrometer test parameters and laboratory tests data. Comments concerning the depositional and the post-depositional effects are made (Imre, 1995b).

7.5 Special depths from CPT data

The mode of failure is changing while the cone tip advances from the soil surface to larger depths. The limit depths (t_h) is such depth below of which the failure pattern is constant. The quantity η can be defined as the ratio of the limit depth (t_h) and the cone diameter (d).

According to Králik (1984d), the well-known limit depth theories can be used only for dense and very dense soils.

His suggestions are the following relations for cohesionless

$$\eta = 42.47 - 1.64 \cdot \frac{q_c}{n_3} \tag{10}$$

and for clayey soil

$$\eta = 25.71 \cdot e^{-0.079 \cdot \frac{q_c}{n_3}} \tag{11}$$

using 38 and 45 data pairs, respectively; where n_3=1 MN/m^2, q_c is cone resistance in MN/m^2. Králik (1984e) states that some decrease in the measured continuous q_c and f_s data can be experienced once the depth of the phreatic line is reached in cohesionless soils. No similar decrease is experienced in the case of silty, clayey soils.

7.6 Parameters from CPT data

The following relation is suggested by Králik (1985) for the bulk density (ϱ) of cohesionless soils on the basis of the cone resistance (q_c) :

$$\frac{\varrho}{n_3} = 1.559 + 0.13 \cdot \frac{q_c}{n_3} \tag{12}$$

where ϱ is in t/m^3, q_c is in MN/m^2, n_3=1 t/m^3, n_4=1 MN/m^2.

The following relation is given between the relative consistency index (I_c) and the cone resistance (q_c) of clayey soils:

$$lgI_c = 0.167 \cdot lg\frac{q_c}{n_5} - 0.591 \tag{13}$$

where q_c is in kN/m^2, n_5=1 kN/m^2. The latter relation is visualized in Figure 4.

8 FUTURE TRENDS, NEW DEVELOPMENTS

New developments with EGS cone penetrometers comprise the inclusion of seismic wave measurements and soil resistivity measurements.

Cone penetrometers with pore water pressure element are intended to be developed so that dissipation tests can be routinely used.

Theoretical work is planned in the field of the evaluation of the rheological type cone penetrometer tests including the solution of both the direct and the inverse problems.

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The Icelandic National Report

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SYNOPSIS: This National Report lists the state of practice in Iceland, and what is planed to do in the next few years.

1. INTRODUCTION

Loose sedimentary formations where CPT test are to be considered are sand, silt and clay. All this sediments in Iceland are of volcanic origin and made of basalt. In the inland areas, the sand is usually porous volcanic ash which is very angular. Along the shore the sediment is usually silty, organic and mixed with fragments of shells. The silt is usually non plastic and includes many thin leaky lenses.

In Iceland it is most common to investigate the soil with Dynamic probing test. For earthquake analysis, SPT and CPT tests have been carried out for some projects. SPT drilling equipment have been available in Iceland for some years and the test results seem to be reliable. CPT drilling equipment have on the other hand been brought to the country for some projects in the last few years. A complete CPT drilling equipment have now been ordered, and will be ready for operation in the autumn 1995.

2. CPT TESTS IN ICELAND

CPT tests have been performed three times in Iceland. During the first two projects, a Cone from A.p. van den Berg, was used , but the last project was carried out using Memocone. The Icelandic Public Roads Administration have ordered Memocone which is expected to be ready for operation in the autumn 1995. No National code or standard exist fore CPT tests in Iceland, but it is expected that Norwegian methods and codes will be adopted as is the case for most drilling tests in Icelandic soil.

3. INTERPRETATION OF TEST RESULTS

Soil classification charts prepared by Robertson (1990) are used for classifying the soil type. Evaluation of CPT data to obtain shear strength parameters and settlement parameters has been done according to methods outlined by Senneset et al. (1988). Pore pressure dissipation tests are used to estimate coefficient of consolidation according to the interpretation proposed by Robertson and al. (1992).

4. USE OF CPT IN GEOTECHNICAL DESIGN

CPT tests have turned out to be a reliable method to obtain soil classification and stratigraphy. Measurements of pore pressure dissipation in CPT tests are easily done and the results seem to be reliable when compared with settlement measurements of fills on similar soil.

The bearing capacity have been estimated by CPT test according to Robertson et al. (1988), Bustamante and Gianeselli (1982) and Norwegian guidelines (1991). Load test on piles, have shown that actual pile bearing capacity is much higher than calculated by CPT tests using the aforementioned methods.

Settlement and slope stability is of considerable interest in silt and silty clay. It is very important to be able to estimate the rate of consolidation in the sediment because it governs the rate of strengthening of the sediment, which is of crucial interest regarding the allowable filling rate for embankment construction. It is hoped that by measurements of pore pressure dissipation in CPT tests, the coefficient of consolidation can be estimated with sufficient accuracy. Comparison of shear strength parameters from triaxial tests and CPT tests show that CPT test underestimates it considerably. Settlements parameters estimated from CPT tests are not adequately accurate as compared to results from oedometer tests. CPT tests are used in Iceland to estimate liquefaction potential in loose overburden according to Seeds methods.

5. FUTURE TRENDS

In the next few years it is planed to perform CPT test on 5 to 10 locations. It is expected that most of the following information are available from earlier investigations at each spot:

Dynamic probing test
Soil classification and stratigraphy
Oedometer tests
Triaxial tests
Pile loading tests
Settlements measurements

Based on these comparison the best interpretation of CPT test in the basaltic sediment of Iceland can hopefully be outlined.

CPT tests are already used to estimate liquefaction potential in loose overburden. It is planed to test further sediments that are expected to liquefy. An earthquake, of the order of 6,5 to 7,0 on the Ricter scale, is expected on the southern part of Iceland in the next few years. It is hoped that after such an earthquake, the interpretation of CPT test for use in earthquake analysis of sediments will be much improved. Shear modulus have not often been estimated in Icelandic sediment, but it is anticipated that Seismocone can be useful for such an estimate.

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STATE OF ART OF CPT IN INDIA

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SYNOPSIS : This national report is an attempt to compile the publications available on CPT in India. The member representing Indian Geotechnical Society (IGS) is convenor of a national committee to produce a document and recommendations related to penetrometers. SPT and DPH are widely used in India. Though used widely on major irrigation projects - Earth dams in India, CPT is relatively least used test for general explorations. Unfortunately the reports of major projects and decisions on interpretations are not handy. A questionire was issued to manufacturers, users of CPT. All published papers were scanned to frame this report.

The Bureau of Indian standards (BIS) publications are compared with TC-16 recommonded reference test CPT. The practice of analysis of data, interpretation of engineering properties, case experiences are then presented.

1.0 GEOLOGICAL AND GEOTECHNICAL DATA

India is bounded by Himalayan ranges in North, Indian Ocean in South, Arabian Sea to its West and Bay of Bengal in East. India, seventh largest country of world, with 3.276 million square kilometer has all types of geological and geotechnical formations.

The broad outline of geology of exact peninsula (Himalaya's), Indogangatic alluvial plains and peninsula of south projecting into Indian Ocean and desearts of Rajasthan is essential to appreciate Geotechnical formations.

The extra peninsula has sedimentary rocks in cold climate. The alluvial Indo-gangatic plains are underlaid by sandstones. The peninsular south comprises of Besalt, Granite and Metamorphic rocks.

Based on physiography, wide range of climatical variations, geological formations, the soils of India could be broadly devided into following groups.

(a) Marine deposits.
(b) Alluvial deposits.
(c) Latteritic soils.
(d) Black cotton (expansive) soil.
(e) Red murrum.

Marine deposits are located on tidal flats along long coast and Rann of Kutch covering about 45,000 sq.km. Entire E-W belt above Vindhyan ranges and below Himalayan ranges is covered by 100 m alluvium. Many rivers have delta deposits and flood planes having stratified alluviums. Wind blown sand deserts covers 5,00,000 sq. km. in Rajasthan. Sand dunes 15 m high generally, rise even upto 150 m height.

The residual soils information is piecimeal and scattered. The laterritic soil has been analysed by geological experts and expansive soils have been solved by R&D group. Map of residual soil of India (Desai M. D. 1985) shows deposits of black cotton and red latteritic deposits.

The subsoils (marine, alluvial, residual and noncohesive sands) have been explored by CPT. The work done is enormous but the published documents are very few.

2. TYPE OF PENETRATION TESTING AND OTHER INVESTIGATION METHODS USED IN COUNTRY

Penetrometers have been widely used by investigators for hundreds of major irrigation and power projects and thousands of highrise buildings. The most popular test is SPT for its universal acceptibility. The DCP (Dynamic cone penetration) test very similar to DPH reference test, is very widely used since 1966. It is quick, cheap, low capital intensive, more dependable in high water table regions, less sensitive to skill of operators.

Desai M. D. (1970) : Subsoil exploration by dynamic penetrometer has developed its interpretations to ascertain, (a) zoning and stratification (b) Tentitative relative density, ϕ, modulus of elasticity, water table and insitu CBR for non cohesive soil. (c) Cohesion and modulus of compressibility for cohesive soil. (d) Probable type of subsoil for known geographical region. This test costs Rs. 700 to Rs. 1000 ($ 30) for exploration upto 8 to 10 m depth. Using analysis of DCPT data, selected bores with limited and accurate sampling and more statistical data of critical parameters for more accurate interpretation, is economically viable. The results of DCPT and DPH reference test (ISSMFE) have been found compatible by Desai M. D. & Mehta B. J. (1982).

CPT was introduced in India around 1962-63 with imported Norwegian equipment. Wide field use was made by indegenous engine driven, 10 t unit at dreged fill sand platform at Ukai (Gujarat) dam by Central soil and material research station in 1963-64 to evaluate the fill for its liquifaction potential. The test has been picking up slowly as a check or confirmatory test for results of SPT and DCPT.

The Central Building Research Institute (CBRI) developed the other test (IS - 4968 (Part-II)) using 65 mm cone with bentonite circulation. It is cased cone test and has been correlated to predict engineering properties (Tolia D. S. et al).

3. TYPE OF CPT - EQUIPMENT USED IN COUNTRY

The Indian Standard (IS) 4968 (Part-III) - Static Cone Penetration Test was introduced in 1976. This standard is compared to standard reference test recommanded by TC-16 ISSMFE.

3.1 Equipment

The apex angle 60° and overall base diameter 35.7 mm ± 0.1 with base area 10 cm^2 is within the limits prescribed by TC-16 (Reference). The roughness of surface less than 1 micrometer, straight length above base (h_e = 7 mm) and gap between cone and other element ($e_0 \leq 5$ mm) will have to be introduced in IS (h_e of 5 mm in IS).

The friction sleeve (d_s) specified as 36 ± 0.2 ϕ mm also satisfies the limits $d_c < d_s < d_c + 0.35$ mm. The length of sleeve will have to be increased from 100 in IS to 133.7 mm. For push rod, deflection requirment at mid point not to exceed 0.5 mm is to be specified. Measuring device is hydraulic gauge with upper limit 1000, 5000, 15000 kpa having sensitivity of 25, 50, 150 kpa for 2-3 t equipment. For 10 T equipment 10,000 and 60,000 kpa with sensitivity of 100 & 500 kpa is specified in IS code.

3.2 Test procedure

The rate of pushing in IS (10 to 15 mm/s) will be revised to 20 ± 5 mm/s to make it reference test. The test is run to push cone (min. 35 mm) and then cone with friction Jacket and finally the entire assembly. Recording is done at 0.2, 0.4, 0.6 m...... interval. Indian standard requirement of calibration and practice to calibrate after mobilization from one site to another could be specified as 6 months interval as per ISSMFE.

3.3 Correction and presentation of test results

As per IS, cone resistance shall be corrected for the dead wt. of cone, sounding rod in use. Combined cone and friction resistance shall be corrected for dead wt. of cone, friction jacket and sounding rods. It is also corrected for the ratio of ram area to the base area of cone.

The cost of equipments for exploring upto 10m (Mannual) 15 to 30 m (Engine driven) are Rs. 75,000 (Aprox. U.S. $ 2450) and Rs. 3,40,000 (Aprox. U.S. $ 11000). In India, the estimated users are 25 to 30, more than 50% of them are academic and R&D centres.

Compared to SPT and DCPT, CPT is not yet popular because, capital cost is high, limitation of penetration through bouldary strata, rods gets stuck in alluvial deposits, high cost, skilled operator essential and difficulties of anchoring. Cost of performing CPT is almost equal to exploration by a drill hole including field tests. CPT equipment based on Dutch sounding system has been developed in India and was first marketed by AIMIL, New Delhi. Later M/s. HEICO, Laurence and Mayo and many small industries are making/selling the equipment. Very few (4-5) may be imported equipment. 30 kN manual and 100 kN power drives models are more popular. Recently HEICO has developed 20 T engine driven model for P.S. Eng. (I) Pvt. Ltd., Madras. These equipments are developed as per IS - 4968 (part III).

As per IS, the observations at 0.2 m interval are recorded, corrected as above and tabulated. They are presented graphically in two graphs (depth vs q_c (kpa) and depth vs f_s(kpa)). It is a practice with some, to plot friction ratio (f_s/q_c x 100) versus depth graph. Presentation details of TC-16, ISSMFE could be incorporated to make the standard more precise.

3.4 National codes

IS : 4968 (part III) - 1976, is the only standard for CPT in INDIA. The major provisions are identical to TC-16 reference test as explained above.

4.0 INTERPRETATION OF TEST RESULTS

The interpretation of test is not uniform and the practice, from published literature is discussed here.

4.1 Zoning of area

The data of friction ratio (f_R), q_c and effective surcharge as plotted on standard scale are super imposed to deneate the area having similar ranges. In known geology vast area is first zoned by DCPT and reconfirmed by CPT. This helps in evolving specific exploration programs of drilling sampling and special field test for critical parameters. This contineous process has been found time saving, economical and more reliable.

4.2 Soil stratification

Based on analysis of plots (para 3.3) and super imposition, zonewise, a typical upper, lower bound ranges of q_c with depth are evolved. For the trends and ranges, classification is evolved for the strata. A soil stratification with probable classification is prepared as a model.

Para 4.2 with details of structures, enables us to tentatively fix the depth, type of foundation and very aproximate size. Thus influence zone is estimated.

Based on above analysis author has evolved economical exploration, saving time as well. The test zone is defined and critical sensitive parameters are identified to plan minimum drilling, sampling, and field test programmes. The conventional specifications for exploration generates a jungle of data leaving designers to be confused. The author's above approach provides more specific data of controlling parameters of design.

4.3 Soil parameters

IS code 2911, 1980 permits indirect evaluation of soil property, if direct reliable results from laboratory tests are not available.

4.3.1 Saturated cohesive soils

The point resistance (q_c) in kgf/cm^2 is considered as λ x C_u. The factor λ is reported to be variable.

Source	Type of Soil	λ
IS 6403 - 81	Normally consolidated $q_c < 20$ kgf/cm^2.	15 to 18
	Overconsolidated $q_c < 20$ kgf/cm^2.	22 to 26
Kasmalkar B. J. (1991)	Soft clays	14 to 16
	Over consolidated clays	24
Berengen et al (80)	Deep Bombay marine clays	15
Varadharajulu G. H. (1990)	0-13 m organic CI-CH calcutta	19
Kasmalskar (1991)	Stiff fissured dessicated clays	25 to 30
Surgunan (83)	Madras clays ($q_c > 120$)	17.5
Varadharajulu G. H. (1990)	Calcutta depth 13 to 20 m	25

J. Amar (74) found increase in q_c of soft clays by 40% with increased rate of pushing cone. Hajira report by KBM gives λ = 36 by vane and λ = 100 by UCC tests on UD samples. Thus λ varies between 15 to 25 depending on stiffness, rate of penetration, dessication. It varies considerably If C_u is evolved by insitu vane or UCC tests.

4.3.2 Noncohesive Soil

The table presented by H. Doscher (67) is :

State	R_D%	N_{SPT} blows/30 cm	q_c kgf/cm^2	$\phi°$
V. Loose	0-20	4	< 20	< 30
Loose	20-40	4-10	20-40	30-35
Compact	40-60	10-30	40-120	35-40
Dense	60-80	30-50	120-200	40-45
V. Dense	> 80	> 50	> 200	> 45

The correlation of Michel and Katti (81) gave :

q_c MPa	< 5	5-10	10-15	15-20	> 20
R_D%	< 15	15-35	35-65	65-85	> 85
$\phi°$	30	30-32	32-35	35-38	> 38

The standard chart after Robertson and Campanella (1983) is widely used to classify sands. Surgunan (83) stated that very loose Madras sands registered zero sleeve friction (f_s'). Friction ratio for Madras sands varied from 0 to 1.8 and from 1.62 to 3.65 for silty fine sands. Bombay's carbonate sand at depths (q_c > 60 MPa) recorded friction ratio 0.5 to 1%. (Berengen'80).

The ϕ predicted by Mayerhof's method was higher than values obtained in direct shear for Madras sand (Sagunan '83). Desai (68) found ϕ values 31°-33° for Tenughat sand (dam foundation) exhibiting q_c 20 to 40 kgf/cm^2. The relative density measured insitu was 30 to 42%.

The Fig. 1 presents correlation of q_c - p_o surcharge pressure and R_D. (Tolia D. S. 1994).

To evaluate E_s ($\cong 1/m_v$) at a stress 1.0 kgf/cm^2 surcharge in sands above water table, H. Doscher (67)'s correlation gave :

$p_o' = r \times t$ kgf/cm^2	q_c kg/cm^2		
	25	50	75
0.0	480	560	620
0.5	300	360	425
1.0	100	200	250

The IS 2950(I) 1981 on raft foundation recomonds :

a) For stratified deposits, CPT is recommended over plate load test to evaluate E_s.
b) q_c obtained in depth = 2 x width of raft at several points, is used to evaluate modulus of elasticity. For all soils, a generalized $E_s = 2q_c$ kgf/cm^2 could be used.

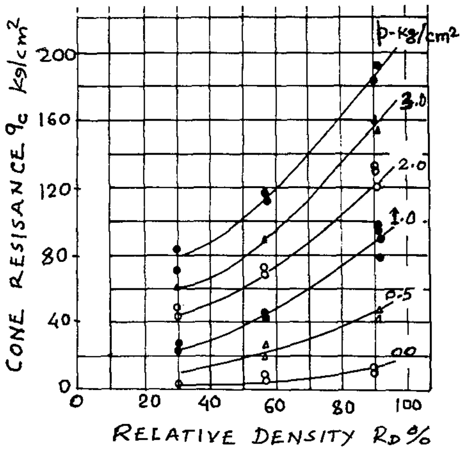

Fig. 1. Correlation between R_D and q_c at different over burden pressures (p) (Uniform coarse sand, Tolia '78).

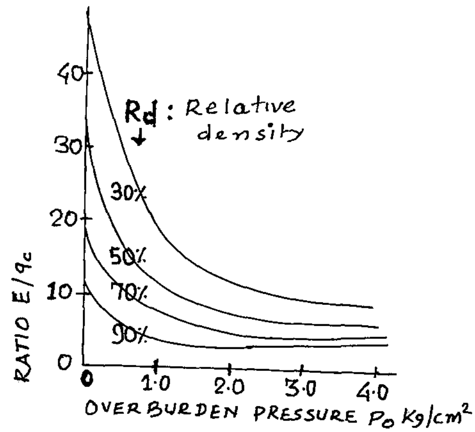

Fig. 2. Correlation of E/q_c and p at different R_D. (Tolia '94).

V. N. S. Murthy (1967) recommends E as $2q_c$, $3q_c$, $3.5q_c$, kg/cm^2 for relative densities of 0-35, 35-85, more than 85%. The correlation of E/q_c versus R_D for different overburden pressures (D. S. Tolia '94) is presented in Fig. 2.

Using $m_v = 1/(\alpha q_c)$ after Sanglerat ('72), α factor worked out for Madras clay by Surganan ('83) is as under :

Soil	q_c kg/cm^2	α
CL	45-60	1.37
	40-45	1.67
	25-40	1.7-2.8
CH, OH	5 to 25	2.8-6.1

These values are quite different from the range of 1 to 2.5 provided by Sanglerat for clays with q_c > 20 kgf/cm^2.

4.3.5 Water table

The depth to water table is not very clearly indicated by test results. It is well established that for same soil, q_c under water table is lower.

5.0 USE OF CPT IN GEOTECHNICAL DESIGN

5.1 Pile capacity

IS 2911 part I load bearing concrete piles are devided : section 1 - driven in cast insitu concrete piles, section 2 - bored cast insitu piles, section 3 - driven precast piles, and section - 4 bored precast piles.

Ultimate resistance of pile in grannular soil uses conventional formulae. The q_c around tip will be used to evaluate ϕ which will gave bearing capacity factors N_r, N_q are evaluated using relevent codes. Piles in cohesive soils is obtained by formulae. (IS 2911 part I, sec.1 and sec.4).

$Q_u = A_p N_c C_p + \alpha C A_s$,

$N_c = 9$, C_p is average cohesion at tip in kgf/cm^2 is obtained using, q_c , C average cohesion through the length of pile in kgf/cm^2 is obtained by using av q_c on length of pile.

For piles in noncohesive soil code recommendations to predict local side friction (f_s) is as under :

Silty clay and silty sands $q_c/100 < f_s < q_c/25$ kg/cm^2.

Sands $q_c/100 < f_s < 2\,q_c/100$ kg/cm^2.

Coarse sands and gravel $f_s < q_c/150$ kg/cm^2.

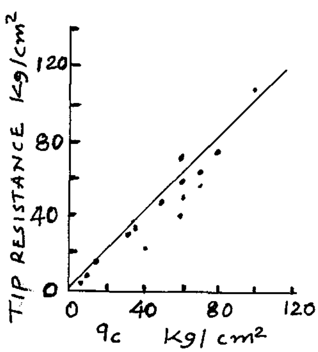

Fig. 3. Tip Resistance of pile vs q_c (D. Mohan '63).

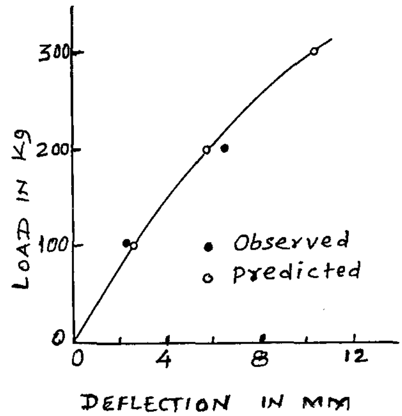

Fig. 4. Load deflection curve at a point 7.5 cm above ground level (U. Dayal '88).

For noncohesive soils ultimate bearing capacity may be computed by following relationship :

q_u = Ult. point bearing capacity

$= (1/2) [\{(q_{co} + q_{c1})/2\} + q_{c2}]$

q_{co} = av. q_c over depth 2d below tip of pile

q_{c1} = min q_c over depth 2d below tip of pile

q_{c2} = av. min q_c over length 8d above base.

d = dia. of pile.

For piles in cohesive soils C_p is predicted using q_c as discussed before. The shaft resistance (local side friction) is estimated by following

	Side friction (f_s)
clays & peats (q_c <10 kg/cm^2)	$q_c/30 < f_s < q_c/10$
clays (q_c >10 kg/cm^2)	$q_c/25 < f_s < 2q_c/25$

Fig. 3 correlates q_c with tip resistance of pile (D.Mohan et al '63). Skin friction is 2% of q_c for soils having q_c ranging from 10 to 100 kg/cm^2 (D.Mohan & Jain '63)

Surgunan et al (83) gave pile capacity as $q_c = C N_c + (\gamma \times D) \times f_s$ for clay and $q_c = (\gamma \times D) \times N_q$ for sand. Madras soils investigated by author gave N_c = 14.7 to 17 against liturature value of 10 for $q_c > 5$ and 18 for $q_c > 27$. N_q for two sites shows wide range 51-250 to 81-328 against recommended range of more than 355. The value of α for ultimate capacity for CL - ML Madras clays obtained by Surgunan is as under

q_c kg/cm^2	α	
45-60	1.37	CL
40-45	1.67	
25-40	1.7 to 2.8	
5-25	2.8 to 6.1	OH - MH

500 mmϕ 15 m deep pile in 0-12 m subsoil with av. q_c = 30 kg/cm^2 and 125 kg/ cm^2 at 13.5 depth recorded 230 t as ultimate load in load test. The IS code predicted ultimate load as 165-174 t.

Latterally loaded piles have been analysed for flexural response. U. Dayal et al (1988) confirmed use of CPT with Reese's Method is fairly accurate. The load deflection pattern observed and predicted using above data are shown in Fig. 4.

Agarwal K. B. (1983) reported 17 bored piles and 32 under reamed piles, pile load test at Paradeep, New Delhi, and Roorkee etc. The subsoil is clayey silt and depth of pile was 19.5 m. The q_c at 14 to 17 m was 25 kg/cm^2 and q_c = 50 kg/cm^2 at 19m. The pile load test at 12 mm settlement gave pile capacity of 90 t for 400 mmϕ pile. The authors confirms safe load criteria based on total settlement criteria of IS 2911 (IV). The safe load based on load test (shear criteria) being lower than obtained by settlement criteria the need to consider lowest is proposed.

U. Dayal's analysis of latterally loaded piles

Depth	p_o' Effective overburden kg/cm^2	q_c kg/cm^2	$q_P = q_c - p_o$'	R_D	ϕ	Modulus of subgrade kN/m^3
0.5	0.08	4	3.9	10	31.78	1927.8
1.2	0.20	8	7.8	20	32.50	3856.4
2.5	0.40	18	17.6	30	33.50	8684.0
4.0	0.66	32	31.4	40	34.50	12243.0

5.2 Settlement parameters

IS code 8009 (Part I) 1976 provides evaluation of settlement of cohesionless soils by semi-empirical approach (4.3.2) based on static cone or DCP or plate load test.

The q_c vs depth curve is splitted into layers of average q_c. The average q_c and thickness of the layer (H_t) is used to compute settlement of each layer by expression.

$$S_{f1} = 2.303\ (H_t/c)\ \log_{10}[(p_o+\Delta p)/p_o]$$

where C = 3/2 x (q_c/p_o), p_o, Δp are effective overburden in kg/cm^2 and change in stress at mid height of layer H_t. Total settlements of all layers in stress zone will be used as final settlement.

5.3 Bearing capacity

IS 6403 (1981) bearing capacity of shallow foundations provides a correlation of q_d/q_c for different B width and D_f/B ratio for cohesionless soils. q_d = net ult. bearing capacity for shear consideration (kg/cm^2), q_c is average for depth 2B below foundation level, minimum of no. of tests for a building.

For cohesive soils q_c/C_u correlation is used to obtain q_d.

The permissible bearing stress (q_p) for settlement criteria is given as :

a) Dry moist sand $q_p = q_c/30$, for $B \le 1$-2 m,
$= q_c/30\ [1+0.3/B]^2$, for $B > 1.2$ m

b) Submerged sand $q_p = q_c/40$
for raft foundation (Kasmalkar '91).

5.4 Design of control specifications

For ground improvement, control tests offer the treatment are specified in terms of minimum q_c. Typical results reported by S. D. Ramaswami et al (1980) for site in Bangla Desh is shown in Fig.5. The 20 hectare plot had 3 to 9 m thick hydraulic fill. The dynamic compaction was used for compaction.

Fig.5 showing efficiency of GRIMTECH in terms of cone resistance. Both degree of improvement and depth upto which the compaction was effective is brought out by test.

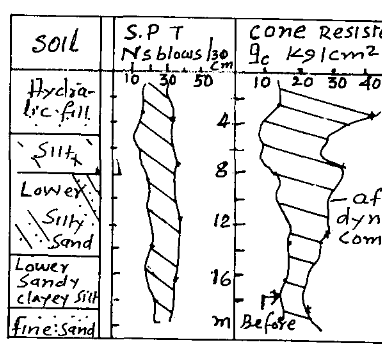

Fig. 5. Control of ground treatment by CPT and SPT (Ramaswami '80).

The plant site Manglore has 0-3.5 m med. sand (N_s - 5 to 15), 3.5-5 m silty sand (N_s - 2 to 10), 5-14 m med. to fine sand overlying stiffclay. Cemindia carried out vibrofloation to attain safe bearing capacity of 25 t/m^2. The specification for control was to attain q_c = 150 kg/cm^2 after treatment. (S. N. Krishnamurthy et al '83).

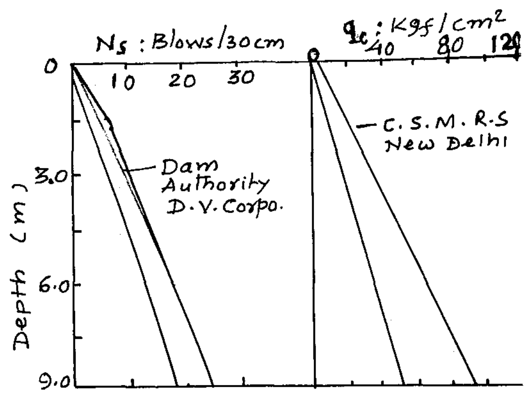

Fig. 6. Variation of N_s, q_c with depth, Tenughat dam foundation (Desai '68)

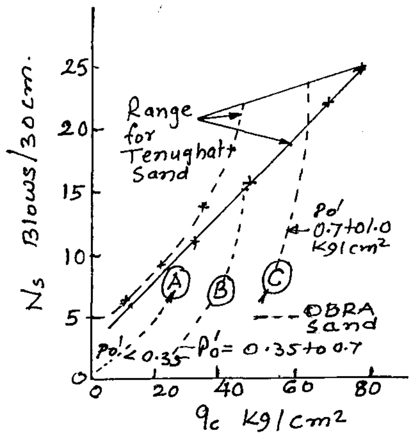

Fig. 7. Correlation of N_s, vs q_c for Obra Sand (Desai '68)

6.0 CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

6.1 Correlation with SPT (N_S)

Dinesh Mohan (1963) observed q_c/N_s = 4 for sand and q_c/N_s = 2 for silty clays for Paradeep, Calcutta soils.

Varadarajulu (1990) observed q_c/N_s = 1.5 to 1.6 of CH soil for calcutta soils.

Vyas M. J. (1995) observed increasing trend of q_c/N_s ratio with average grain size from 2 mm to 6 m mean size.

Desai M. D., et al, observed the relation effected by effective overburden pressure and type of soil, the N_s vs q_c (kg/cm^2) is given in Fig. 6 for Tenughat sand and Fig. 7 for Obra sand.

Naresh D. N. et al (1994) observed q_c/N_s = 1.983 for medium to dense sandy silt and dense to very dense silty sand.

Singh M (1988) reported a variation of 2.3 to 5.5 of q_c/N_s ratio.

The evaluation of Modulus of subgrade based on q_c was attempted by U. Dayal (1988). The observations are presented in above Table.

6.2 Correlation with DCPT (N_c) (eq. to DPH)

Desai M. D. (1968, 74) reported the relation q_c = k x N_c kg/cm^2, where k = 4 for fine sand, 8 for coarse sand, 10 for gravally sand, 2 for silty clay, 2.27 for silty fine sand (North India). This ratio varies with effective overburden pressure and is sensitive to water content.

6.3 Correlation with other methods

Naresh D. N., et al, (1994) reported relation between q_c , P_l and 'E' obtained by PLT (plate load) and pressarometer (P_l).

These results are retabulated below :

Soil Type	q_c kg/cm^2	q_c/P_l	Eq_c kg/cm^2	Eq_c /E_{PLT}
Loose silty sand	20-30	6.2	40-60	1.2
Med. Silty sand	30-40	5.0	60-80	0.7
Dense Silty sand	50-100	2.5	100-200	0.6
V. Dense Silty Sand	100-175	2.8	200-350	---

Varadarajulu (1990) reported q_c/V_u (undistarbed shear strength from vane test) equals 17.5 (average) with a wide scattered and f_s /V_u equals 0.9.

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CPT Testing in the Republic of Ireland

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SYNOPSIS: Geotechnical engineering investigation and design on the Republic of Ireland is dominated by the mantle of competent coarse glacial deposits which covers large areas of the country. Consequently CPT testing has not been considered as an appropriate tool for many investigations. However, deposits of soft alluvial material occur in low lying areas and along river flood plains. In these areas CPT testing has been used in investigations mostly associated with highway embankment design and construction. This paper presents the history and current state of CPT testing in the Republic of Ireland together with a summary of other common testing methods.

1. GEOLOGY & GEOTECHNICAL CONDITIONS IN IRELAND

The coastline of Ireland is formed of strong older igneous and metamorphic rocks such as sandstones, granites and quartzite. The central basin of Ireland is formed of carboniferous limestone. However the engineering geology of Ireland is dominated by the mantle of 10,000 of 15,000 year old glacial tills and glacial sands and gravels which cover most of the country. These deposits generally have a high gravel, cobble and boulder content and are usually medium dense to dense or stiff to very stiff in consistency. Later alluvial activity has covered the glacial deposits with soft clays, silts and peats in some low lying areas and along river flood plains.

2. COMMON INVESTIGATION METHODS IN IRELAND

2.1 Standard Penetration Testing

By far the most common technique of investigation for superficial deposits in Ireland is by drilling shell and auger boreholes, taking disturbed and undisturbed samples for identification and laboratory testing and carrying out in-situ standard penetration tests (SPT) in the boreholes.

2.2 Dynamic Cone Penetration Testing

The methods described in Section 2.1 have not proved entirely satisfactory in the glacial soils mostly due to the inability to take undisturbed samples and the concern that SPT tests, at intervals in a borehole, were not picking up weak zones. In the 1970's the Irish Science & Technology Agency, Forbairt, began to develop and promote the use of dynamic cone penetration testing (DCPT). They choose the Swedish Borros Automatic Ram Sounding machine. The equipment uses a 63 kg hammer, falling 500 mm and a 45 mm diameter cone with a 90° apex angle and a mantle length of 90 mm. This equipment is similar to that specified in British Standard BS 1377 : Part 9 : 1990. The number of blows is recorded for each 200 mm of penetration (N_b) and a correction is made for rod friction.

Correlations have been developed between N_b and SPT N, undrained shear strength and CBR value. The DCPT testing is also used for assessment of ground conditions prior to piling and as a control in ground treatment contracts. The experience with DCPT in Ireland has been described by McGrath et al (1989).

2.3 Other Methods

Other common methods of investigation of superficial deposits in Ireland include the use of hand held dynamic probing equipment, such as the "Macintosh Probe", light drilling equipment such as the "Cobra Drill" or "Minute-Man", hand augers and of course trial pitting.

3. CPT EQUIPMENT USED IN IRELAND

3.1 Equipment

CPT equipment was first used in Ireland in the later 1970's when Site Investigation Limited of Newcastle, Co Dublin purchased 2.5 tonne mechanical (Begemann) CPT equipment. The cone diameter is 35.7 mm with a semi-angle of 30°. A penetration rate of 2 cm/sec. is used. The equipment was mostly used in the investigation and design of embankments for highways and for storage lagoons in industrial projects.

In 1986, the Soil Mechanics Laboratory in University College Galway (UCG) purchased an electric piezocone from GMF (Holland).

Their piezocones have a 60° apex angle, a cross-sectional area of 10 cm^2 and are pushed into the ground at a rate of 2 cm/s. The piezocone was rated for a maximum load of 20 kN. The pore pressure sensor is located immediately behind the cone. Data recorded during the test, every 100 mm, are recorded on a paper tape printer for later input into a micro-computer for interpretation. Some of their experience is described by Rodgers (1989 & 1993), Rodgers and Joyce (1994) and Rodgers and Fahy (1995). Computer logging facilities are also available for checking purposes. The UCG piezocone is used for penetrating soft soils only and is not used in the medium dense/stiff glacial deposits.

Fugro Limited of Holland have also been occasionally employed by the major Irish site investigation contractors as CPT Sub-contractors. Fugro use equipment similar to that of UCG. Fugro are usually prepared to penetrate the upper layers of the glacial material as well as the alluvium. Data is usually recorded electronically.

3.2 Market for CPT Testing

In general the market for CPT testing in Ireland is very limited. The mechanical equipment owned by Site Investigation Limited is no longer used. UCG have carried out approximately 35 projects since 1986 mostly associated with new roads or road improvement schemes. A typical UCG project would comprise 15 to 20 piezocone tests in association with in-situ vane testing and laboratory tests. Fugro have typically carried out 1 significant project (say 40 No. 10 m deep CPT tests) per year in recent years.

3.3 Test Procedure

The test procedure uses the standard technique of pushing 1 m rods into the ground at a rate of 2 cm/s. The results for cone tip resistance are corrected in the usual way by subtracting overburden effects. The results are presented in the form of cone tip resistance, soil/sleeve friction and pore water pressure versus depth. An interpretation of the soil type is usually presented as standard with the results. The piezocones used by UCG are calibrated in a triaxial cell, before each use.

In Ireland CPT testing is usually carried out under the guidelines given in British Standard BS 1377 : Part 9 : 1990.

4. INTERPRETATION OF TEST RESULTS

4.1 Soil Classification and Stratigraphy

Standard methods such as those presented by Meigh (1987) and Robertson and Campanella (1983) and used in classifying soil.

4.2 Soil Parameters

CPT test results have mostly been used to provide estimates of undrained shear strength. Estimates of modulus of deformation, co-efficient of radial consolidation and sensitivity have also been obtained from CPT test results. Local correlation have occasionally been developed for N^1_{kt} and N_{kt} the relationship between effective and total cone tip resistance and in-situ vane shear strength. O'Riordan et al (1982) found that unique values of N^1_{kt} and N_{kt} could not be established for very soft alluvial deposits at Athlone, Galway and Belfast (c_u approx. 5 kN/m², tip resistance less than 300 kN/m²). They explained the wide scatter in N^1_{kt} and N_{kt} by the cumulative efffects of low strength, low plasticity, hgih liquidity index and strain rate on the different modes of failure measured in the vane and CPT tests.

The co-efficient of consolidation determined from CPT tests have not generally been used in analysis as it is felt that the CPT test gives a value at one vertical effective stress only.

4.3 Environmental Data

The Fugro "BAT" groundwater monitoring system has been used on one site in Ireland to extract groundwater samples from contaminated land.

5. USE OF CPT IN GEOTECHNICAL DESIGN

5.1 Foundation Design

In general, in Ireland, no specific design mehtods have been developed which make direct use of CPT test results in foundation design. The CPT test results are converted to SPT N, shear strength or a similar parameter to enable the design to be carried out using well established local correlations.

5.2 Embankment Design

CPT tests have been used to determine parameters such as compression modulus and co-efficient of consolidation in embankment design. The CPT test results would generally be used as a back-up to data from laboratory testing.

5.3 Liquefaction Assessment

The work of some Japanese researchers and the CPT test results have been used to determine the liquefaction potential of some loose silts beneath hydro-electric storage embankments.

6. CORRELATION OF CPT WITH OTHER METHODS

Efforts to correlate CPT results with other methods have largely been confined to N^1_{kt} and N_{kt} values, see Section 4.2. N_{kt} values increase with plasticity index from 11 to 15 at PI = 0% to 17 to 21 at PI = 50%. N^1_{kt} values vary between 6 and 12 (Rodgers, 1989, 1993).

7. RESEARCH ACTIVITIES

In view of the small market in Ireland, research activities into the use of CPT have been limited. Research students at UCG have included a study of CPT results as part of their projects, see for example Rodgers and Joyce (1994) and Rodgers and Fahy (1995).

8. FUTURE TRENDS

Due to the development of most powerful and efficient excavating machines embankments on soft ground are now frequently being constructed by excavating the soft material and replacing it with rockfill. Attempts are also made to ensure that the route of a new road avoids areas of soft ground.

Consequently the requirements for CPT testing in Ireland have been reducing in the past number of years. However, the CPT is considered to be a valuable and accurate tool which provides very accurate soil profiling, a significant amount of soils data and therefore it will continue to be used in major projects in areas of soft ground.

The development of contaminated former industrial sites in towns and cities in Ireland is increasing and there may be some further requirments for CPT testing which provides environmental data. To this end, UCG have recently purchased a conductivity cone which they hope to employ in contaminated land sites.

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Cone penetration testing in Italy

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SYNOPSIS: This paper briefly summarizes the Italian current state of practice in the use of static cone penetration testing. After a short description of the complex geological features of the country, a comparison is made among the relative diffusion and use of the various penetration testing techniques over the nation. The comparison shows that dynamic (SPT) and static (CPT) tests are still the most widely used techniques for site investigation and that the incorporation of new measuring devices and sensors is rather far from gaining popularity, at least in common practice. Cone penetration tests with pore pressure measurements are shown to be the most efficient tool for soil profiling and detection of macrostructure, the latter representing a quite common problem in Italian natural deposits. Most of research work is directed towards the interpretation of penetration tests to provide the mechanical and flow parameters of the tested soils. This research tendency is reflected in design practice, where indirect methods tend to prevail over direct correlations with the performance of structures. A look at the near future shows new trends and developments for research, such as the envirocone, the abyssal cone and other multi-purpose probes for both traditional and environmental investigations.

1. GEOLOGY

Italy is located in the southern part of the European Continent, in a position that distinctly characterizes the geography of the Mediterranean basin. The peninsula extends South-Eastward for about a thousand kilometers, and includes the two main islands of the Mediterranean Sea: Sicily and Sardinia.

From the geological point of view, the Mediterranean area is young and very complex; actually the zone is the seat of the so-called "Adria micro-plate", around which the three main plate borders of Asia, Europe and Africa interact. These borders, deformed during the Alpine-Himalayan orogenesy, developed in the Cretaceous period and are still active.

During the orogenetic process, other important events took place such as the Plio-Pleistocenic marine transgression and the intense endogenous activity. The first one occurred in recent geological time and led to the deposition of fine marine sediments in shallow areas, while the second one caused volcanism nowadays still active. As soon as the first uplift of the Alpine and Appennines chains started, the erosion process started as well, and during the Quaternary period the geomorphological modelling of the territory took place. These geological process produced a territory that mainly consists of mountains and hills, while flat land areas (Fig. 1) cover only 20÷25% of the whole territory. The flat land areas are mainly located along the Adriatic Sea, the most important being the Padana Valley.

The Italian geological features allow a wide use of penetration testing both on-shore and off-shore , especially in the alluvial flat lands and hills in the pedimountain areas, where thick layers of medium grained and fine deposits are encountered.

2. USE OF CPT AND OTHER PENETRATION TESTING METHODS IN ITALY

In situ penetration tests represent a fundamental part of almost any site investigation program in Italy. From a recent survey through private and public consulting groups and firms, the most common penetration tests and investigation methods are summarized in Table 1.

Table 1. Summary of use and general features of in-situ testing in Italy.

Comparison basis	SPT	CPT-M CPT-E CPT-U	VT	DMT	PMT SBPT	PLT
Simplicity of the equipment	simple	relatively simple	simple	simple	complex	simple
Testing execution	easy	relatively easy	easy	easy	complex	easy
Availability on the market	excellent	good	good	fair	poor	excellent
Present use	routine	routine to moderate	routine	routine	limited	routine
Historical use	substantial	moderate to substantial	moderate	moderate	limited	substantial
Unit cost	*medium*	low to medium	*medium*	low to medium	*high*	*low*

Reasons associated with the ground geomorphology as well as historical use, availability and ease of testing render the dynamic penetration tests (SPT) and plate loading tests (PLT) the most utilized tests, in site investigation programs.

Although pressuremetric technology is available with the Menard pressuremeter (PMT) and the self-boring pressuremeter (SBPT), applications are somewhat limited due to the complexity of execution of the test as well as to reasons connected with historical use and cost.

Vane (VT) and dilatometer (DMT) tests are locally used through the Peninsula. The remarkable contribution of basic and interpretative research work so far performed

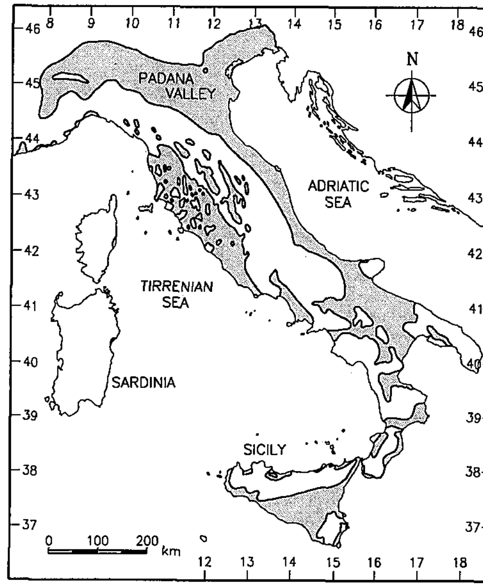

Figure 1. Location of main Pliocene and Quaternary deposits in Italy.

allows a routine use of the dilatometer in both cohesive and cohesionless deposits (Marchetti et al., 1991).

Referring to the "CPT family" (Fig. 2), the most popular and widely used is still the mechanical cone (CPT-M). Following the new trends and the international standard, an increasing number of electric (CPT-E) and piezo (CPTU) cones are available. They are equipped with inclinometer and the filter of the piezocone is located at the cone base.

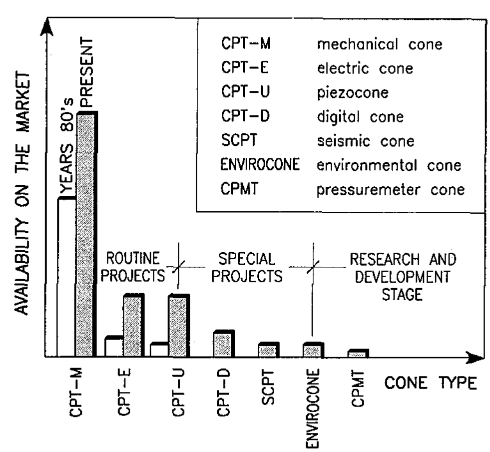

Figure 2. The CPT family.

However, only a few institutions can cover almost all the state of the art in terms of cone types and equipment for on-shore and off-shore services.

The "CPT family" also includes the seismic cone (SCPT) (Brignoli et al., 1995). The main advantage is its ability to investigate the soil at small and high strain. The relatively low cost will certainly concur, in the near future, to a wider application in special and routine site investigation programs.

The envirocone recently developed by ISMES consists of a special cone probe for polluted sites assessment (O'Neill et al., 1995). In addition to point resistance, excess pore pressure records and pore fluid temperature, it allows to take groundwater, gas, soil samples and to carry out dissipation and hydraulic conductivity tests.

Finally, the cone pressuremeter device (CPMT) (Ghionna et al., 1995), still in the research and development stage, will be introduced into the market in the near future integrating the well known performances of the pressuremeter.

Generally rigs are built for penetration testing purpose, but in some cases the hydraulic of auger boring rigs are used with the recently introduced digital cone. The capacity of cone rigs ranges from 15 kN to 200 kN, even if the rig capable to operate in the different Italian deposits is the one mounted on a truck with 100 kN of allowable thrust.

Nowadays, for most mechanical cones, the load transmitted to the rods are measured electrically and fed into a signal amplifier/conditioner unit for data processing and plotting directly on the site.

Although the companies distributing cone equipment supply well defined and detailed calibration and maintenance recommendations, there is a need for a national standardization of the testing procedures.

3. BASIC CHARACTERIZATION OF NATURAL DEPOSITS

3.1. Soil profile and classification

The geotechnical literature abounds with classification criteria and charts based on the values of point resistance (q_c), sleeve friction (f_s) and of the friction ratio FR = f_s/q_c. To date, the success and practical use of such correlations in Italy has been rather limited, probably for the following main reasons:

- the measurement of f_s is less reliable and repeatable than the measurement of q_c;
- the values of q_c and f_s are measured at different locations, spaced 20÷25 cm apart. This may induce substantial errors for deposits characterized by pronounced macrostructure and thin soil layering, which represent the majority of Italian deposits.

These as well as other observations have led to the appreciation that the FR-based classification criteria cannot be considered as substitute of a well executed borehole, and that a local calibration of the classification system is inevitably required (Rippa and Vinale, 1982; Pelli and Ottaviani, 1994).

On the other hand, it is believed that the simultaneous measurement of q_c and pore pressure (u) typical of CPTU may allow a straightforward detection of soil profiling. This is evidentiated from the results obtained at the Porto Tolle site and reported in Fig. 3, where the sudden drops in the pore pressure values recorded below a depth of 10 m reveal the presence of thin sand layers, a few centimeters in thickness, in soft clay deposit. Such macrostructural features of the deposit are much less evidentiated by the point resistance values, and totally unidentified by the lateral resistance values. A further evidence of soil profiling capability is furnished by the CPTU recently performed for the leaning Tower of Pisa. These have revealed that the upper layer of the subsoils, which extends to a depth of 7-8 m from the Tower foundation and overlies a stratum of normally consolidated clay, is much more cohesive than had been considered in the past.

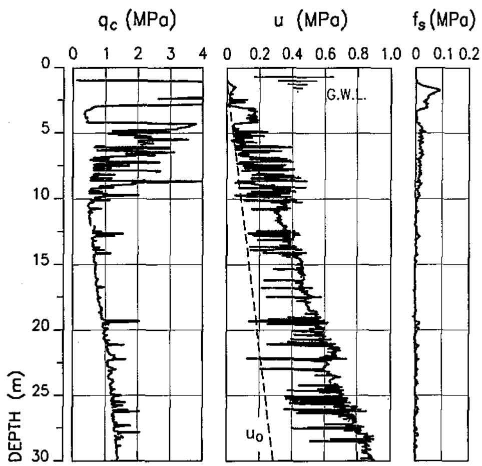

Figure 3. Results of CPTU in cohesive formation with highly developed macrostructure, Porto Tolle (Bruzzi & Battaglio, 1987).

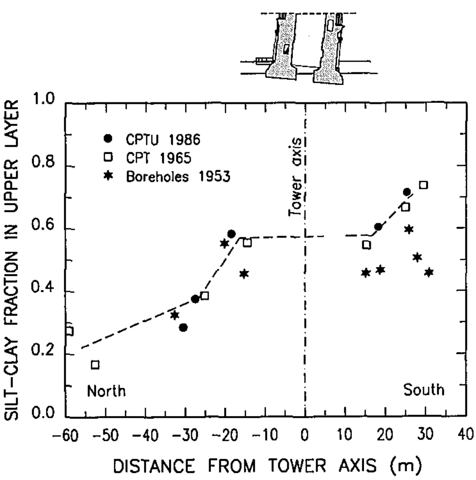

Figure 4. Estimated fines fraction in the upper layer of subsoils, Tower of Pisa (Calabresi et al., 1993, from data by Lancellotta & Pepe, 1990).

Furthermore, as shown in Fig. 4, the cohesive fraction markedly increases along the plane of maximum tower inclination (North-South plane). These findings, as well as a recent analysis of the kinematics of the Tower (Burland and Viggiani, 1994), indicate that such superficial soils might have an important role in the overall Tower behaviour.

Ten years after Jamiolkowski et al. (1985) statement, we can still affirm that CPTU represent nowadays the major tool for detecting layering, macrostructure and drainage boundaries. It is hoped that this technique will gain more popularity in common practice.

3.2. Cohesionless deposits

In the last twenty years a comprehensive series of field and calibration chamber (CC) cone penetration tests have been performed with the main purpose of validating and improving the existing correlations between q_c and the geotechnical parameters of sands (Jamiolkowski et al., 1988). The national CC testing program was carried out by two major research centers, namely, ENEL-CRIS and ISMES.

For the characterization of cohesionless soil behaviour it is necessary to identify: initial stress state and stress history, relative density, strength and deformability.

Unfortunately, most existing penetration tests in drained conditions can only provide two (CPT, DMT) or three (CPMT, SCPT) independent measurements. Therefore the only possibility for a comprehensive geotechnical characterization of natural cohesionless deposits remains to restrict CPT correlations to specific groups of similar soils (Jamiolkowski et al., 1988).

Unless explicitly noted, the relationships reported in the following have been validated for uncemented, unaged, uncrushable to moderately crushable, pluvially deposited silica sands. Extrapolations of these correlations to other types of cohesionless materials are not always possible and should be exercised with great caution.

a) State Parameters

As pointed out by Baldi et al. (1986), the initial relative density (D_R) is the parameter that mainly influences the penetration resistance q_c; stress state (σ'_{ho} and σ'_{vo}) and stiffness follow in order of decreasing importance. It seems therefore logical, for a given sand, to seek relationships between q_c and appropriate combinations of D_R and mean effective stress p'_o , denoted as state parameters. One of such relationships for 8 silica sands is shown in Fig. 5, using the state parameters v_λ and ψ respectively suggested by Schofield & Wroth (1968) and by Been and Jefferies (1985).

b) Relative density

Taking into account the unavoidable restrictions to specific soil types, the fitting of CPT carried out in the CC, under controlled boundary conditions, allows the establishment of reliable q_c-D_R correlations within the context of the fundamental link between q_c and state parameters.

Figure 6 shows the q_c-D_R-p'_o relationship obtained from 228 CC tests performed on both NC and OC Ticino sand specimens. As suggested by Skempton (1986), the use of such correlations for aged sands can lead to an overestimation of D_R. On the other hand, the same correlations will lead to an underestimation of D_R if applied to sands more crushable and compressible than those used in CC research, or to sands with more than 5-10% fines content. The effect of compressibility is evident in Fig. 7, which summarizes the result of 144 CC tests performed on 5 NC sands. The Figure also allows to appreciate the uncertainties involved in the evaluation of D_R from q_c measurements.

In order to compare the assessments of D_R in CC with the one associated to natural deposits, a series of static and dynamic penetration tests have been performed in a Quaternary deposit of Po river silica sand (Fig. 8). The values of D_R obtained by different penetration tests and correlations show a scatter of the order of 10%÷15%, that is, comparable to the one shown in Figs. 5 and 7.

c) Initial stress state and stress history

Due to the difficulties of undisturbed sampling, as well as to the substantial independence of density and strength from stress history, the only possibility for assessing the preconsolidation pressure of

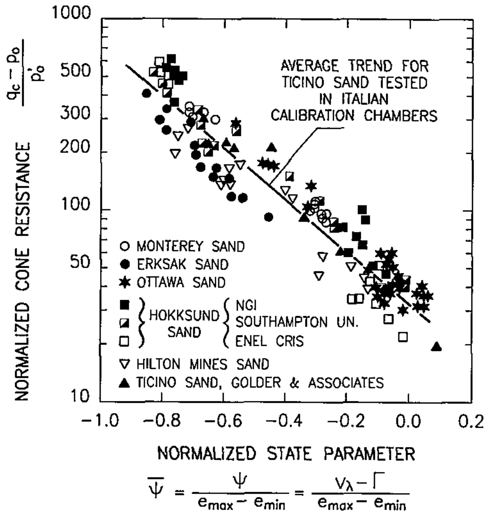

Figure 5. Cone resistance vs. state parameters from CC tests (Jamiolkowski et al., 1988).

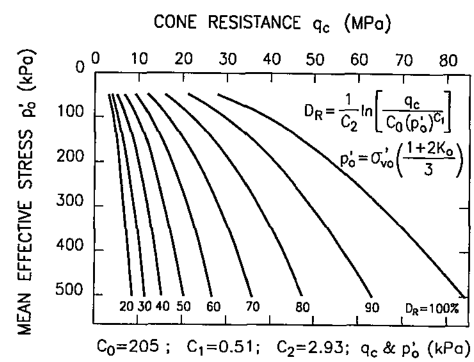

Figure 6. D_R versus q_c for NC and OC Ticino sand (Jamiolkowski et al., 1988).

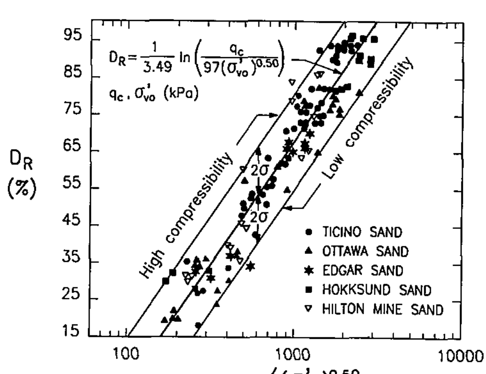

Figure 7. Correlation between D_R and q_c through σ'_{vo} for NC silica sands (Lancellotta, 1987).

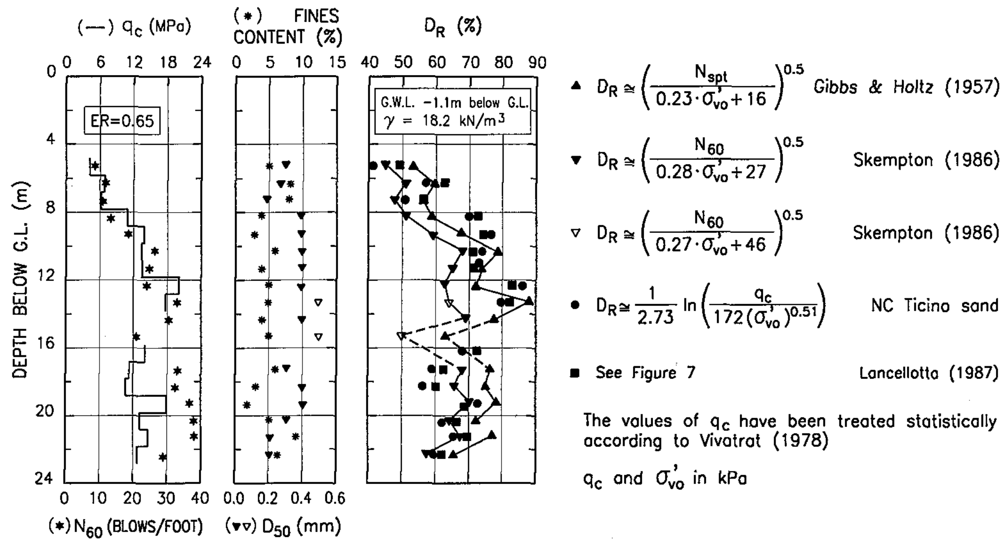

Figure 8. D_R of Po river sand from correlations with SPT and CPT (Jamiolkowski et al., 1988).

cohesionless natural deposits goes through reconstructions of the geological history and/or correlations with the initial horizontal in situ stress σ'_{ho}. Unfortunately, despite of the broadly recognized importance of σ'_{ho} (e.g. D_R determination once assessed the state parameters in OC sands), a fully reliable procedure for its determination in cohesionless deposits has not been yet established.

It has been suggested (Jamiolkowski et al., 1988) that the combination of point resistance from CPT with independent measurements from other kinds of field tests (e.g. PMT, DMT, CPMT, lateral stress cone) may allow the assessment of σ'_{ho} for a given sand, and various relationships have been proposed among σ'_{ho}, q_c, and K_D by DMT or K_{CPT} by lateral stress cone (Jamiolkowski & Robertson, 1988). Based on the results of CC tests on Ticino sand, Manassero (1991, 1994a) has recently suggested the following relationship among K_0 , q_c and the cylindrical cavity limit pressure p_u extrapolated from SBPT:

$$A \cdot K_0 \left[\frac{3(q_c - u_o)}{\sigma'_{vo}(1 + 2K_0)}\right]^B = \frac{p_u - u_o}{\sigma'_{vo}} \qquad (1)$$

where K_0 is the coefficient of earth pressure at rest and A, B are fitting coefficients (A=3.06; B=0.358).

Figure 9a shows a fair agreement between the values of σ'_{ho} calculated from eq. (1) and those applied to the CC specimens. In addition, the equation seems to give reasonable results when applied to natural deposits of silica sand, as shown in Fig. 9b for two sites in Northern Italy.

d) Shear strength

The direct assessment of the peak friction angle (φ') from q_c in cohesionless soils can be carried out referring to the following approaches (Jamiolkowski et al., 1988): (1) bearing capacity theories (rigid-perfectly plastic body); (2) cavity expansion theory (elastic-perfectly plastic body); (3) empirical correlations with stress dilatancy theories.

Due to the highly non-linear stress strain behaviour, softening phenomena, curved failure envelope, as well as to the difficulties of assessing the value of operational confining stresses around the penetrating cone, the use of methods (1) and (2) is not immediate.

A possible alternative approach is shown in Fig. 10: for a given boundary value problem, the estimate of the initial D_R allows to use Bolton (1986) stress dilatancy theory to obtain the peak friction angle φ'_o at a reference value of the normal effective stress on the failure plane (σ'_{ff}=267 kPa). Other values of φ' at different σ'_{ff} levels, may then be estimated using Baligh (1975) equation of the curved failure envelope, reported in the same figure.

e) Drained stiffness

The stiffness of cohesionless soils at medium to large strains is markedly influenced by aging and stress-strain history. Since the latter is largely obliterated by the insertion of the penetrometer probe, no unique correlation can exist between penetration resistance and soil stiffness. This is clearly shown by the data collected by Berardi et al. (1991) from CC tests, field trials and other in situ tests, and reported in Fig. 11; here the ratio between secant drained modulus (E') at an average strain level of 0.1% and q_c is seen to vary considerably with the age and stress history of the deposit. Recommended values of E'/q_c for calculations of settlements of shallow foundations in silica sands are given in Table 2.

Table 2. Recommended E'/q_c values for silica sands

Type of deposit	Reference secant modulus (related strain)	E'/q_c
man-made fill very recent NC	$E'(\varepsilon_a$=0.25%)	2.0 to 3.5
NC aged	$E'(\varepsilon_a$=0.10%)	3.5 to 6.0
OC	$E'(\varepsilon_a$=0.10%)	6.0 to 12.0
For a given soil E'/q_c decreases with increasing D_R		

On the other hand, the experience gained so far indicates that q_c can be correlated in a fairly reliable manner to the soil stiffness at

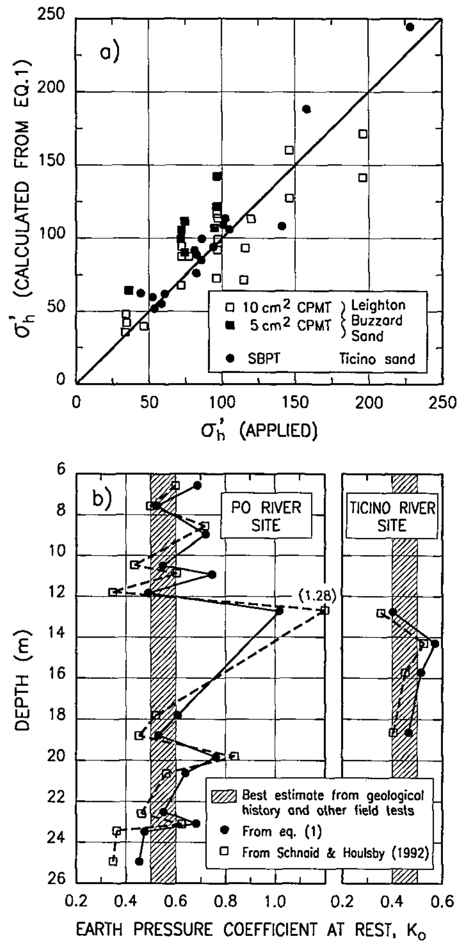

Figure 9. a) Estimated and applied horizontal stress in CC tests; b) evaluation of K_o for two Italian natural deposits; (Manassero, 1994a).

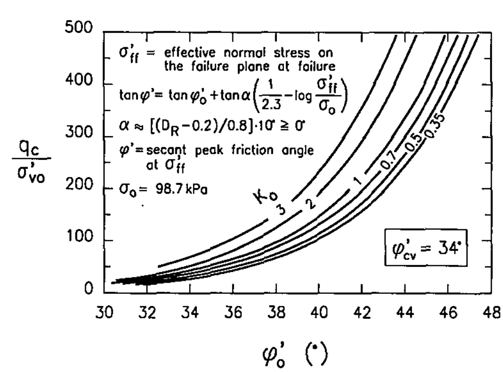

Figure 10. Angle of shearing resistance φ'_o using Bolton (1986) stress-dilatancy theory (Jamiolkowski et al. 1988)

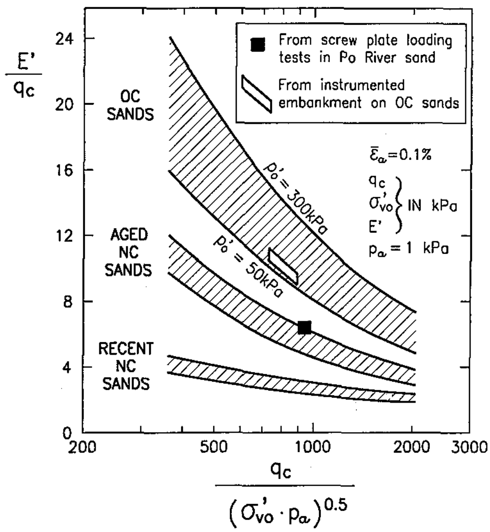

Figure 11. Evaluation of drained Young's modulus from CPT on silica sands (Berardi et al., 1991).

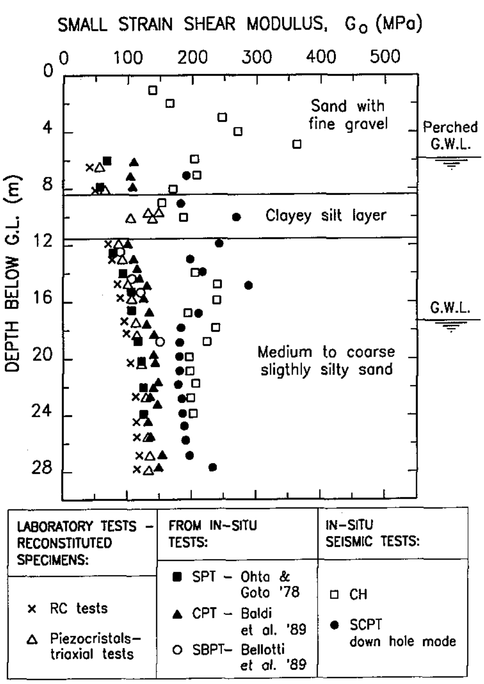

Figure 12. Small strain shear modulus of Ticino sand at Pavia University site (Ghionna & Jamiolkowski, 1991).

small strains (G_0 or E_0), at least for the uncemented silica sands of some Italian deposits along the Po and Ticino rivers (Jamiolkowski et al., 1988). However, it must be emphasized the important role played by the soil structure on the small strain stiffness, due to phenomena such as aging, diagenesis and cementation. This is evidentiated by the profiles of the small strain shear modulus below the groundwater table (not affected by capillary tensions) plotted in Fig. 12. Only the seismic tests (see in particular SCPT results) can reliably assess the G_0 values of the deposit, whereas the laboratory tests on reconstituted samples and correlations with SPT, CPT, SBPT furnish lower stiffnesses, associated with unaged, non-structured sand.

3.3. Cohesive deposits

Admittedly, the contribution of the Italian scientific community towards the understanding of undrained penetration in cohesive soils has been somehow more limited, with respect to drained penetration in cohesionless soils. Nevertheless, significant cone penetration (CPT, CPTU, SCPT) data has been gained for the characterization of several cohesive Italian deposits ranging from normally-consolidated to heavily overconsolidated conditions, and integrated with the results of other field and laboratory experimental techniques.

a) Stress history

One of the major challenges of penetration testing is the assessment of the preconsolidation stress or overconsolidation ratio (OCR) from the simultaneous measurement of pore pressure and tip resistance. Unfortunately, due to the complexity of the undrained penetration process, the problem has been so far faced through purely empirical approaches which relate the OCR to the piezocone pore pressure ratio, B_q. Among the various expressions proposed for B_q, it is worthed to cite the one suggested by Lancellotta (1987):

$$B_q = (q_c - \sigma_{vo} - \Delta u) / \sigma'_{vo} \qquad (2)$$

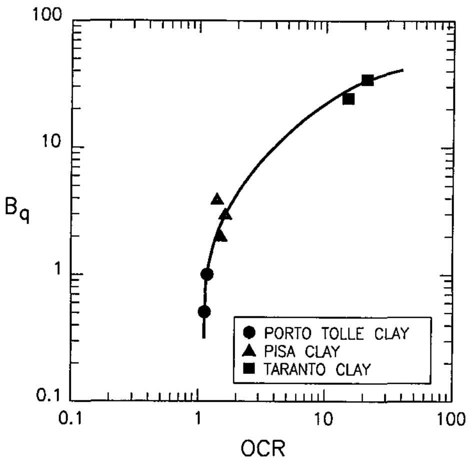

Figure 13. Pore pressure ratio versus OCR for three Italian clays (Lancellotta, 1987).

which refers to the concept of undrained normalization. Fig. 13 shows the variation of the above pore pressure ratio with OCR, for three Italian clays of different stress-history; although the approach seems to be promising, it must be emphasized (Jamiolkowski et al., 1985) that the measured pore pressure response also depends on other factors such as preconsolidation mechanism, soil type and sensitivity, local heterogeneity. Hence, at present, no B_q-OCR relationship can be intended as a substitute of the preconsolidation pressure obtained from laboratory oedometer tests.

b) Undrained shear strength

Despite of the different approaches so far proposed (Meyerhof, 1961; Vesic, 1972; Baligh, 1985), the theoretical analyses developed for an assessment of the undrained shear strength (c_u) are all based on oversimplifying assumptions. In addition, in practical problems the selection of a proper reference strength leads to uncertainties which are comparable to the ones deriving from the use of empirical approaches based on non-dimensional bearing capacity factors, N_c (Battaglio et al., 1986). As a consequence, in Italy the everyday procedure for determining c_u from CPT is the empirical one:

$$c_u = (q_c - \sigma_{vo})/N_c \qquad (3)$$

where the factor N_c is usually " calibrated" by means of field vane or unconsolidated-undrained triaxial tests.

Many Italian cohesive soils exhibit a significant structure. Fig. 14 shows the profile of c_u obtained by means of field and

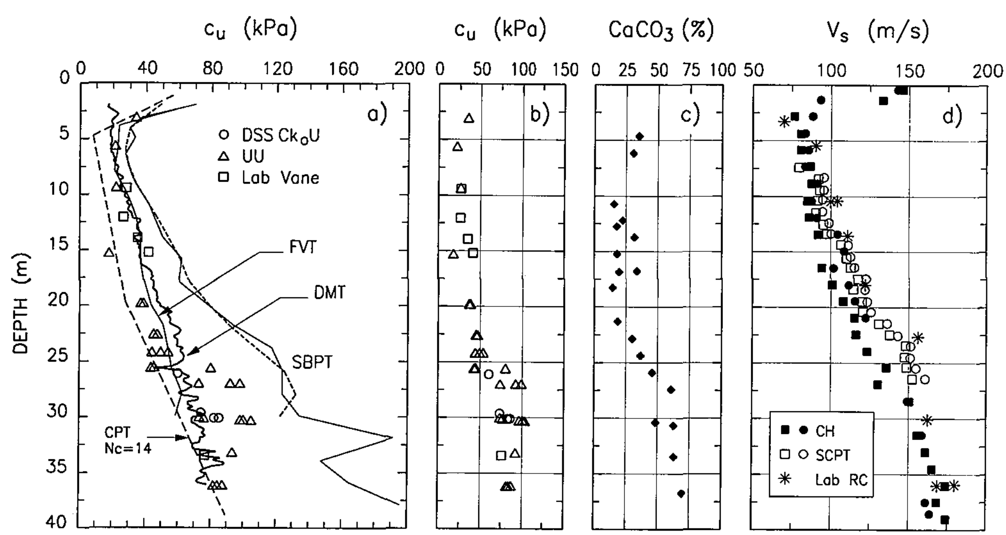

Figure 14. Profiles of c_u and shear wave velocity from field and laboratory tests on Fucino clay (A.G.I., 1991).

laboratory test on the soft, homogeneous, highly structured, $CaCO_3$ cemented Fucino lacustrine clay. It is noted that:

- with reference to the results of the various field tests, below the upper desiccated crust there is a fair agreement between c_u values resulting from FVT, DMT and CPT if the latter are employed with $N_c=14$. This value is in substantial agreement with the mean one indicated in the literature for normally consolidated clayey soils (Battaglio et al., 1986). On the other hand, the SBPT lead to values of c_u significantly higher than those resulting from other in situ tests;
- with reference to the laboratory test results, the values of c_u measured by means of TX-UU and DSS-CKoU tests show a marked increase below a depth of 20 m, reflecting the shape of the $CaCO_3$ profile. This feature is not displayed by the results of the field tests, with the exception of SBPT.

The above observations suggest that in highly structured soft clay deposits like Fucino clay the results of many field test (e.g., CPT, DMT and to some extent FVT) may be hampered by the destructuration of the tested

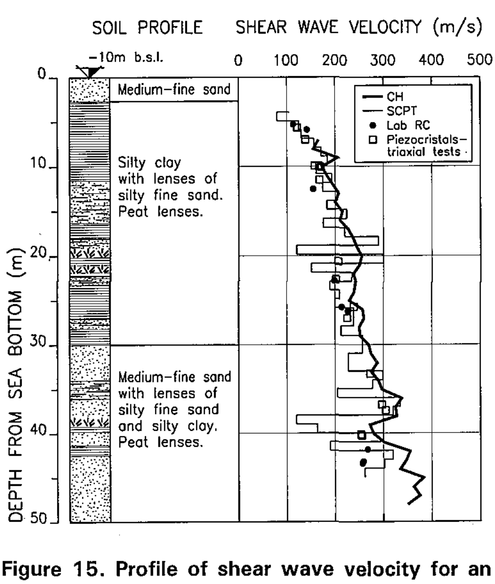

Figure 15. Profile of shear wave velocity for an off-shore Italian site (Brignoli et al., 1995).

soil, and lead to an underestimate of c_u. Care must therefore be devoted to the interpretation of field tests based on empirical correlations which have been validated in less structured materials (AGI, 1991).

c) Undrained stiffness

The estimate of an operational undrained stiffness from CPT is believed to be affected by the same ambiguity and uncertainties previously mentioned for drained penetration in cohesionless soils. On the other hand, the successful measurements of the shear wave velocity and maximum shear modulus from SCPT evidentiated for cohesionless deposits seems to hold also for cohesive deposits. As an example, in Fig. 14d) the values of v_s measured on Fucino clay by means of the seismic cone (SCPT) are compared to those obtained by means of field cross-hole tests (CH) and laboratory resonant column tests (RC). The comparison is deemed to be good, despite of the inherent differences of both experimental procedures and interpretation criteria. In addition, both in situ and laboratory tests reveal an increase of v_s below a depth of 20 m, i.e., appear to be influenced by the $CaCO_3$ cementation.

Additional evidence of the SCPT capability is given in Fig. 15 for an offshore silty-clay deposit of medium consistency. The Down Hole procedure of the seismic cone (SCPT) seems to lead to a more detailed profile than the one obtained by the well accepted Cross Hole method. Thin layers of softer soil can be more clearly identified due to the relatively small distance of the two receivers located in the cone shaft.

d) Flow and consolidation

Another attractive use of CPTU is the measurement of the excess pore pressure dissipation with time, in order to provide an estimate of the coefficient of consolidation in the horizontal direction, c_h (Torstensson, 1977; Randolph and Wroth, 1979; Baligh and Levadoux, 1986). Fig. 16 shows the results of several dissipation test in the hard, heavily

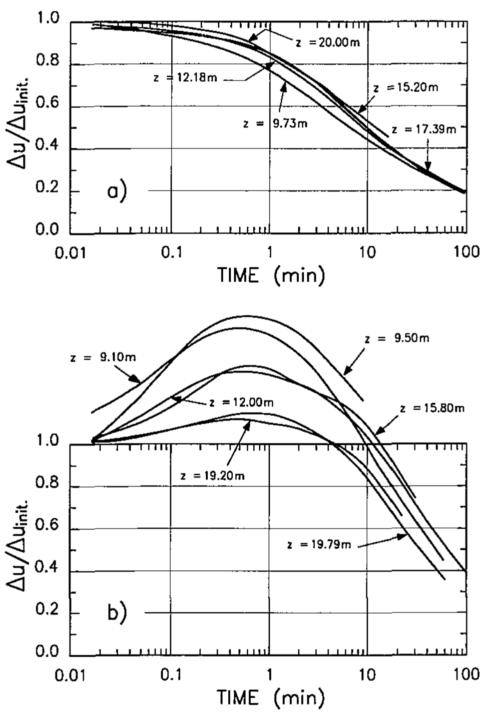

Figure 16. Dissipation tests on hard Taranto clay: a) filter at cone tip; b) filter at cone base (Bruzzi & Battaglio, 1987).

overconsolidated Taranto clay, using the Wissa piezocone (filter at the cone tip) and the ISMES piezocone (filter at the cone base). It is noted that the normalized dissipation curves obtained with the Wissa piezocone lie within a rather narrow range, in spite of the different depths at which the tests were performed, and are similar in shape to the well know theoretical ones. On the other hand, the curves obtained with the ISMES piezocone show a marked increase of pore pressure at the beginning of the test, the maximum value of u being reached about 1 minute from the test start. Qualitatively similar results have been obtained at the same site by means of piezometer probes of different configurations.

Since the reported tests were performed under carefully controlled conditions, this non-monotonic behaviour is not attributed to improper saturation of the pore pressure measuring device but, rather, to the intrinsic flexibility of the whole testing system (i.e., thrusting rig, penetrometer frame, rods, ect.) or to pore pressure redistribution following cone arrest, as qualitatively predicted by some recent theoretical analyses (Teh and Houlsby, 1991; Elsworth, 1993).

In Tab. 3 the values of c_h and c_v estimated by CPTU in two Italian normally-consolidated clay deposits are compared to the ones obtained from laboratory tests and/or large scale field test. Taking into account the high degree of approximation usually involved in the assessment of the consolidation coefficients, and the different role played by macrostucture in flow problems of different scale, the comparison provided in Tab. 3 seems fairly good.

Table 3. Consolidation coefficients from piezocone dissipation tests and other field and laboratory tests.

	Porto Tolle clay			Fiumicino clay	
	c_h (OC)	c_h (NC)	Reference	c_v (NC)	Reference
Piezocone	70-170	10-30	1)	2-8	-
Back-analysis of piezometer readings	63-95	-	2)	6	4)
Back-analysis of settlement records	-	15	3)	10	4)
Oedometer tests on standard specimen	9	17	3)	0.2-1	4)
Oedometer tests on large specimen	-	-	-	3	4)

Values of C_v and C_h in m^2/year

1) Battaglio & Maniscalco, 1983
2) Jamiolkowski & Lancellotta, 1984
3) Battaglio et al., 1986
4) Burghignoli & Calabresi, 1975

4. DIRECT METHODS FOR GEOTECHNICAL DESIGN

The Italian geotechnical community has recently provided some contributions towards the use of direct design methods based on CPT. The theoretical modeling of mobilized base and shaft resistance of piles is a very difficult task and, therefore, direct correlations with CPT measurements are often used in practical design. Indications for assessing the base resistance (q_b) of bored piles in silica sands from q_c are given in

Table 4. The correlations have been obtained from "deep" plate load tests in CC on Ticino sand.

Table 4. Values of q_b in dry Ticino sand from plate load tests in CC (Ghionna et al., 1993).

D_R (%)	p'o (kPa)	$\frac{q_b(0.02)}{q_c}$	$\frac{q_b(0.05)}{q_c}$	$\frac{q_b(0.10)}{q_c}$
91	40 to 200	0.08	0.14	0.20
89	62	0.08	0.17	0.21
55	37 to 320	0.10	0.19	0.23
q_b: base resistance mobilized at relative settlement s/D equal to 0.02,0.05, and 0.10 respectively.				

Similarly, the ratio E'/q_c of Table 2 can be used directly in the Schmertmann (1970) method to calculate settlement of shallow foundations in stratified soils.

Within the framework of the new Eurocode on geotechnical design, it is worthed to mention the work by Levi & Lancellotta (1991) regarding the assessment of ultimate bearing capacity (q_{lim}) of deep and shallow foundations in cohesionless soils. This has shown that direct relationships between q_{lim} and penetration resistances are better conditioned, from a statistical point of view, than theoretical equation based on rigid-perfectly plastic models.

With regard to cohesive soils, direct design methods based on CPT are not commonly used in Italy. This is probably due to the complexity and heterogeneity of many Italian cohesive deposits, which would result in local correlations of limited applicability, and to a higher reliability offered by the approaches based on laboratory tests results from high quality undisturbed samples.

5. FUTURE TRENDS, RESEARCH AND DEVELOPMENTS

Future development in penetration testing will involve equipments, applications to special investigation programs, basic research for interpretation, and testing procedures (Baldi, 1995; Jamiolkowski & Manassero, 1995).

The equipment for on-shore investigation will certainly increase its capabilities; due to the relatively low cost and to the development of electronics, more cone types will be soon available in the market.

The increasing public concern about both hazardous and non-hazardous waste disposals and polluted subsoils will expand the applications of CPTU (Manassero, 1994b), and of the recently introduced Envirocone, in terms of number of soil parameters monitored. With regard to off-shore investigation, an outstanding research program led by ISMES will produce an equipment able to operate in abyssal plans under 1000-2000 m of water, with a digital cone and compensated pore pressure measuring system (Smits et al., 1994).

The evaluation of the in-situ stress state and of the stress-strain history of soils probably represents one of the most challenging tasks in the interpretation of penetration tests. The use of multi-purpose probes such as the cone pressuremeter, and the combination of different in-situ devices, seems promising at this regard. Among the basic research activities it is worth mentioning the characterization of "intermediate" soils (sandy gravels, silty-clayey sands under partially drained penetration), being these subsoils conditions very common in the national scenario. To this end, penetration testing in calibration chambers equipped with shear and compressional wave velocity measurements is foreseen in the near future.

With regard to testing procedures, there is a strong need for standardization and quality control of several aspects of penetration testing, such as measurements precision, checks and calibrations, precautions, and equipment maintenance. At the same time, the use of sophisticated devices will inevitably require a deeper specialized knowledge and a continuous upgrading of the technicians involved in the execution. For these purposes a Committee has been set up by the Italian

Geotechnical Association, and a cooperation with an international Committee would be highly desirable.

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National Report -the current state of CPT in Japan-

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SYNOPSIS: The current state of cone penetration testing in Japan is presented. The CPT standard was established by Japanese Geotechnical Society in 1994. For determining the standard, the comparative study was conducted using 8 different types of the cone. There is no description in the standard on sleeve friction because much scatter in friction was observed. The cone factor N_{kt}, which is a very important factor to predict undrained shear strength, ranges from 9 to 14 for Japanese clays, based on the vane shear strength.

1. INTRODUCTION

In Japan, the standard penetration test (SPT) has been extensively used for all type of the ground except the soft clayey ground, and design parameters such as friction angle, deformation modulus, or resistance against liquefaction have been correlated with N values, using a lot of data accumulated in the past. However, the cone penetration test (CPT) has been gradually employed in practical soil investigations since the last decade. This is because: 1) scatter caused by operators in the CPT is much less than that in the SPT: 2) geotechnical information on the soil layers can be continuously obtained in the CPT.

This paper describes the current state of the CPT in Japan, in accordance with a request by the organizing committee of CPT'95.

2. GEOTECHNICAL DESCRIPTION OF JAPANESE SOILS

Population and industrial activities in Japan are mainly concentrated on coastal areas, in particular on alluvial plains where normally consolidated soft clay is thickly deposited. On such a ground, main geotechnical concerns are stability and settlement of foundations.

As a typical example of geotechnical properties of the alluvial plain in Japan, the soil profile at Kurihama is shown in Fig. 1. This site has been used by the Geotechnical Engineering Division of Port and Harbour Research Institute (PHRI) to study sampling methods and in situ tests for cohesive soil (Tanaka et al., 1994). The plasticity index, I_p for Japanese clays is generally large, especially compared with Scandinavian clays. Figure 2 shows the relation between I_p and clay fraction (particle size of clay fraction is smaller than 5 μm, according to the standard for soil classification of the Japanese Geotechnical Society (JGS)). It can be seen that the activity for Japanese clays is much greater than that for Drammen or Bothkennar clays. This may be attributed to differences in clay minerals as well as environmental condition during sedimentation. Geological survey indicated that the clay layer at this site is geologically normally consolidated: that is, the layer has not been previously subjected to the consolidation

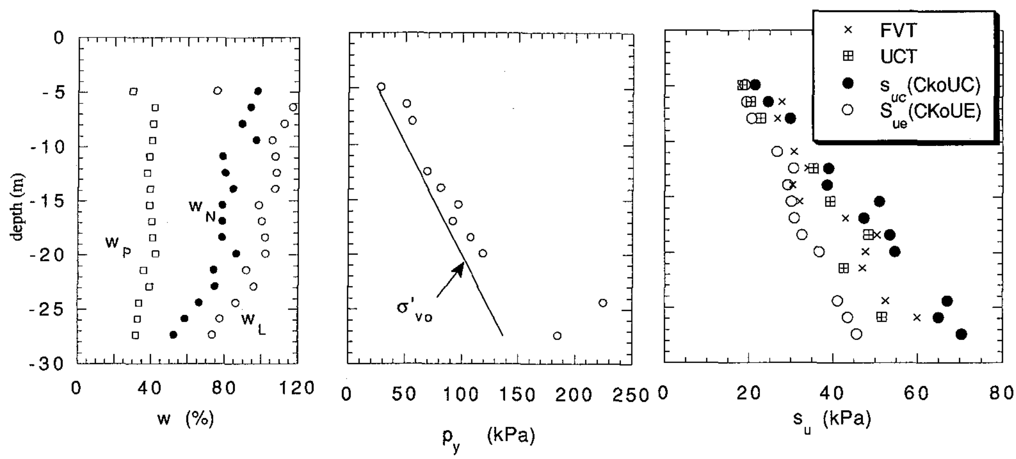


Fig. 1 Soil profile at PHRI's test field

pressure greater than the present burden effective pressure, σ'_{vo}. However, the yield stress measured by the Constant Rate Strain (CRS) oedometer test under strain rate $\dot{\varepsilon}$ = 0.02 %/min, p_y is slightly greater than σ'_{vo}. This may be due to the aging effect such as the secondary consolidation or the cementation. A usual clayey layer in Japan presents the apparent over consolidation ratio OCR of around 1.2 to 1.5.

In Japan, the undrained shear strength for cohesive soils, s_u is usually evaluated by the Unconfined Compression Test (UCT), while the Field Vane Shear test (FVS) is rarely used in practice except for research purpose. Tanaka (1994) has shown that the s_u value obtained by the UCT is the same as that by the FVS except at small I_p. It is recognized that the UCT cannot give a proper strength for soils with small I_p because these soils lose the negative pore water pressure when the specimen is exposed to the atmosphere. It can be seen in Fig. 1 that both strengths from the UCT and the FVS are also identical at the PHRI's test field. The shear strength obtained from the triaxial compression and extension test, s_{uc} and s_{ue} respectively, are shown in Fig. 1 where a specimen was consolidated under in situ K_o stress conditions. It is known that the s_u values obtained by the UCT or the FVS are almost the same as the average of s_{ue} and s_{uc} for Japanese clays.

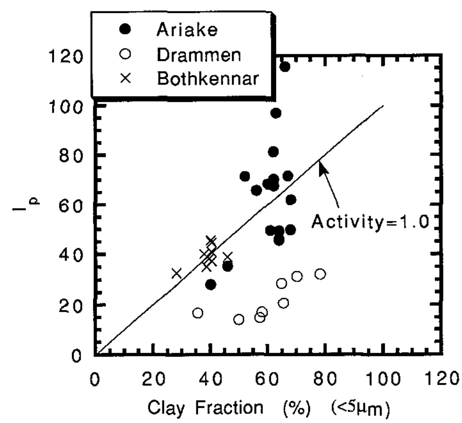


Fig. 2 Activity of clays at different regions

For the sandy ground, the main geotechnical concern is liquefaction at earthquake rather than bearing capacity or settlement. The great Kobe earthquake, which took place in January of 1995 and killed more than 5,000 people, severely damaged structures in particular along the coastal zone by liquefaction. It is known from the previous studies that resistance of liquefaction suddenly reduces when the relative density D_r becomes a certain value. Since Japan is a narrow and mountainous country in spite of much population, industrial and public facilities are quite often constructed on the ground reclaimed by

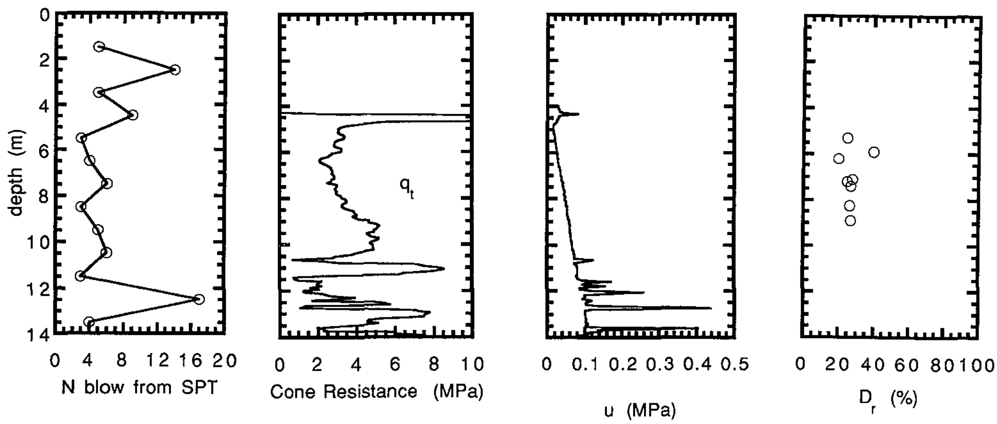


Fig. 3 Soil profile at a reclaimed land

sand. One of the most important task for the geotechnical investigation is to examine whether liquefaction takes place under an expectable size of earthquake. Results from a site investigation at a typical reclaimed ground is shown in Fig. 3. The site is East Ohgishima located 40 km south of the center of Tokyo. A loose sand layer reclaimed about 20 years ago is found from the ground surface to a depth of 10 m. The N values from the SPT are around 5 in this reclaimed layer. Using soil sample collected by a freezing sampling method, D_r was measured by laboratory test and its values are between 20 and 30 %.

3. CPT STANDARD OF JGS

The Japanese Geotechnical Society organized the technical committee in 1993 to establish the standard for CPT. At that time, eight different types of the cone had been already used in practice. Once the standard is determined, some types of the cone whose specification is out of the standard may be denied to be used. Before the standard was determined, the comparative study using 8 different types of the cone was conducted at PHRI's test field (see in Fig.1), with aid of private geotechnical consultants to know how much different values each cone measures at the same conditions.

3.1 Comparative study

The outline of the cones used in the comparative study is listed in Table 1. If an organization has several types of cones with different capacity, the committee asked the organization to bring a cone suitable for the soft clayey ground. The cone was penetrated at 1 m/min of the penetration speed and the distance of each penetration point was at least 2 m to avoid the influence of the previous investigation. The apex angle of the cone end is 60° for all cones used in this test. The cross sectional area (CA) of the cone is 10 cm^2 (diameter is 33.7 mm), which is the same as that for the International Reference Test Procedure (Penetration testing, 1988), except for the type f where the CA is 15.7 cm^2 (diameter is 44.7 mm). The capacity of the cone resistance q_t ranges from 20 to 50 MPa. The effective area ratio α, which is an important number to eliminate the influence of water pressure acting on the filter, was also reported by each organization.

All types of cones are capable of measuring friction f_s and pore water pressure u as well as q_t. Surface area for measuring f_s (SA) is between 100 and 200 cm^2. There are two methods to measure f_s: in one method (A in the table), q_t and f_s are independently measured: in another method (B in the table), q_t is measured in the same manner as the A method but f_s and q_t are recorded together. The f_s value can be calculated to reduce q_t from the combined record of q_t and f_s.

The location of the filter for measuring pore water pressure is the shoulder of the cone. However, the thickness of the filter (TF)and

Table 1 General information on the cones used in the study

Type of cone	Tip Resistance			Friction			Pore Pressure					
	C.A.[1] (cm^2)	Max (MPa)	α[2]	S.A[3] (cm^2)	Max (MPa)	M F[4]	F M[5]	P F[6] (mm)	T F[7] (mm)	Max (MPa)	D A[8]	R[9]
a	10	30	0. 62	150	0. 5	A	C	8. 5	6. 5	2	V. W.	N
b	10	20	0. 75	200	0. 5	B	PM	31. 1	12	1	V. G.	Y
c	10	50	0. 75	150	3. 3	B	?	10	5	5	?	Y
d	10	20	0. 72	100	0. 5	A	C	10	10	1	B. W.	Y
e	10	40	0. 7	150	1	A	C	7	5	2	US. G.	N
f	15. 2	20	0. 7	200	0. 6	A	PM	7	5	2	V. G.	Y
g	10	51	0. 8	150	1	A	PP	9. 8	5	3. 6	V. G.	Y
h	10	30	0. 7	100	1	A	C	10	6. 5	1	V. G.	Y

Notes: 1.Cross sectional area

2. effective area ratio
3. Surface area for measuring friction
4. Measuring System A: q_t, f_s independently, B: q_t, (q_t+f_s)
5. Filter Material C:ceramic, PM:porus metal, PP:polypropylene, ?: no information
6. Location of filter (h_e in Fig. 4)
7. Thickness of filter
8. Saturation method V: vacuum, B: boiling, US: ultra-sonic
   fluid W: water, G:glycerin
9. Rubber membrane Y: use, N: no use

its distance from the edge of the shoulder to the upper end of the filter (PF) were different in each types. Several kinds of material were used as a filter: fine ceramic (C), porus metal (PM) and porus polypropylene (PP). Saturation of the filter is very important to measure pore water pressure precisely. Each organization has own method for filter saturation because they use different material as a filter and followed their experiences. Two kinds of pore fluid are used to saturate pores in the filter: water (W) and glycerin (G). Some organizations apply vacuum to the filter (V), boils the filter (B) or vibrate the filter with ultra-sonic vibration (US). Some organizations cover the cone with a thin

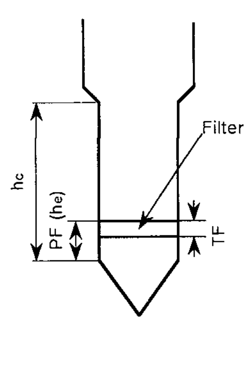


Fig. 4 Geometry of cone

rubber membrane to keep the filter saturated before cone penetration, while others do not.

Test results from the study is shown in Fig. 5. Even though q_t was corrected by effective area ratio α, there are still remarkable differences in the q_t value. This difference cannot be ignored when we intend to obtain s_u from q_t using eq. (1).

$$s_u = (q_t - \sigma_{vo})/N_{kt} \qquad (1)$$

where σ_{vo}: total burden pressure at the depth, N_{kt}: cone factor.

Lunne et al. (1986) conducted the similar comparative study to use 8 organization at 3 of NGI's reference sites, and pointed out that the differences in q_t can be significantly reduced if zero shift due to temperature and pore pressure effect (effective area ratio) are considered. Temperature effect on q_t was measured using the type a, and it was found that 60 kPa of the thermal zero shift was recorded for a temperature change of 10°C. This value is relatively larger than that reported by Lunne et al. (1988). The comparative study was carried out in August of 1993, when more than 30°C was usually recorded in the daytime. It is found from another survey that the temperature in the ground deeper than 3 m under the sea bottom is approximately constant through a year and its order is around 15 to 17°C. It should be considered that the site for the comparative study is not under the sea but on land, so that the temperature in the tested ground may be higher than the above figure. However, the difference in temperature between the atmosphere and the ground should be too great to be ignored.

As seen in Fig. 5, when a depth is deeper than 3 m, it seems that q_t linearly increases with depth. This observation is reasonable if we consider the fact that it took more than 6 minutes to penetrate the cone to a depth of 3 m. The duration of 6 minutes seems to be long enough for a thermal effect on q_t to be constant. Table 2 presents the gradient of q_t to depth, q_t/z and q_t at the ground surface q_t (z=0), assuming that the linearity of q_t to the depth at deeper than 3 m can be held to the ground surface. The values of q_t

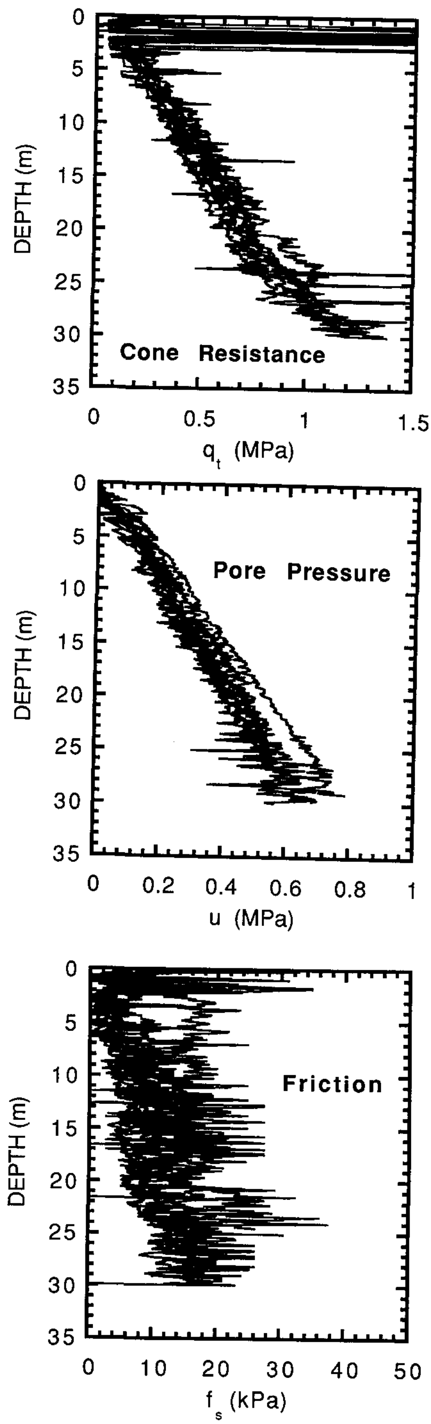


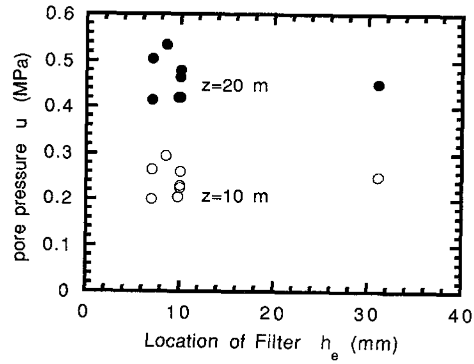
Fig. 5 Comparative study using different types of cone

Table 2 Cone resistance and pore water pressure at certain depths

Type of Cone	q_t		u	
	q_t(z=0) (kPa)	q_t/z (kPa/m)	z=10m (MPa)	z=20m (MPa)
a	50	35	0.295	0.535
b	50	35	0.25	0.45
c	0	33	0.225	0.465
d	100	32	0.261	0.48
e	100	35	0.265	0.505
f	50	28	0.2	0.415
g	75	33	0.205	0.42
h	70	36	0.23	0.42

(z=0) considerably vary with types of the cone. Two reasons can be considered: the sensitivity of the thermal effect may vary with types of the cone (unfortunately there is still no data on each cone except for the type a), another reason may be the difference in temperature at each penetration test. We should also point out that some organizations took care of the cone not to be directly exposed to the sunshine during the test. As a result, it seems that there are much difference in q_t (z=0) among different types of the cone as shown in Table 2. Indeed, the values of q_t/z are almost the same of all cones except for the type f whose cross sectional area is 15.2 cm^2. The q_t values should not be theoretically influenced by the diameter of the cone in case of cohesive soil, or in other word, under undrained condition. It may be too early to make conclusion whether the different q_t/z of the type f is caused by different cone diameter or by another reason.

The difference in pore water pressure, u among different types of the cone may be attributed mainly to location and width of the filter for measuring u, because the thermal shift on u is much smaller than that on q_t. It is well known that location of the filter has a strong influence on u. The u values measured at depths of 10 m and 20 m are shown in Fig. 6. Although the filter for the type of b is located farthest from the shoulder of all cones, the u value is not so much different from other types. Figure 6 suggests that the magnitude of u is not only influenced by the location of the filter, but also by the width of the filter or the filter material.



Fig, 6 Pore water pressure measured by various cones

The most shocking result from the study is friction f_s. Although there are many reasons for large scatter in f_s: temperature, pore water pressure as pointed out by the CPT International Reference Test Procedure, the most likely reason is the magnitude of f_s considering its capacity. The orders of measured f_s by various types of the cones are around 10 kPa, which corresponds to only between 1/330 to 1/50 of the capacity of sleeve friction (see Table 1).

3.2 CPT Standard

The CPT standard of the JGS was established in 1994, with reference to results from the comparative study conducted at the PHRI's test field and International Reference Test Procedure. The main points of the standard are as follows:

size and shape of the cone

1) apex angle of the cone: 60°
2) cross sectional area : 10 cm^2
   (diameter of the cone: 35.7 mm)
3) length for constant diameter
   h_c (see Fig. 4): longer than 300 mm

Filter for pore water pressure

4) location: behind of the shoulder
   h_e (see Fig. 4): less than 33 mm
5) width: no description
6) material: no description
7) method for saturating: no description

cone resistance

8) method for obtaining effective area ratio α : Chamber test

9) calculation of q_t: eq. (2)

$$q_t = P_m/A_p + (1-\alpha)u \quad (2)$$

where P_m: total force measured by strain gauge in the cone, A_p: cross sectional area (10 cm^2), u: pore water pressure, α: effective area ratio.

10) penetration speed: 1 to 2 cm/s

In the standard determined in 1994, there is no description on friction measurement because large scatters were observed in the comparative study and any reasons for them could not be identified. The effective area ratio, α is a very important factor to calculate q_t and may be obtained from a design drawing of the cone. However, the cone is not always manufactured precisely followed the design drawing, and due to existence of rubber rings to prevent water from coming into the inside of the cone, it is not clear which diameter should be taken as an effective area. Therefore, in the standard, α should be obtained by the chamber test in which water pressure can be properly applied to the cone.

4. INTERPRETATION OF TEST RESULTS

4.1 Soil classification

In Japan, soil classification using data from the CPT is conducted according to the chart proposed by Robertson et al. (1986) in which soil is classified by q_t and f_s values, or by Sennest and Janbu (1984) in which q_t and u values are used. However, as already shown in the comparative study, the accuracy of f_s is quite questionable. Further, we should bear in mind that the meaning of the soil classification. The soil classification system in most countries is based on particle size: that is, content of clay, silt and sand. However, as shown in Fig. 2, if we say clay, the activity is complete different, for example, between Drammen and Japanese clays. We need establish the world wide standard for soil classification based on characteristics of soils such as strength, permeability or compressibility, not based on size of soil particles.

4.2 Undrained shear strength

The cone factor N_{kt} (see eq. (1)) has been obtained by several researchers and there is an argument whether the N_{kt} factor is dependent on I_p or not. It should be kept in mind that what type of shear test is conducted for obtaining N_{kt}, because the s_u value is strongly affected by anisotropy and rate effect. In Japan, s_u for cohesive soils is evaluated by the Unconfined Compression Test (UCT) and is not modified by I_p like Bjerrum's correction factor μ to the vane shear strength. Figure 7 shows N_{kt} based on s_u from the UCT for Japanese marine clays (in the following test result, the type a cone was used). Although there is much scatter in N_{kt}, the N_{kt} value seems to be constant against I_p and ranges between 8 and 16. Figure 8 shows N_{kt} based on s_u measured by the filed vane test and its range is between 9 to 14. This tendency for Japanese clays is completely different from that for Scandinavian clays reported by Jamiolkowski et al. (1988), where N_{kt} based on the vane shear strength decreases with I_p increasing. Furthermore, they reported if the vane shear strength is corrected by Bjerrum's μ, then N_{kt} is nearly constant against I_p. Tanaka (1994) carried out the vane shear test to several Japanese marine clays and compared the vane shear

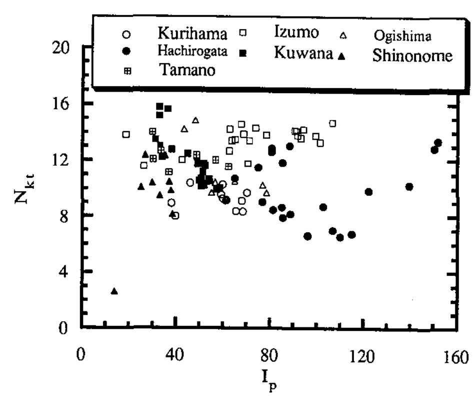


Fig. 7 Cone factor N_{kt} based on s_u from the UCT

strength normalized with yield consolidation pressure s_u/p_y to that in other areas as shown in Fig. 9. The s_u/p_y value for Japanese clays is constant against I_p and there is much difference between Japanese and Scandinavian clays especially at small I_p. Therefore, it might be considered that the reason for the large N_{kt} factor at small I_p is due to the small vane shear strength.

4.3 dissipation test

When the penetration of the cone is stopped, the excess pore water pressure gradually dissipates and the measured u finally becomes the hydrostatic pressure. The dissipation speed should be correlated with permeability of the ground. Robertson et al. (1992) reviewed a lot of published data and correlated the coefficient of consolidation from laboratory oedometer test, c_v, with time for a half of the excess pore water pressure dissipation, t_{50}, as shown in Fig. 10. The data observed in Japan are added in the figure and it can be seen that both Robertson et al.'s and the author's data have the same tendency that t_{50} decreases with c_v increasing. The data for Japanese, however, are located at much lower part compared with Robertson's. It is said that c_v has anisotropy due to the difference in permeability and compressibility between vertical and horizontal directions, and c_v for horizontal direction, usually denoted by c_h, governs the rate of dissipation of the excess pore water pressure in the cone test. It may be concluded that Japanese clays possess much stronger anisotropy in consolidation than data of Robertson's, as shown in Fig. 10. However, as already presented that the u value during cone penetrating is significantly influenced by type of the cone, it is

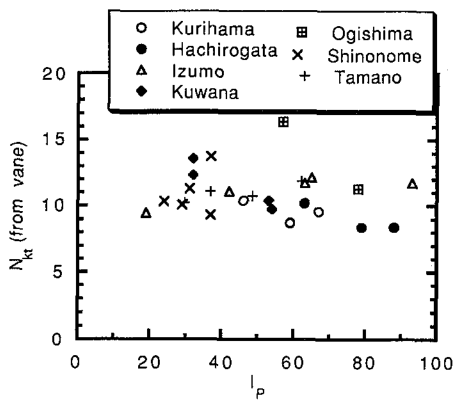


Fig. 8 Cone factor N_{kt} based on s_u from the vane shear test

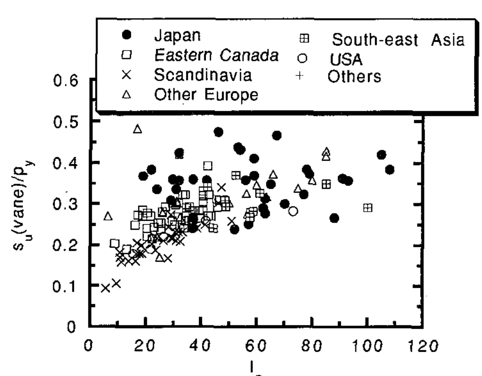


Fig. 9 Vane shear strength normalized with p_y versus I_p

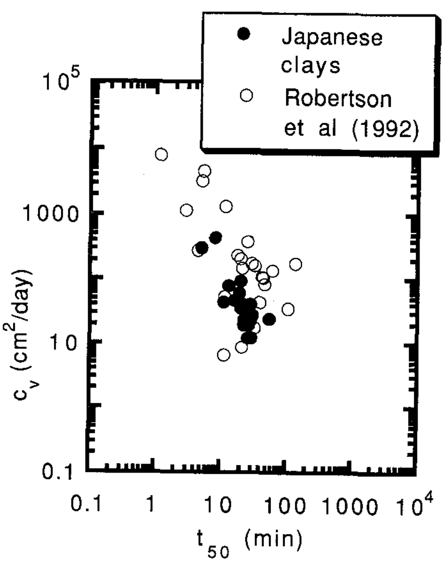


Fig. 10 Correlation of coefficient of consolidation from oedometer test with CPT dissipation test

likely that the dissipation speed is also affected by a specification on the filter, including location and material. Furthermore study is required to draw the defined conclusion whether consolidation behaviour is different from the regions.

5. APPLICATION TO PRACTICE

5.1 Use of the CPT in geotechnical design

Some geotechnical engineers in Japan recognize the superiority of the CPT, especially with capability of measuring pore water pressure, piezocone, to obtain design parameters or classify soils on the cost and accuracy over the conventional exploration methods, such as the SPT, or laboratory testing using soil specimen. However, the utilization of the CPT for practical investigation in Japan is quite restricted because the design codes determined by the Government or the authority are based on the N values from the SPT or the shear strength from the UCT.

Consequently, even when estimating the bearing capacity of a pile, the measured value from the CPT such as q_t should be transferred into the N value, then engineers are able to use "the authorized equation" such as q (tf/m^2) = 40N. It should not be forgot that soil exploration method is strongly connected with the design method.

However, in a relatively new field such as prediction of liquefaction, there is possibility to use the CPT. Some researchers proposed the method for liquefaction resistance using the CPT (Shibata and Teparaksa, 1988, Sugawara, 1989).

5.2 New development

The most advantage of penetration type in situ testing over other testings is that a borehole is not necessary. In the soil exploration, making borehole is generally so expensive to occupy the most cost need for the investigation. It is natural for geotechnical engineers to invent various type of in situ testing using the cone.

The cone capable of measuring water content or bulk density in the ground, using radio isotope and called RI cone, was invented by Mimura et al. (1995). In case of sand, it is very difficult to know the relative density, although it is a very important parameter to govern the mechanical behaviour. The freezing sampling method is quite effective to collect undisturbed sample for sand, but the problem is the cost. Instead of such an expensive method, the RI cone is expected to another method to obtain the in situ bulk density.

6. CONCLUSIONS

Because of limited space in this paper, emphases are mainly placed on the Japanese CPT standard established in 1994 and the test results from the comparative study. Unfortunately, because of large scatter in friction, there is no specification on the friction measurement in the standard. It is again recognized that the temperature effect on the cone resistance is important. It is necessary to develop the cone that is not affected by thermal change. The cone factor N_{kt} for Japanese clays are constant against I_p and its values range from 8 to 16 based on the shear strength by the Unconfined compression test, and from 9 to 14 by the field vane test.

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Cone Penetration Testing in Lithuania

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SYNOPSIS: The paper presents a short review of CPT testing in Lithuania. It was written trying to meet the requirements of the instructions to the authors of National Reports.
The author apologises for some subjective approaches, if any, and for not answering to all the questions presented because of lack of some information.

1. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION

Lithuania is a Balto-Scandinavian state. Its territory belongs to the East European Plain. The highest point of Lithuania is only at 295m elevation above the Baltic Sea level.

A crystalline bedrock surface in Lithuania is at the depth from 50m to 2200m below the sea level. On the bedrock deposits of all geological periods are encountered. Deposits from Devonian and Silurian periods are thickest.

Quaternary deposits cover the whole territory of Lithuania. Most of them originated from glaciers which came from Scandinavia. The thickness of quaternary deposits are from a few meters to three hundred meters. More than 50% of the territory of Lithuania is covered by glacial till, about 20% - by sands of glacial water streams or wind blown or river deposits. About 5% of the territory is occupied by swamps. Mostly these quaternary deposits served as the ground for buildings and structures with a small exception in the East-Northern part of the country. Distributions of the main types of quaternary deposits are shown on the map in Fig1.

In a soil profile at every point taken, as a rule, not less than two types of soil by nature (for example marine and glacial) could be found. Density of sands and moisture content of clays varies with depth. The ground water table is near the earth's surface (0--3m). For characterising ground conditions in Lithuania a plot of resistance versus depth is used. Table 1 shows six typical ground types. These types of ground were obtained on the basis of cone penetration tests.

2. SITE INVESTIGATION METHODS IN LITHUANIA

Site investigation in Lithuania started from the boring of wells. So at the beginning it was a description of soil types encountered during the boring by a geologist (later an engineer-geologist). The next stage was to check some physical properties in the laboratory, mainly on particle size distribution for sand and on moisture content and plastic limits for clayey soils. The next stage was to take undisturbed samples from pits and boreholes and to perform laboratory testing on them to obtain shear strength and stiffness.

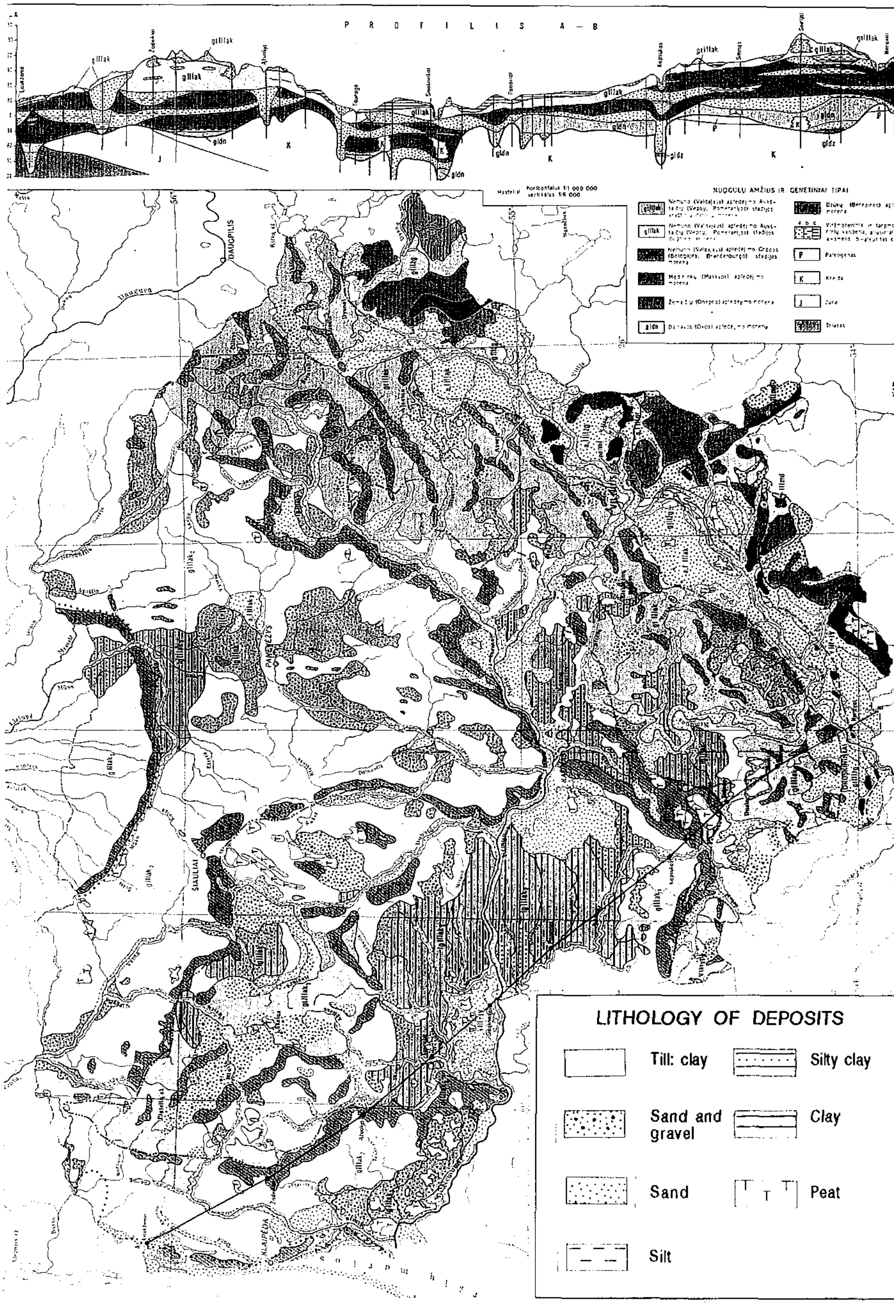
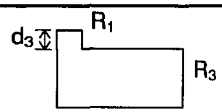
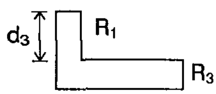
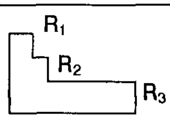
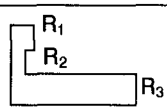
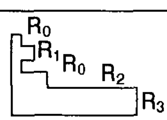


Fig.1.Map of Quternary deposits of Lithuania

Table.1.Type of ground in Lithuania (schematic view).

Type	Scheme	Featu-res	Description	Distribution %
1		$R_3 >> R_1$ $d_3 \le 3m$	Two- layered ground: 1st layer– thickness ≤ 3m, – resistance 150-300 kPa; 2d layer – resistance more ≥600kPa.	19.7
2		$R_3 >> R_1$ $d_3 \le 6m$	Two- layered ground: 1st layer– thickness ≤ 6m, – resistance 150-300 kPa; 2d layer – resistance more ≥600kPa.	51.0
3		$R_3 >> R_1$ $d_3 > 6m$	Two- layered ground: 1st layer– thickness > 6m, – resistance 150-300 kPa; 2d layer – resistance more ≥600kPa.	16.7
4		$R_3 >> R_1$ $R_2 > R_1$ $d_3 > 6m$	Three- layered ground	5.6
5		$R_3 >> R_1$ $R_2 < R_1$ $d_3 > 6m$	Three- layered ground	4.9
6		$R_3 >> R_1$ $R_0 << R_1$ $d_3 > 6m$	Ground with one or two layers of weak soil (resistance- R_0)	2.1

In 1965 the first cone penetration testing equipment was obtained at the Kaunas Polytechnic Institute. That was the beginning of CPT testing in Lithuania.

Today the main tool of field testing in Lithuania is cone penetration test.

Besides CPT, the following tools were extensively used:

- n-n and γ-γ ray logging (for density and moisture content),

- pressiometer (for soil modules and stiffness),

- plate load test (for soil modules and carrying capacity),

- screw plate load test (for soil modules carrying capacity of soil and behaviour of short bored piles),

- vane test (for weak soft soils),

- load test of various foundations (models and a natural size, for behaviour of foundations) with the maximum load of up to 2 MN.

A careful analysis with comparative methods revealed that n-n and γ-γ ray logging is very expensive and does not justify the value of the information obtained.

Pressiometer tests were accepted as reliable but because of being complicated for use in boreholes and owing to sensitivity of the results on installation and the state of boreholes, the pressiometer lost popularity and nowadays are not used at all.

The other methods are sometimes used in practice by site investigation firms.

Earlier many of them together with CPT were used to elaborate calculation methods for prediction the behaviour of various types of foundations (for example, piles, short bored piles, foundation on fill, etc.).

3. TYPE OF CPT-EQUIPMENT

Today in Lithuania electrical cones are mostly used. Two different types are spread among investigation firms:

- PIKA (produced by NII osnovaniy, Moscow, Russia),
- GEOTECH cordless (produced by GEOTECH, Sweden)

Table 2 shows the type of CPT-equipment and their distribution among firms.

Table.2.CPT equipment in Lithuania.

	Q	Type	Parameters		
			q_c	f_s	u
AB Inžineriniai Tyrinėjimai Vilnius	2	Pika	+	+	
UAB Geostatyba Vilnius	3	Pika	+	+	
UAB Kauno Inžineriniai Tyrinėjimai Kaunas	2	Pika	+	+	
AB Krašto Projektai Vilnius	1	Hydraulic head	+	total friction	
UAB MTJ Geoprojektas Klaipėda	1	Geotech cordless	+	+	+
UAB Geotechnikos Grupė Vilnius	1	Geotech cordless	+	+	+
Vilnius Technical University	1	Pika	+	+	

Calibration of the probes is made using calibration equipment which consists of a loading device with axial loading and high precision load cells. Usually force is produced by a hydraulic press.

CPT testing results are presented graphically as curves for the parameters q_c and f_s versus depth using the following scales: depth 1 m/10 mm, q_c 2 MPa/10 mm, f_s 20 kPa/10 mm.

The GEOTECH probe users use all possible scales which are involved in software, usually trying to meet standard requirements. But often the scale 1 MPa/10 mm is used for q_c graphs if weak soils are encountered.

As a rule, an interpreted soil profile or that obtained according to the borehole results is presented together with CPT parameters graphs.

Until now Lithuania has had no standard on CPT of its own. There are some collisions: the PIKA users are trying to meet the requirements of the old Soviet Union standard GOST, the GEOTECH users are trying to meet the requirements of the Swedish standard (1992). The Lithuanian Geotechnical Society is going to accept the last one as a national standard.

4. INTERPRETATION OF TEST RESULTS

CPT results are used for the profiling of the ground and for an assessment of soil parameters.

Soil profiling is made according to the changes of q_c values and according to f_s/q_c ratio. The GEOTECH probe users have a better possibility because of the pore pressure parameter, which enables to see the clay content.

For investigating the ground containing weak soil layers, first of all CPT testing is made. According to the results obtained soil profiling is done. This assumed profile is used as a bench profile and using the MOSTAP (van den Berg, the Netherlands) sampler checking of profile at definite points and depth is performed. According to the obtained and checked data a soil profile is corrected.

That CPTU is the best way of obtaining a soil profile under extremely complicated conditions is confirmed by Fig.2, where the graphs of two CPT tests of the Koenigsberg Cathedral ground are shown (Furmonavičius, 1995).

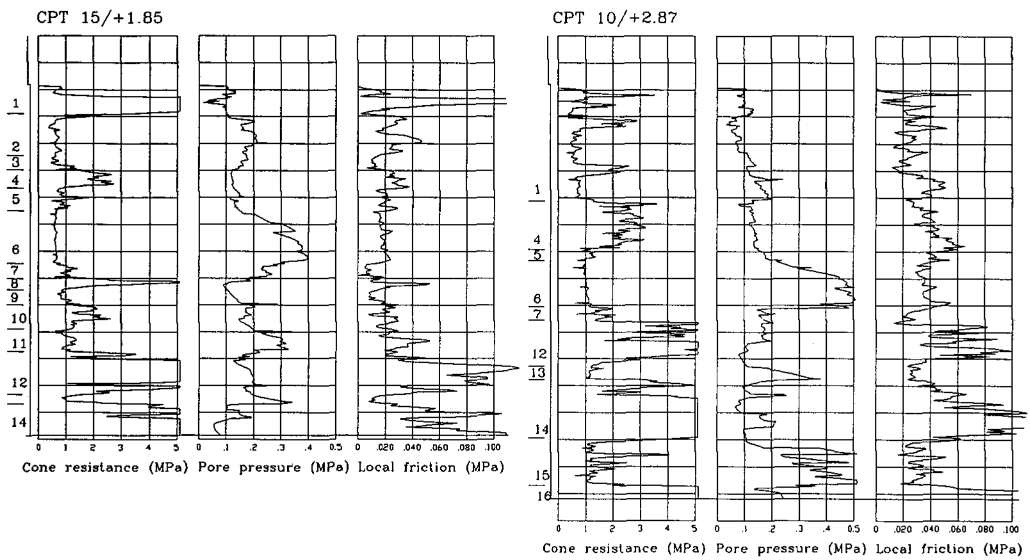


Fig. 2. The use of CPTU for soil profiling (Koenigsberg Cathedral site).

There are two tendencies of using CPT for soil profiling in Lithuania. In firms which are under the influence of the Vilnius Technical University CPT testing is used instead of borehole data, in firms which are under the influence of the Vilnius University borehole data are preferred for soil profiling. This is also reflected in the ratio between the number of CPT and that of boreholes. In Kaunas inžineriniai tyrinėjimai Ltd this ratio is 0,4 but in Geotechnikos grupė Ltd it is more than 5.

In Lithuania soil parameters are mostly determined according to the CPT results.

The formulas which are used for assessing soil parameters are shown in Table 3. When investigating sites for large projects for determining deformation modules, plate load tests are used, but a selection of the location and depth is always based on the CPT results.

5. USE OF CPT IN GEOTECHNICAL DESIGN

These are the foundations when CPT data are directly used for their design:

- shallow foundations on fill,
- short bored piles,
- piles.

For designing shallow foundations on fill, first of all sufficient compaction of fill is needed.

Cone resistance (for sand) and cone resistance and skin friction are prescribed for it. So CPT testing is used for checking compaction index. Allowable pressure on fill is shown in Table 4.

Table.3.Relationships use for assessing stiffness and strength.

Soil	Soil modulus		Shear strength
Sand	$E=1.5q_c$		φ according to Meyerhof graph
	$E=3q_c$		
	$E=7.8q_c^{0.71}$	*Brilingas 1985*	
Clay	$E=10q_c$	*L.Furmonavičius*	$c_u=q_c/20$
Silt	$E=5q_c$	*L.Furmonavičius*	$c_u=q_c/25$
Till: clayey	$E=7.4q_c+7.2$	*Brilingas 1985*	
Very soft organic clay, peat	$E=q_c$	*L.Furmonavičius*	$c_u=q_c/15$

Table.4. Allowable pressure on the fill.

Fill	Allowable pressure
sand	$q_c/20$
gravel	$q_c/30$
clay	$q_c/10$

The method of calculating short bored piles is based on the CPT data. It was developed by an experimental investigation of the behaviour of these foundations (full scale load tests) installed in a clayey till and sand and by comparing the results obtained with the CPT.

Fig.3 and Fig.4 shows the relationships between q_c and R_{sn}. In these figures R_{sn} means resistance at a desired settlement. The settlement is expressed in percentage of the pile diameter. This case shows how q_c values could be used directly to obtain resistance at a desired settlement.

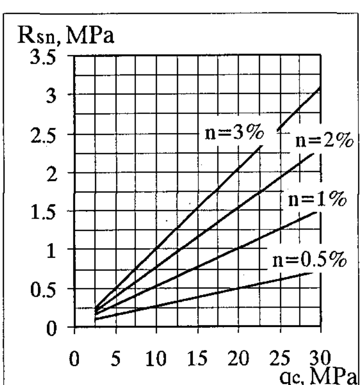


Fig.3. Graphs for defermining ground resistance under base of bored foundation for sand.

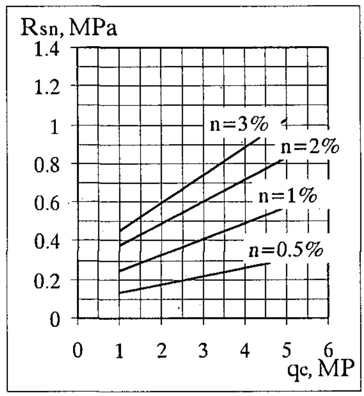


Fig.4.Graphs for defermining ground resistance under base of bored foundation for clayey till.

For determining carrying capacity of driven piles many simultaneous pile loading tests and CPT were carried out. An analysis of these tests yielded factors for obtaining the values of design point resistance and design skin friction. The draft of the National Code on pile foundations is based on these values. These are the following:

Table.5.Factors for pile carrying capacity.

Soil	Factors	
	K_p*	K_s**
sand	0.5	0.6
clay	1.0	1.0
saturated silt	0.5	0.8
	*$R_p=k_p \cdot q_c$	**$R_p=k_s \cdot f_s$

6. FUTURE TRENDS AND NEW DEVELOPMENT

It could be said that the main field tool of site investigation in Lithuania is CPT testing.

The first task is to standardise the CPT procedure. Another problem is to establish the Geotechnical Code based on design according to CPT data as being most reliable. At present ground models are being developed on the basis of computerised CPT results. Lithuania has no penetration testing for determining soil pollution. An introduction of such probes is also a major concern in Lithuania. CPT testing, undisturbed soil sampling together with adequate laboratory testing would provide for a new stage in determining ground conditions.

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Cone Penetration Testing in the Netherlands: State-of-the-Art

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SYNOPSIS: Soil conditions in The Netherlands suit Cone Penetration Testing (CPT). This led to the development of a CPT "industry" with an annual market of over 100,000 tests in the depth range 10 m to 60 m. ISO 9000 accreditation of CPT companies is in progress.

CPT interpretation is not the exclusive domain of geotechnical engineers and engineering geologists, but CPT knowledge is also common for civil/structural engineers and architects. This has obvious impact on the widespread use and acceptance of CPT's. In addition, technology transfer from geotechnical applications to hydrogeological and environmental applications is taking place.

Advancements in cone penetration testing are mainly in the area of equipment development, including a wide variety of probes and thrust equipment. The high efficiency of cone penetration testing extends into interpretation and geotechnical design. In particular, CPT's take a prominent place in the Dutch application of Eurocode 7 for geotechnical design. This includes a direct design method for piles based on CPT data. New concepts for standardisation of cone penetration testing accommodate current practice and anticipate future trends.

1. GEOLOGICAL SETTING

Geologically, The Netherlands is a delta area. At the end of the Mesozoic, The Netherlands formed part of an extensive subsiding basin, the North Sea Basin. The Paleozoic massifs of the Ardennes and Eiffel form the borders of the basin to the south and the east respectively. Sediment accumulations in this basin reached a thickness of about 1000 m in the western part of the Netherlands. Mesozoic limestone outcrops in the east and southeast. Quaternary deposits are of interest to most civil works. The depositional regimes of the Quaternary soils are mainly alternating marine, coastal, river and glacial.

Geotechnically, a distinction can be made between the western part of The Netherlands, called Holland, and the other areas. In Holland extensive areas with up to 20 m thick Holocene clay and peat layers are present, overlying sands. These layers are susceptible to settlements and foundation piles to about 25 m depth are common. In the centre, east and south of The Netherlands structures can often be founded directly on Pleistocene or Tertiary sand layers. NMSMFE (1985) provides further details.

The Dutch sector of the North Sea includes major oil and gas facilities. Soil conditions are mainly Pleistocene sands and clays. The sands are typically very dense and the clays are typically very stiff.

It is clear that the soil conditions in The Netherlands suit Cone Penetration Testing (CPT). This led to wide application of a range of cone penetrometers and thrust equipment.

2. CPT APPARATUS

2.1 Cone penetrometers

Table 1 presents a summary of cone penetrometers and cone-shaped probes that are in commercial use.

Table 1. Cone penetrometers and other push-in probes

Probes	*Projects*		*Measurements*
	Geotech.	*Environ.*	
Cone Penetrometers			
mechanical cone penetrometer (with friction sleeve)	****		cone resistance, sleeve friction
electrical cone penetrometer (with friction sleeve)			cone resistance, sleeve friction, inclination
- 100 mm^2 cone	*		
- 500 mm^2 cone	*		
-1000 mm^2 cone (standard)	****	**	
-1500 mm^2 cone	***	**	
piezo-cone penetrometer (with friction sleeve)	***	**	cone resistance, sleeve friction, inclination pore pressure
electrical conductivity cone penetrometer	*	**	cone resistance, sleeve friction, inclination, electrical conductivity
cone pressuremeter	*		cone resistance, sleeve friction, inclination, pore pressure, pressuremeter stress-strain load-, reload-, and unload data
seismic piezo-cone penetrometer	*		cone resistance, sleeve friction, inclination, pore pressure, S-wave velocity
accelerometer cone penetrometer	*		cone resistance, sleeve friction, inclination, accelerations for vibration monitoring
hydrocarbon cone penetrometer		**	cone resistance, sleeve friction, inclination, hydrocarbon detection
Cone-shaped Push-in Probes			
permeability probe	*	*	horizontal permeability
electrical water conductivity probe	*	*	electrical conductivity of pore water
nuclear density probe	*	*	soil density
chemical probe		*	pH, electrical conductivity, redox potential, chloride content, temperature
hydrocarbon probe		*	hydrocarbon presence
gas sampling probe		*	gas sampling
ground water sampling probe		**	ground water sampling
push-in piezometer	***	***	ground-water level

Geotech. = geotechnical site investigations
* = less than 100 tests per year
** = between 100 and 1,000 tests per year
Environ = environmental site investigations
*** = between 1,000 and 10,000 tests per year
**** = more than 10,000 tests per year

The following types of cone penetrometers warrant further discussion:

- The non-cilindrical electric cone penetrometer (NNI, 1982; ISSMFE, 1989) is no longer in use in The Netherlands.
- The 100 mm^2 cone penetrometer is a scaled-down version that fits lightweight thrust equipment operated from small vessels or light track/truck vehicles (Plasman and Peuchen, 1995)
- The 500 mm^2 cone penetrometer allows cone resistance measurements of up to 180 MPa, which suits the very dense sands of the North Sea.
- Currently, electric penetrometers are rapidly replacing mechanical cone penetrometers.

The use of specific "environmental" CPT's and similar cone-shaped probes is fairly sparse, but increasing. The principal reason for this is competition with hand-auger boreholes in conjunction with laboratory analyses. The application of the hydrocarbon cone penetrometer or hydrocarbon probe (Olie and Sellmeijer, 1995) is promising. The hydrocarbon probe provides continuous in-situ measurements of pure petroleum products. During testing, the hydrocarbon probe radiates ultra-violet (UV) light, which results in emission of fluorescence light by the petroleum product. A photomultiplier in the hydrocarbon probe detects the fluorescence emission. The occurrence and degree of emission depends on the presence, type and concentration of a hydrocarbon. The hydrocarbon probe detects gasoline, diesel, domestic fuel, motor oil and tar, each with different detection limits.

No manual data recording takes place in The Netherlands. Analogue graphic recorders are common for mechanical CPT's and for the older electric CPT equipment. Analogue/digital (A/D) systems are gradually replacing the analogue systems.

2.2 Calibration

The adoption of systematic calibration of CPTs or, better, metrological confirmation systems for CPTs (ISO, 1992), remains an area of concern in practice. In The Netherlands, accredited systems for truck-mounted electric friction cone penetrometer systems became operational in 1994. There are no formal systems for mechanical systems and more complex systems such as the piezo-cone penetrometer.

The key items of the confirmation system for electric cone penetrometer systems include the following:

- penetrometer geometry checks by calibrated caliper measurements.
- penetrometer sensor calibration by a laboratory load facility
- depth sensor checks by survey tape measure-ments
- calibration of load and depth sensors every 3 months.

There are no formal checks for measurement errors resulting from vibrations and shocks during penetrometer penetration, eccentric loading of sensors, water and air pressure variations, temperature shift and variations in electric source (Post, 1995).

3. TYPES OF CPT-EQUIPMENT USED IN THE NETHERLANDS

Table 2 shows the major features of thrust equipment operating in The Netherlands. Currently, the number of operational CPT units exceeds 100, a majority of which is truck-mounted. Cone penetration testing in The Netherlands may be classified as an industry that operates in accordance with well defined processes. This not unexpected, as the annual market comprises over 100,000 tests in the depth range 10 m to 60 m.

Table 2. Thrust equipment

CPT Thrust Equipment	*Use*
Onshore	
thrust machine mounted on 6 * 6 truck	****
thrust machine mounted on track-truck with crawler part between the axles	***
truck with piggy-back tracked vehicle equipped with thrust machine	***
thrust machine mounted on tracks	***
thrust machine mounted on trailer and other special mounting	**
Offshore	
wireline downhole CPT in borehole	**
downhole CPT in borehole with no umbilical	*
seabed CPT, remote controlled	**
Thrust Capacity	
200 kN	****
100 kN	****
20 kN	**
10 kN (for 100 mm^2 cone *penetrometer)*	*
Special Features	
sideways push of CPT rods by means of a clamp	***
mud injection for friction reduction	**
backfilling by bentonite injection during pulling of CPT push rods	**
continuous (wheeldrive) thrust for seabed CPT	**
* = less than 100 tests per year ** = between 100 and 1000 tests per year *** = between 1,000 and 10,000 tests per year **** = more than 10,000 tests per year	

Cone penetration test equipment for use in canals, rivers and lakes is similar to "onshore" equipment, except that equipment mounting consists of a range of vessels and small jack-up platforms. Special equipment is in use for offshore CPT's.

Demand for deeper penetration of CPT's increases as a result of increased in-ground and underground construction. Techniques available for deep CPT's in The Netherlands include:

- conventional drill-out of a CPT stroke followed by installation of casing and further penetration
- downhole wireline CPT's in a borehole (Power and Geise, 1995)
- mud injection to advance the limits of thrust of conventional thrust machines and push rods (Van Staveren, 1995).

Figure 1 shows a diagram of the mud injection technique.

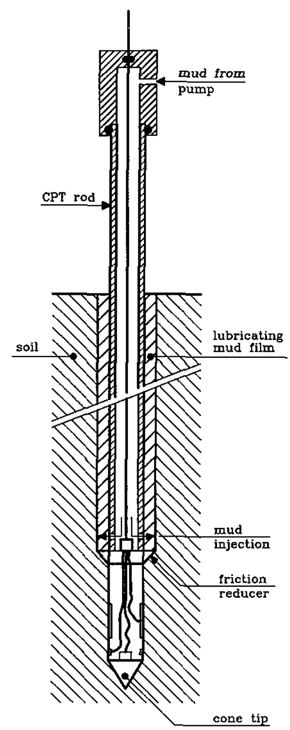

Figure 1. Mud Injection

4. INTERPRETATION OF TEST RESULTS

4.1 Dissemination of CPT knowledge

Educational programmes for civil/structural engineers and architects in The Netherlands include basic interpretation of cone penetration tests. CPT interpretation is not the exclusive domain of geotechnical engineers and engineering geologists. This has obvious impact on the widespread use and acceptance of CPT's.

4.2 Soil classification and stratigraphy

Cone resistance and friction ratio are the basis for most procedures for interpretation of soil type and stratigraphy. These parameters are supplementary to local knowledge of soil conditions and geology.

Digital data recording facilitates software-based interpretation of soil classification and stratigraphy; for example program UNICLASS (Peuchen et al., 1995). Huijzer and Hannink (1995) developed a semi-automatic method for modelling of soil stratigraphy for use in data base systems. Boundaries of layers are determined by normalised cone resistance and sleeve friction profiles.

4.3 Soil parameters and other data

In general, geotechnical parameters are interpreted from conventional CPT results by using empirical correlations. Difference is made between limit states at which failure of ground occurs and limit states at which large deformations of ground occur. Furthermore plasticity models for drained (sand) and undrained (clay) behaviour are distinguished. Regarding failure, correlations between cone resistance and the effective angle of internal friction ϕ' (in sand) and the undrained shear strength c_u (in clay) are used. Concerning deformation, correlations of cone resistance with Young's modulus E, shear modulus G, coefficient of volume change m_v and coefficient of consolidation c_v are applied.

4.4 Environmental data

The piezo-cone penetration test and push-in piezometers are the important (CPT-type) sources for environmental data in The Netherlands. Piezo-cone penetration test data provide information for hydro-geological studies related to environmental projects, in particular:

- permeability of cohesive soil layers
- detection of thin clay layers within permeable deposits
- determination of piezometric heads with depth, and laterally in case of multiple tests.

The push-in piezometer (Figure 2) consists of a cone-shaped probe with a filter element protected by a sleeve. Conventional CPT thrust equipment allows rapid installation without drilling and placement of filter/sealing materials. Exposure of the filter is by retraction of the protective sleeve. Further data acquisition and interpretation for the push-in piezometer is as for a conventional standpipe piezometer.

5. USE OF CPT IN GEOTECHNICAL DESIGN

5.1 Extent of soil investigation

Geotechnical design requires the generation of a geotechnical model, with a balance of cost, applicability, measurement accuracy and sensitivity of results. CUR (1992) provides detailed guidance on good practice for soft ground such as found in The Netherlands. The guidance includes rating systems for various CPT techniques, supplementary techniques such as engineering geophysics, boreholes and piezometers, laboratory testing and associated empirical correlations.

5.2 Direct method for pile design

Dutch standard NEN 6743 (NNI, 1991b) specifies a direct CPT method for calculation of axial compressive pile bearing capacity. In fact, NEN 6743 is an elaboration of Eurocode 7 (CEN, 1994). The CPT method uses cone resistance only. A suite of standardised adjustment/correlation factors is available for calculation of end bearing resistance and shaft friction for a wide range of pile types and three types of soil groups. The partial resistance factor applies to the total pile bearing capacity, that is the shaft resistance plus end bearing. The actual value of the partial factor depends

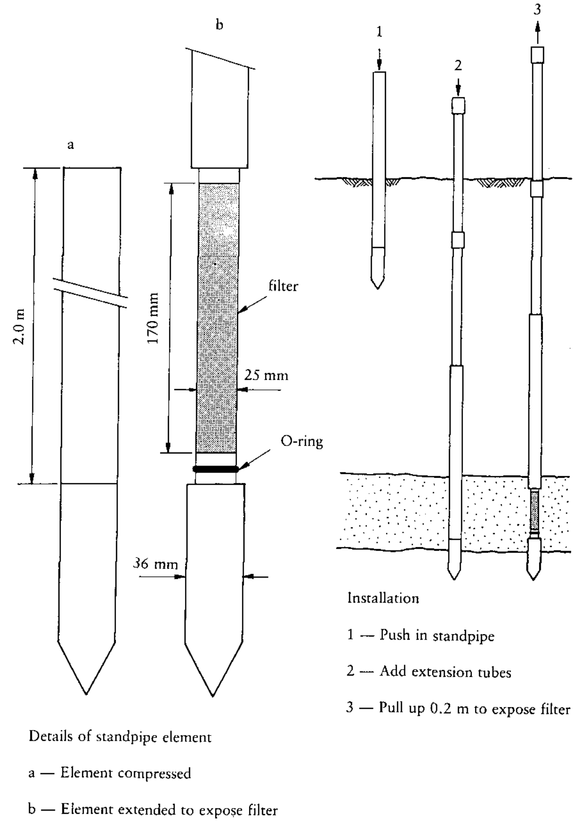

Figure 2. Push-in piezometer

on:
- the ability of a structure to redistribute load between the piles
- the ratio of the quantity of CPT's to the quantity of piles; in other words, there is a trade-off between the extent of a site investigation and the economics of a pile foundation.

Computer program NENGEO (Delft Geotechnics, 1993) reads digital CPT data and performs the pile calculations according to NEN 6743.

Principles of statistics form the basis for the selected values for the partial safety factor. It is necessary to apply due care in case of a non-stochastic soil distribution, for example due to geological features (Van Tol, 1995).

5.3 Indirect methods

Dutch standard NEN 6740 (NNI, 1991a) permits indirect geotechnical design where CPT results are available but no specific measurements of soil parameters. The entry to the NEN procedure is cone resistance normalised to a vertical effective stress of 100 kPa. The friction ratio allows classification into soil types. Resulting geotechnical parameters include undrained shear strength, angle of internal friction, Young's modulus of sand and compression indices for clay and peat.

CUR (1992) presents further details on cone penetration test interpretation for soft ground. In particular, it summarises a range of empirical correlations for undrained shear strength, compression index and oedometer modulus.

6. COMPARISON AND CORRELATION OF CPT'S WITH OTHER INVESTI-GATION METHODS

The cone penetration test is the major soil investigation technique. There is no significant use of other in-situ test techniques such as the standard penetration test, the dilatometer test, the vane shear test or dynamic probing. An exception is the pressuremeter test, the popularity of which is increasing due to the availability of the more efficient cone pressuremeters that suit conventional CPT thrust equipment.

Comparisons of CPT results and conventional laboratory test results are common. Comparison of CPT results and cone pressuremeter test interpretation is a relatively recent development (Zuidberg and Post, 1995).

Integration of CPT's with boreholes and geophysical techniques showed a significant increase over recent years. Onshore, the electro-magnetic and geo-electrical methods are common geophysical techniques. Seismic reflection is the most common geophysical technique for offshore soil investigations.

7. RESEARCH AND DEVELOPMENT

Continuing research and development of commercial CPT equipment takes place in The Netherlands. For obvious reasons, such research and development remains unpublished until commercial advantage applies. Developments include:

- optimisation of special CPT probes such as the cone pressuremeter (Zuidberg and Post, 1995) and application of P-wave measurements for the seismic cone penetrometer
- thrust equipment permitting more than 300 m CPT production per day
- so-called track-trucks that allow both wheeled vehicle and tracked vehicle access to sites with minimum change-over time.

Subsidised/PhD type research and development is limited and mostly confined to desk/laboratory work. Some examples are given below.

Prediction of variations in cone resistance before and after excavation continues to receive interest. The reason for this is the CPT-based pile design method. CPT performance usually takes place before construction. Subsequent excavations can affect the cone resistance and hence the pile capacity. Everts and Webb (1994) describe some recent research on this subject.

Research of the cone penetration process in clayey soils includes finite element modelling in clayey soils. The results show relationships between the roughness of the cone, the cone resistance, the clay stiffness/strength and the initial state of stress of the clay (Van den Berg, 1991 and 1994). Similarly, Van den Berg et al. (1992) and Van den Berg (1994) conducted finite element modelling of cone penetration in sand. The results indicate that the cone resistance is highly dependent upon the friction angle and initial state of stress.

8. FUTURE TRENDS

8.1 Equipment

Cone penetration testing in The Netherlands shows an obvious trend for increased application of advanced electronics. This includes A/D data conversion in the cone penetrometer, wireless data transmission and downhole recording (Power and Geise, 1995).

The demand for deep CPTs increases. Mud injection for friction reduction is not new, but

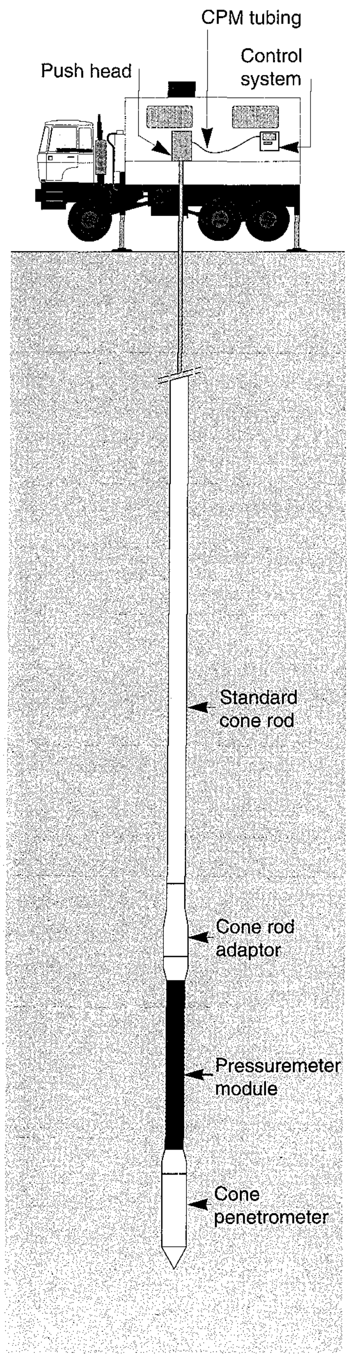

Figure 3. Cone pressuremeter

more cost-efficient and practical apparatus is now in use in The Netherlands to meet this demand (Van Staveren, 1995). The mud injection technique also allows efficient backfilling of the hole upon retraction of the push rods/drill rods. Backfilling requirements are becoming more common for hydro-geological or environmentally sensitive projects.

The so-called track-trucks are becoming increasingly common because of their efficient access to a wide variety of terrain conditions.

An increase in the use of lightweight CPT systems based on a 100 mm^2 cone penetrometer is likely. In particular, over-water operations from small vessels in canals, rivers and coastal/offshore are commercially attractive.

The trend of increasing the number of measurements and/or combination of geo-technical and/or environmental functions in the same probe will probably continue. In particular, the cone pressuremeter (Figure 3) is leaving the prototype phase and the annual number of tests will probably rise to above 1000 within 1 or 2 years. Similarly, the experiences with the ground water sampling probe and the push-in piezometer will probably lead to an increased use for geohydrological and environmental applications.

8.2 CPT practice

The draft version of the Dutch NEN 5140 standard (NNI, 1995) introduces new concepts for standardisation of electric cone penetration testing. The most important feature is a classifi-cation system for the accuracy of the CPT results. The classification system allows the designer to select a CPT class that suits the project requirements. The accuracy classes include uncertainties of 50 kPa, 250 kPa and 500 kPa for the cone resistance q_c (Peuchen et al., 1995). Current testing is mostly according to the 500 kPa class. A trend towards higher accuracy is likely for special projects. The introduction of formalised quality control and accreditation of CPT companies started in 1994. In future, it will probably become a "license for business".

8.3 Data applications

The increased availability of digital CPT data is likely to result in more advanced, but also more common, automated stratigraphic classification systems, including digital ground modelling. Since 1990, the city of Amsterdam operates a regional data base (GIS) system that relies on CPT data (Herbschleb, 1991).

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Cone Penetration Testing in New Zealand

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SYNOPSIS: This paper is a national report on cone penetration testing in New Zealand which has been prepared in response to an invitation from the organising committee for the CPT'95 International Symposium.

Cone penetration testing was first introduced into New Zealand by the then Ministry of Works in 1960. The testing technique is well suited to the soft soils encountered in many parts of the country.

The popularity of the technique has varied with the most use and development being in the upper North Island where there are extensive soft estuarine, alluvial and volcanic soils. In recent years electric systems have been further developed with portable PC data collection and processing and the introduction of the piezocone.

1. INTRODUCTION

This paper is a national report on cone penetration testing in New Zealand which has been prepared in response to an invitation from the organising committee for the CPT'95 International Symposium.

The Cone Penetration Test (CPT) was first introduced into New Zealand by the Ministry of Works in 1960 (MOW, 1961). Dr Begemann of the Delft Soil Mechanics Laboratory visited New Zealand to assist with the investigations for an oil refinery in Northland. Results from the CPT testing were found to be very encouraging and led to the wide spread application of the CPT by the then Ministry of Works.

The testing technique is well suited to the soft soils encountered in many parts of the country.

The popularity of the technique has varied with the most use and development being in the upper North Island where there are extensive soft estuarine, alluvial and volcanic soils. In recent years electric systems have been further developed with portable PC data collection and processing and the introduction of the piezocone.

2. GEOLOGY

New Zealand is a relatively young country in geological terms with a great variety of terrain. Located on the active boundary of the Pacific and the Indian tectonic plates, New Zealand is a country of moderate seismic activity. The geology (Figure 1) is varied and complex with igneous, sedimentary and metamorphic rocks and many land forms associated with recent fault movement and volcanic activity.

Much of the central and upper North Island is soft rock (tertiary mudstone, siltstone and sandstone) with extensive volcanic tephra deposits. The lowland areas and coastal margins comprise alluvial and estuarine soils which are typically fine grained (sand, silt, clay and peat). In the lower North Island sandstone and argillite rocks are widespread.

The South Island is dominated by the sandstone, argillite and schistose rocks which form Southern Alps. Erosion of these rocks have produced extensive alluvial gravel deposits forming the coastal plains along the eastern and southern parts of the island.

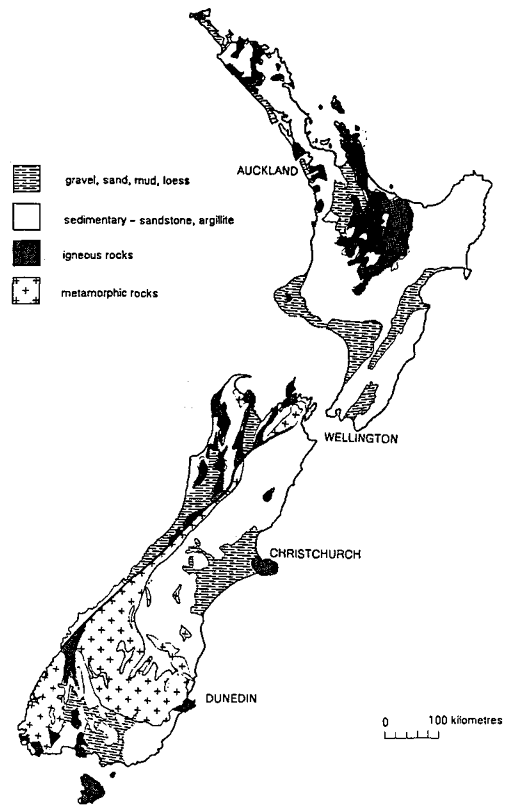

Figure 1 Simplified Geology of New Zealand

3. TESTING AND INVESTIGATION METHODS

Geotechnical site investigation methods utilised in New Zealand range from conventional borehole drilling through to non destructive tests such as ground radar. Site methods commonly used include:

- Scala (dynamic) penetrometer
- auger bore holes
- cored and wash drilled holes
- SPT (standard penetration tests)
- CPT (cone penetration tests)
- geophysical (surface and downhole)

While the Scala penetrometer is a light and convenient method of profiling soils it is limited in depth and in terms of reliability. Drilling, together with SPT testing, is the most common subsurface investigation technique utilised in New Zealand. It is widely recognised that the CPT is a cost effective and useful method for subsurface profiling. Use of the CPT is common in the volcanic, alluvial and estuarine soils in the upper North Island. With increasing emphasis on the seismic performance of foundation soils the CPT is now widely used for the assessment of liquefaction potential.

4. CPT EQUIPMENT

CPT testing was first introduced into New Zealand by the former Ministry of Works in 1960. Test rigs were operated throughout the country by the Ministry of Works. Electronic cone penetration testing was introduced in 1975.

In the past decade a number of test rigs have been converted to perform both manual and electronic CPT tests. Data is now gathered by computer based systems and can involve large volumes of data. Test rigs are operated by:

- Works Consultancy Services Ltd (Six rigs)
- Contractors (Two rigs)
- Universities (Two rigs)

There are only two truck mounted rigs in operation. The largest rig is a 20 tonne rig operated by Works Consultancy Services, Hamilton. Most rigs are 10 tonne units. Mechanical probes are widely used in gravel soils. The CPT is the most economic and effective subsurface field testing technique currently in operation in New Zealand.

Usually test probes extend to about 20m depth but in some situations tests to greater than 50m have been undertaken to prove pile foundation conditions (Figure 2) and soil profiles for liquefaction assessments.

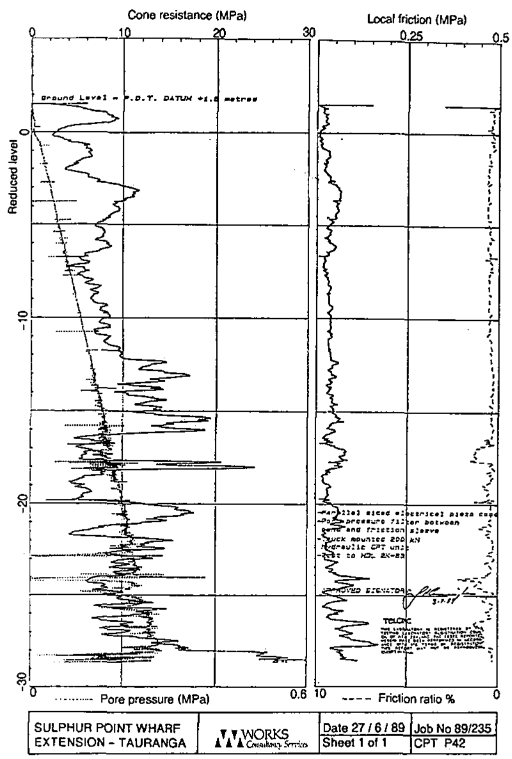

Figure 2 Typical deep CPT probe

Some investigation test methods have been specified in the New Zealand Standard, NZS 4402. The cone penetration test method (NZS 4402:1988) specifies the mechanical and electric CPT equipment and test procedure.

Continued vigilance is required with electronic testing to ensure that test results are calibrated and reliable. This is particularly important when testing soft soils as zero shifts and drifts can significantly influence results. Care is required where changes in ground temperature, eg as found in geothermal areas, can produce unreliable results.

5. INTERPRETATION OF CPT RESULTS

Using correlations between indices of field and laboratory tests and geotechnical model parameters is well established in geotechnical engineering. There have been numerous relationships established and published in the international literature for the CPT test, however there are no correlations that have been specifically developed for New Zealand soils.

In New Zealand,publications from Europe and North America have been adopted. In the 1960's interpretation was based on the work of Begemann (MOW, 1960). Experience in New Zealand led to the publication of local interpretation applications of the CPT (Belshaw, 1973).

One of the first more comprehensive guidelines widely utilised in New Zealand was the FHWA 1978 manual (Schmertmann, 1978). Following this the CIRIA publication on methods and interpretation (Meigh, 1987) provided an excellent update and guidelines which are widely applied.

Subsequently the geotechnical industry has more actively followed international cone penetration testing and interpretation developments. Interpretation techniques have been assessed for local soils and the effectiveness and application of CPT testing continues to be developed. The two university civil engineering schools in New Zealand are actively using the CPT to characterise and assess the performance of soils.

6. DESIGN AND ANALYSIS

The primary design applications for the CPT relate to:

- profiling soil distribution
- assessing settlement potential
- design of embankments
- design of foundations
- pile design in soils
- assessment of liquefaction potential

The cone penetration test is commonly used to investigate and characterise soils. Classification is commonly based on Robertson (1989).

The piezocone has added an important dimension to CPT testing and its application. For example the ability to measure consolidation characteristics with the dissipation test has real benefits in providing direct measurement of soil properties (eg Jones & Rust, 1993).

7. CPT CORRELATIONS

No New Zealand correlations have been established and published to date. A number of geotechnical engineers have wide experience in the use of the CPT and have developed a sound understanding of its applications and limitations. For example settlement estimates in normally consolidated alluvial soils can be reliable (Belshaw, 1973) whereas application of the same technique to highly weathered soils of volcanic tephra origin can provide grossly conservative settlement estimates.

8. MAJOR RESEARCH ACTIVITIES

Research in New Zealand has focussed on the evaluation of assessment techniques for New Zealand conditions. This has particularly related to the assessment of liquefaction potential. Because of the range of soil types, including lightweight volcanic pumice soils common in the North Island, there have been questions regarding the applicability to New Zealand conditions.

In recent years a series of projects have been undertaken by Works Consultancy Services to evaluate the effectiveness of the CPT and published correlations in areas of historical seismic liquefaction of soils. This programme of research has considered a variety of analysis techniques (eg Seed & De Alba, 1986; Shibata & Teparaksa, 1988; Sugawara, 1992). The research has found that the method of Sugawara (1992) has generally provided an effective assessment of liquefaction potential consistent with field observations.

Auckland University is planning a programme to investigate pumiceous sands and weathered soils to establish local correlations for soil properties.

Canterbury University has undertaken a number of projects which have investigated the use of the piezocone to assess liquefaction potential. This has led to more detailed research into layer effects on penetration resistance (Berrill et al, 1995).

9. FUTURE TRENDS AND DEVELOPMENTS

The CPT is widely accepted as an effective and efficient geotechnical investigation technique. It is ideally suited to the fine grained soils found in many parts of New Zealand. With the increasing ability of PC's to manage and manipulate data it is clear that systems will be further developed to enable the mass of data generated to be more effectively processed and analysed.

10. ACKNOWLEDGEMENT

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NATIONAL REPORT ON CONE PENETRATION TESTING NIGERIA

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1. INTRODUCTION

Penetration testing basically involves pushing or driving a steel tube with open end or cone tip into the subsoil and monitoring the resistance to penetration mobilised by the soil. Although penetration testing has been extensively used in some parts of the world, particularly in Europe and America, it is only relatively recent that the technique has achieved widespread use in the wider english speaking world.

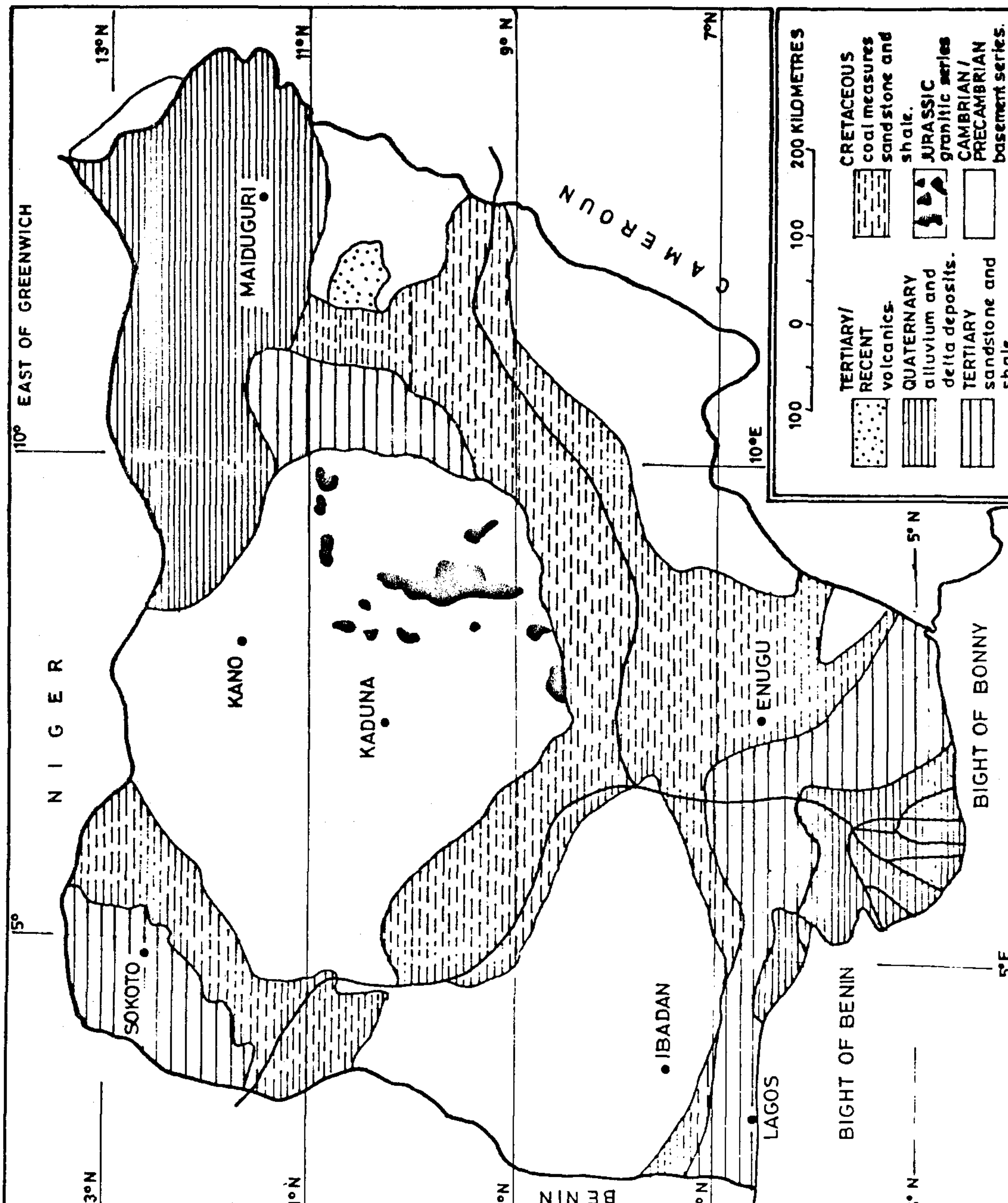
The standard penetration test method was introduced into Nigeria in about 1954 while the (Dutch) static penetration type was about four years later. Both has since gained wider application as a supplement to borehole and soil sampling modes of subsoil investigation. The equipment and methods of application have been strictly restricted to the practice in Europe and particularly in accordance with the relevant British Standards.

Hence, the two most widely used penetration test methods in Nigeria are:-

i) the dynamic penetration test in which a falling weight drives a string or rods connected to a split tube sampler or steel cone (Standard Penetration Test) in a borehole

ii) the static (DUTCH) cone penetration technique using the 2 to 20 tonne capacity machine.

NIGERIA - GEOLOGY



In testing recently deposited Qarternary sedimentary soils prevalent in the southern part of Nigeria, see Figure 1, these methods of testing have presented no problems in execution and results application. However, experience in the greater part of the country underlain by lateritic, residual or redeposited soils of the Pleistocene to Pre Cambrian era has not produced results which are as reliable as they are known for essentially sedimentary or temperate climate soil conditions.

2. STATIC CONE PENETRATION TEST

The static cone penetration testing technique and equipment commonly used in Nigeria are generally in accordance with the Dutch system and particularly, using the 2, 5, 10 and 20 tonne capacity machines with hydraulic gauges or electronic measuring device. The test cones are of 60^0 apex angle with 10 sq.cm cross - sectional area, forced into the soil at a rate of 20cm per second. The types generally used are the mantle cone or friction jacket cone provided with a friction sleeve having a 150 sq.cm surface area. Using the latter type of test cone soil adhesion around the sleeve (fs) as well as cone resistance (q_c) can be measured during the test. See Figure 2 for typical result. The use of electric cone and piezo-cone for testing is just generally gaining ground.

Primarily, the problems posed by the use of the Dutch cone penetration testing gear are connected with the scarcity of spares for maintenance of machines and replacement of worn-out parts. However, the most common operational problems often over looked by operators are:

Location : CREEK ROAD, APAPA LAGOS.
Job :
Date : 6/2/95.
Ground Level : -

Test No. : 2
Machie : 20T
Cone Type : PIEZOMETER
Ground Water: 0.5m BELOW GL.

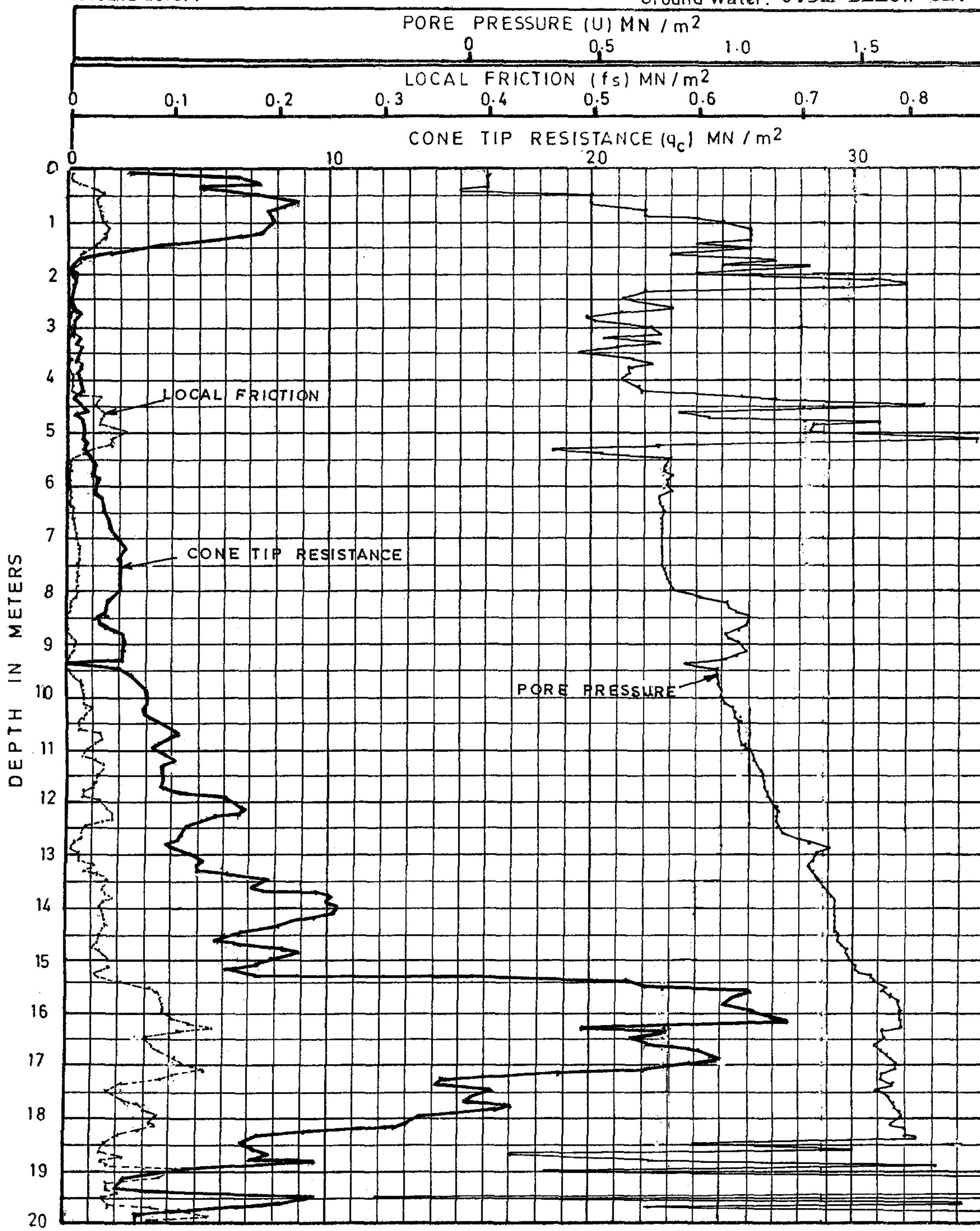


FIG. 2

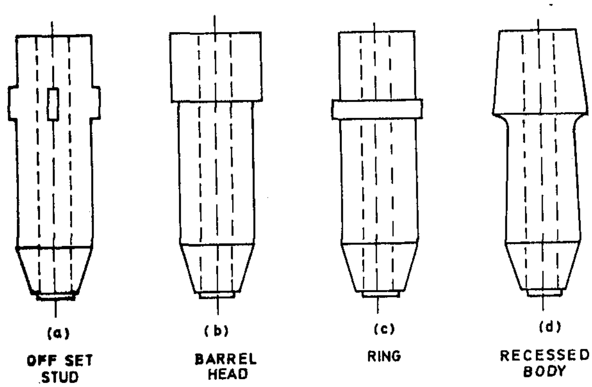
i) For using the mechanical cone, the friction sleeve of the test cone frequently becomes jammed with fine soil particles after long usage, due to infrequent cleaning and oiling or due to the clearance created by the sleeve sliding section which progressively gets worn. These soil particles soon become tightly packed and cause load transfer between the cone tip and sleeve.

11) Adhesion (skin friction) developed along the penetrometer rods by tropical and residual soils become so high that deep penetration is usually unacomplished. Normally, friction reducers are fitted above the cone assembly to minimise soil adhesion on the string of rods which pushes the cone to testing positions. The four commonly used type of friction reducers are shown in Figure 3. Efficiency of usage of these friction reducers vary with soil types and improper test termination has very often occurred due to wrongful usage of the elements. The 'Offset stud' type of friction reducer (Figure 3a) allowed farthest penetration in the tropical stiff sandy clay soils. The next efficient type of friction reducers is the 'Enlarged Head' type, (Figure 3b), then the 'Ring' type (Figure 3c), while the least effective in the residual and lateritic soils is the 'Recessed Body' type (Figure 3d), which is most easily worn-out by intense usage. However, this recessed body type friction has reducer proved most efficient in testing sedimentary sandy deposits.

iii) Besides the restricted penetration effect, misuse of these attachments also leads to erroneous penetration results. An example of such results is given in Figure 4 showing deeper penetration with stud type reducer than with recessed body type. Preboring and testing from bottom of boreholes can eliminate some of these problems.

iv) Infrequent recalibration of the pressure gauges used for resistance measurements leads to production of unreliable results.

For tests carried out with an electric or piezo-cone, the reliability of results, sensitivity of the equipment and operational cost are much greater than with a mechanical cone. However, it is appreciated that there are significant differences in the results from testing with electric or mechanical cone in similar soil deposits. Investigation of this phenomenon has not yet been reported for the Nigerian practice, owing to the relatively little usage of the electric cones to date. Some correlation between Standard Penetration Test (SPT), Cone Penetration Test (CPT) with mechanical cone and Laboratory Triaxial Compression Test for residual/lateritic deposits of Nigeria has been reported by Ajayi and Balogun (1988).



TYPES OF FRICTION REDUCERS

FIG. 3

FACTORY SITE AMUWO ODOFIN, LAGOS.

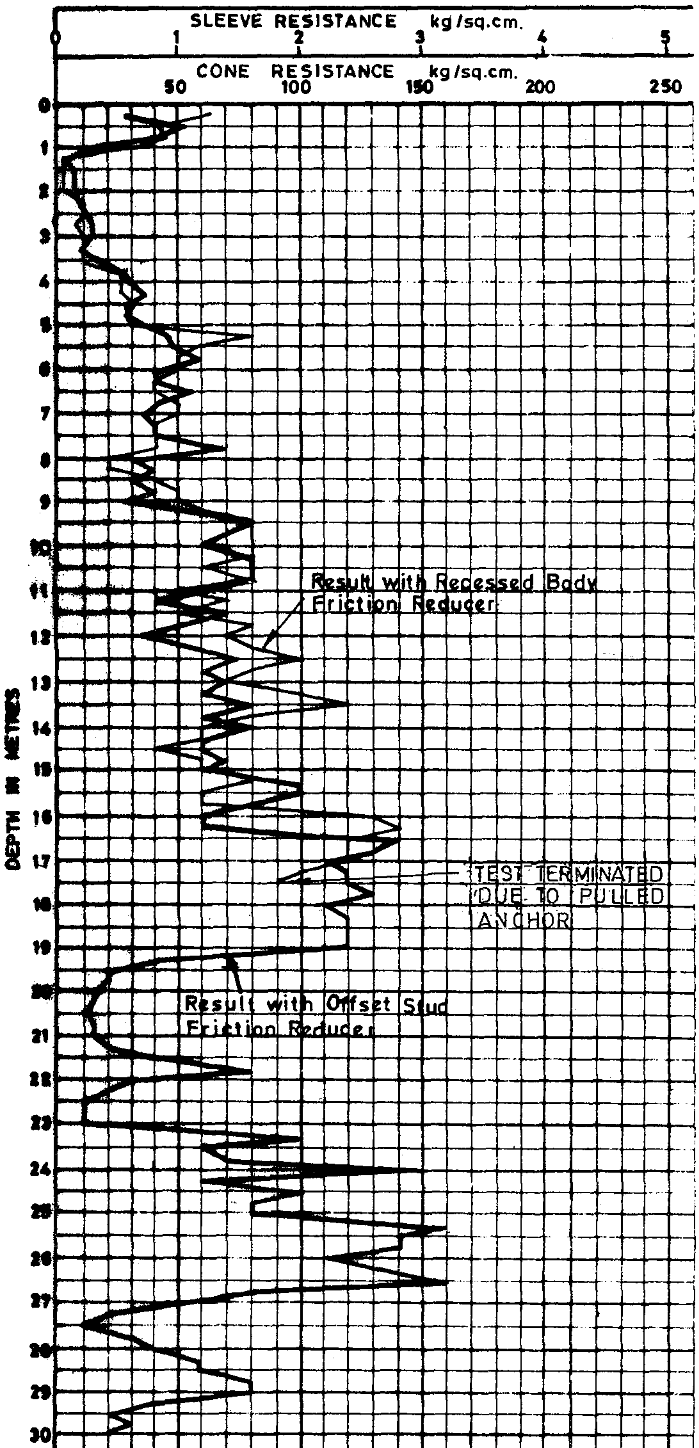


FIG 4

3.0 INTERPRETATION AND EVALUATION OF CONE PENETRATION TEST RESULTS

Interpretation of cone penetrometer test results is usually based on parameters and methods established in Europe - mainly by Dutch and British engineers, G. G. Mayerhof, C. Van der Veen, J. H. Schmertmann, J. B. Burland, M. J. Tomlinson and others. Cone penetration test results are usually applied to solving problems encountered in the design of shallow foundations, piled foundations, earth retaining structures and for the efficacy of soil improvement methods.

3.1. Soil Classification

When cone penetration tests are conducted with friction jacket cone, use is made of the f_s/q_c (fs = sleeve friction, q_c = cone resistance) for rough assessment of the soil types based on grain size distribution. However, soil classification obtained in this way is often combined with the results from geoetchnical borings and laboratory testing for proper determination of the subsoil stratigraphy. The following characteristic values have been proved to be valid for some soils in Nigeria.

Soil type	$\frac{fs}{q_c} \times 100$
Gravel and sand	< 2
Silty sand, clayey sand, silt	2 - 3.5
Clays	> 3.5

3.2 Shear Strength

Undrained shear strength of cohesive soils is normally estimated from the measured cone resistance q_c using the following equation:-

$$q_c = Cu.Nk + Po$$

Where Cu = the undrained shear strength

Nk = non - dimensional empirical cone factor

Po = overburden pressure

For practical purposes, the simplified relationship $Cu = q_c/Nk$ is very often adopted. Using a mechanical or mantle cone, Nk values varying between 12 and 20 have been confirmed for normally consolidated deposits encountered in the southern part of Nigeria. The values of the undrained shear strength for the comparison have been derived from unconsolidated undrained triaxial tests on mostly 38mm diameter samples.

The results of cone penetration tests in partly saturated tropical clays such as lateritic and residual sandy soils cannot yet be conveniently interpreted with a high degree of certainty. These soils are found to display comparatively high angles of internal friction and cohesion and normally do not fall within the ambit of fully saturated clays which the above correlation cater for. However the work of Ajayi and Balogun (1988) is an attempt at improvement of this lapse.

3.3. Shallow Foundations

The design of spread footing and rafts in Nigeria is primarily based on results from borings and laboratory testing. However, these results are sometimes supplemented by results from cone penetration tests. Where only the results of cone penetration soundings are available, the relationships proposed by **Meyerhof, Schmertmann, Tomlinson** and others are generally used.

For square of strip footings with widths not greater than 1.2m, the allowable bearing pressure is given by

$$q_a = 3.6\ q_c \quad \text{or} \quad q_c/30 \quad KN/m^2$$

Where q_c is measured in Kg/cm^2 with a mechanical cone.

An approximate equation to cover all foundations irrespective of the widths is

$$q_a = 2.7\ q_c \quad \text{or} \quad q_c/40 \quad KN/m^2$$

Where q_c is measured in Kg/cm^2 with a mechanical cone.

The calculated allowable bearing pressures are halved for clayey or silty sands or when the ground water level is encountered at or above the foundation level. For raft or pier foundation, the q_a may be doubled.

Local experience shows that for simple structures founded on clay, the allowable bearing pressure can be evaluated from the following relationship.

$$q_a = q_c/10\ KN/m^2$$

Where q_c is measured in Kg/cm^2 with a mechanical cone.

For a footing on sand, settlement can be directly obtained from the relationship.

$$S = Pn\ B/2q_c$$

Where Pn is the net applied load on the foundation

B is the foundation width

q_c is the average q_c value over a depth equal to footing width B.

3.4. Piled Foundations

The littoral soil conditions encountered generally in the southern part of Nigeria consist essentially of weak recent sediments of peats, clays and sands overlying overconsolidated sands. Ajayi (1989). In general, such soil profile favours the use of piled foundations for the support of most structures. Hence, most of the cone penetration tests conducted in Nigeria take place in this area.

The ultimate load carrying capacity of a pile (Q_u) is taken as the summation of the ultimate base resistance (Q_b) and the ultimate shaft resistance (Q_s)

$$Q_u = Q_b + Q_s$$

In sands, and considering parallel sided displacement (driven) piles, the base resistance $Q_b = q_b \cdot A_b$

Where

$$q_b = 0.5\left(\frac{A + B}{2} + C\right). Ap$$

A = Average N or q_c below pile base level to over a depth of 4 pile diameters.

B = Minimum N or q_c below pile base level to a depth of 4 pile diameters.

C = Average N or q_c above pile base level to a height of 8 pile diameters.

Ab = Area of pile base.

The ultimate shaft friction Q_s can be calculated from values of measured local side friction (f_s) or cone resistance (q_c), but it is preferable to use (q_c).

Using the local side friction (f_s) values,

$$Q_s = S_1 \int_0^L f_s . D.dL$$

Where

D is the diameter of pile

dL is the length of pile in the sand stratum

and S_1 is 0.7, irrespective of type of driven pile, if f_s value was measured with an electric cone. However, for f_s measured with a friction sleeved mechanical cone, S_1 for parallel sided concrete or steel driven pile with flat shoe is 0.3; with pointed shoe 0.55m;for 'Vibro' type pile 1.0 and for open ended steel tube or H pile 0.35.

Using cone resistance (q_c) values.

$$Q_s = S_2 \int_0^L q_c . D.dL$$

D is the diameter of pile

dL is the length of pile in the sand stratum

Again, values of S_2 depends on the type of cone used for measuring q_c. If electric cone was used, S_2 for parallel sided precast concrete or steel displacement pile is 0.012; for precast concrete pile with an enlarged base and pile in a dense group 0.018 and for open ended steel tube 0.008. If q_c was measured with a friction sleeve mechanical cone, the above values of S_2 should be halved.

For a bored pile, the values of Qu obtained above are halved.
A factor of safety of 2.5 or 3 is normally used to determine the pile working load.

4.0 FUTURE DEVELOPMENTS

Cone penetration tests are considered extremely useful for ground investigation, particularly, when combined with other methods of ground exploration. The Dutch types of penetrometers are the ones mostly used in Nigeria. As the local manufacturing capabilities are yet inadequate, we will still depend on developments in techniques and equipment emerging from Europe. However, our engineers are working hard at developing parameters appropriate for our types of tropical deposits.

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Cone Penetration Testing - CPT'95 National report for Norway

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SYNOPSIS: Since the first CPTs were carried out in Norway in 1953, the test was only used on a few occasions for many years. Over the last 4-5 years, the CPT/CPTU has become a more important part of onshore soil investigations, and the use of the method is growing steadily. CPT has also been an indispensible part of offshore soil investigations in the North Sea for almost 25 years. A lot of research has been carried out, resulting in a high standard of testing and advanced use of CPT/CPTU results for foundation design.

1. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION

1.1. General background

In total, only about 25% of the land area in Norway is covered by soil deposits. The major part is dominated by bedrock, or just a very thin cover of morainic materials, normally not representing serious geotechnical challenges. Areas covered by deep soil deposits are particularly concentrated in the lowlands, in the bottom of the valleys and in a narrow strip along parts of the coastline. The geological origin of these deposits is mostly related to the Quaternary period, covering the last 2 million years of the geological history. However, recent processes like landslides, erosion and weathering have many places influenced the local geology considerably.

During the Quaternary period, the climate was generally very cold and glaciers covered large land areas. This last ice-age represented several sequences where the Scandinavian peninsula was covered by a massive ice-cap. However, a number of warm interglacial periods caused the ice-cap to retreat temporarily. This period of time, rich of climatic fluctuations, represents the most significant influence on the Norwegian geology, both with respect to the type and structure of deposited materials.

The ice-cap was at its maximum extension about 20 000 bp, compressing the land masses to a level several hundred meters below the present. The movement of the glaciers scoured the rock surface, and transported eroded material in front, beneath and incorporated in the ice mass.

About 10 000 years ago, a warmer period set in and the glaciers started to retreat permanently. During deglaciation, large rivers of meltwater from the retreating icecap deposited the glacially transported material into the sea. The sea level in the deglaciation period was significantly higher than present, reaching about 180-220 masl in some regions in the eastern and central parts of Norway. At the same time, the land masses started to heave, explaining the presence of marine sediments on dry land today. Land areas below this marine limit comprise large parts of the populated areas in Norway.

1.2 General geology below the marine limit

The geology below the marine limit is generally dominated by the sedimentological processes taking place in a marine environment. At some locations, glacifluvial deltas have been formed, mainly containing gravel and sand, but also stratas of finer material. Clays and silts were deposited in the sea at some distance from the retreating glacier, and these soils dominate large areas below the marine limit.

Glacial tills of variable thickness usually cover the bedrock, containing all soil fractions from clay to block.

1.3 General geology above the marine limit

The thickness of the soil deposits above the marine limit is generally less and more patchy than in the lowlands. The geology in these areas is dominated by glacial tills, glacifluvial sediments or dead-ice deposits from a stagnating glacier. Lacustrine clays and silts are scarce, but can be found in or near existing lakes or previous glacial lakes.

A typical soil profile above the marine limit may be described as follows:

- Top layer of organic matter, slide debris or other mixed materials.
- Alluvial gravels and sands.
- Lacustrine or sediments deposited in glacial lakes, mainly consisting of silts and sands.
- Moraines and/or glacifluvial sediments overlaying the bedrock.

1.4 Geology on the Continental Shelf

The glacier that once covered the Scandinavian peninsula reached the bank areas of the North Sea at its peak, whereas Great Britain at the time was covered by another glacier. Between these two glaciers, a separate ice cap was formed, covering the central parts of the North Sea. Before the ice age, sedimentation of fine particles from the major rivers of the European Continent took place in this area.

The geological history of the northern parts of our Continental Shelf is even more complex. This part of the shelf was covered by ice several times, causing re-glaciation and overconsolidation of previously deposited materials.

Over large parts of the Norwegian Continental Shelf the bedrock is hence covered by Quaternary deposits, ranging in thickness from a patchy cover to more than 300 m thick sediments. The thicker deposits are usually found in the channels and depressions, such as the Norwegian Trench with water depths exceeding 300 m. The soil cover in the shallow bank areas is generally much thinner.

The deposits can be separated into several units, where the most recent were deposited about 13000 - 10000 bp. They occur as complex linear belts running mainly parallel to the present coastline. Generally, the stratigraphy includes a top layer of sand, overlaying marine and/or glacio-marine clays. However, in the deep waters of the Norwegian Trench, clays and silts may be encountered from the very surface.

1.5 Geotechnical properties

Coarse grained soils

Norwegian sand and gravel deposits usually contain equal portions of quartz and feldspar, sometimes representing as much as 80 to 90 % of free mineral grains. Typical geotechnical properties of our quartz-rich sands are summarized in the following:

unit weight:	γ	18-21 kN/m^3
unit weight of solids:	γ_s	26,0-27,0 kN/m^3
max. porosity:	n_{max}	45-50 %
min. porosity:	n_{min}	30-38 %
cohesion:	c	0-10 kPa
friction:	$\tan\phi$	0.6-1.1
modulus number:	m	150-850

The offshore sand deposits tend to be much denser than the onshore deposits.

Fine-grained soils

Norwegian clays generally consist of low-activity clay minerals such as vermiculite and chlorite, in addition to minerals like feldspar and muscovite.

Some of the marine, fine-grained deposits have been overconsolidated due to ice loading, removal of previous overburden, capillary effects or a lowered groundwater table. The soft clays and silts show a small apparent overconsolidation, probably due to aging or creep effects.

Typical ranges in properties for Norwegian clays and fine silts are shown below:

clay fraction:	< 2 μm	25-60 %
void ratio:	e	0,6-1,5
water content:	w	25-55 %
liquid limit:	w_l	35-45 %
plastic limit:	w_p	17-27 %
plasticity index:	I_p	10-35 %

unit weight:	γ	16-20 kN/m^3
unit weight of solids:	γ_s	26,5-28,5 kN/m^3
sensitivity:	S_t	> 5
undrained shear strength:	s_u	10-150 kPa
cohesion:	c	5-50 kPa
friction:	tanϕ	0.4-0.7
modulus number:	m	15-50
coefficient of consolidation	c_v	5 - 100 m^2/yr

The detection of possible *quick clay* deposits is usually on the agenda when performing site investigations in fine-grained soils. This clay liquefy completely when remoulded, and the debris from a quick clay slide may overflood large areas. The undisturbed strength may however be considerable, which means that the sensitivity may be very high (100 - 1000). This behaviour can be explained by the open grain structure in a marine clay, established during sedimentation. The structure turns into a labile "cardhouse-structure" if salt ions are removed from the porewater by long-term leaching processes or groundwater transport.

With soil conditions ranging from soft quick clays to very stiff moraines, it is necessary to apply different geotechnical field equipment to remedy an optimal site investigation. The variations in geological conditions also represent given restrictions for general use of some types of field equipment.

2. TYPES OF SOUNDING AND PENETRATION TESTS

In Norwegian field investigation practice, it is convenient to distinguish between *onshore* and *offshore* testing, since the two types of investigations involve very unlike field operations carried out by different companies. In the following, emphasis will be put on in situ tests carried out onshore.

2.1 Onshore testing

Rotary weight sounding

In the test, the rod system with a screw point is rotated by hand or machine, carrying a weight varying from 100 kg (starting value) to 25 kg (soft conditions with the rod system sinking). The number of half turns per 0,2 m penetration is recorded, and expresses the variation in soil stiffness in the profile. This method is now mostly used on small projects, where access by a drill rig is difficult or too costly for the project.

Rotary pressure sounding

The rotary pressure sounding was developed by the Norwegiab Geotechnical Institute (NGI) and the Norwegian Road Research Laboratory (NRRL) in 1967. The aim was to develop a rational and efficient sounding method, adjusted for Norwegian soil conditions, climate and terrain. The Norwegian Geotechnical Society (NGF) has issued guidelines for performance of the method.

The equipment is operated by a multipurpose drilling rig, with principle of operation as shown in Fig. 2.1. The equipment consists of a drill bit extended by rods with flush couplings. It is forced into the ground at a constant rate of penetration (3m/min) and a constant speed of rotation (25 rpm). The thrust necessary to maintain the penetration according to these requirements is measured and plotted versus depth.

After more than 20 years of experience, it may be concluded that the aim to a great extent has been achieved, and the method is now used extensively all over Norway (Rygg and Andresen (1988)). The method is often used in preliminary or initial investigations, where a grid of rotary-pressure soundings usually are the first investigations to be carried out. Based on results from these tests, the optimal location of sample boreholes and in situ tests can be determined.

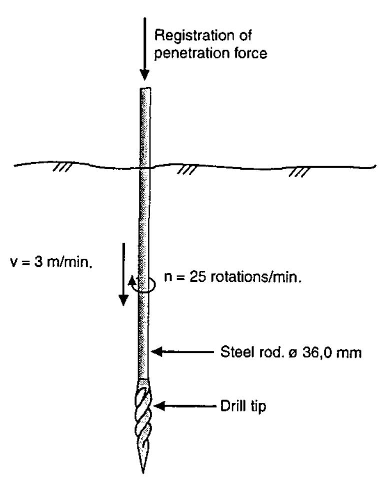

Fig.2.1. Principle of operation for rotary pressure sounding.

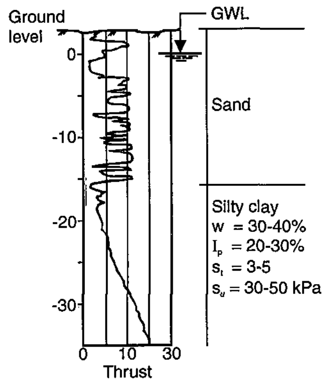

Fig.2.2. Test profile from a rotary pressure sounding.

Fig.2.2 shows a test profile from a rotary pressure sounding in a layered deposit, which indicate that the results can be used to identify soil type and stratification.

Total sounding
Where the rotary-pressure sounding equipment cannot penetrate due to very hard layers, total sounding can be used. This sounding method combines the principles of rotary-pressure sounding and rock drilling into an effective method for mapping of soil conditions and determination of depth to bedrock. Total sounding is developed through cooperation between NGI and NRRL.

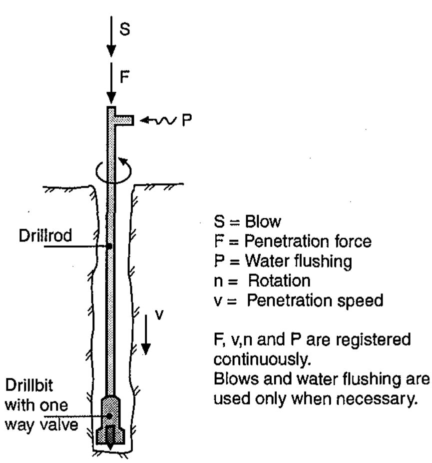

Fig.2.3. Test principles of total sounding.

When performing total sounding, the rig must be equipped with a hydraulic drilling system with sufficient capacity to supply a penetration force of 30 kN on the rod system. In additon, a percussion hammer and a flushing system may be activated for penetrating hard soil strata or boulders and rock. Fig.2.3 summarizes the principles and procedures used in total sounding.

The ϕ45 mm rod system is equipped with a ϕ57 mm drillbit with a spring loaded valve, which prevents clogging of the flushing system during penetration. When the flushing fluid is pressurized, the valve opens and drilling with flushing is continued, with or without additional activation of the percussion hammer. The drilling operations may be reverted to rotary pressure sounding procedures at any time during penetration.

Other penetration tests
Over the last 10 years, several other penetration test methods have been used for research projects in Norway, such as nuclear density probe, electrical resistivity probe, seismic cone, dilatometer and cone pressuremeter.

Other in situ tests
The vane test has been used extensively in soft clay, but the number of tests has recently been reduced since the use of piezocone tests have become more common. In addition, various pressuremeter concepts and the screw plate have been used for research purposes.

2.2 Offshore testing

Due to space limitations, reference will be made to Lunne and Powell (1992), with only a short summary included herein.

Penetration procedures
CPT is carried out either from seabed rigs, mostly giving continuous penetration up to 40-50 m; or by using wireline operated tools with 3 m strokes below the drillbit.

In situ tools
CPT/CPTU constitute more than 90% of the in situ testing. Other equipment used occasionally include offshore vane, push-in-pressuremeter, BAT probe, dilatometer, seismic cone and hydraulic fracture test.

3. TYPE OF CPT EQUIPMENT USED

Since the first offshore CPTs were carried out in the North Sea in 1972, CPT/CPTU has played a very important role. Onshore CPTs have been carried out occasionally since 1953, with increased use in commercial soil investigations over the last 4-5 years.
The following sections concentrate on equipment used in onshore practice. For a more detailed review of offshore practice, reference is made to Lunne and Powell (1992).

3.1 Cone penetrometers and rigs

Due to a very price-competitive market, almost all CPT operations in commercial projects are handled by one person. This is possible mainly due to the use of cable-less CPT systems, which recently have become very popular in Norway.

Rigs

In Norway, CPT/CPTUs are done from hydraulic drill rigs since the use of special CPT trucks are inconvenient with our topography. These rigs push about 1 m in each stroke before a new rod has to be added. NTH uses a tailor-made CPT-rig with continuous pushing of the rod system.

Penetrometers

Two types of Swedish cone penetrometer systems dominate the Norwegian market:

ENVI Memocone

This equipment consists of the Memocone penetrometer itself and the corresponding Geoprinter data aquisition unit. The system is cableless, with the data acquisition based on syncronized clocks within the Geoprinter and the Memocone.

After the completion of the profile, the penetrometer is coupled to the Geoprinter and recordings of q_c, f_s and u are stored and plotted for every 2 cm penetration. Zero readings are registered at the end of data transmission. A drawback with this equipment is that no recordings from the cone can be seen while the test is in progress.

Another special feature of the ENVI piezocone is that no filter is used. A steel ring just behind the cone replaces the filter and provides a 0.3 mm opening slot into the cone. The cavities of the cone are saturated with grease or similar, whereas the pore pressure chamber is filled with de-aired water or antifreeze liquid. This procedure is very quick and efficient, and parallel tests with filter piezocones have shown good comparison between measured pore pressures.

GEOTECH cordless cone penetrometer.

With this system, data measured at the probe is transformed into a sound signal via a microprocessor. This signal is transmitted through the CPT rods to a microphone which is located between the top rod and the drill rig. The data is then transferred through a cable from the microphone to the Geologg CPT interface. The same interface is also fed with the exact penetration depth from an optoelectric sensor. Measured CPT data are displayed real time during the test, which is a large advantage compared to the memory type cone penetrometers. In this system, pore pressures are measured just behind the cone.

Other types of CPT/CPTU equipment, such as Fugro penetrometers, are also available, but they are mainly used for offshore investigations and onshore research purposes.

3.2 Calibration and presentation of data

The Norwegian Geotechnical Society issued in 1994 guidelines for use of CPT/CPTU. These guidelines include detailed requirements for important features such as calibration of sensors, accuracy of measurements and presentation of test results. The Norwegian guidelines are based on the ISSMFE internationally recommended procedures from 1989. They also resemble very close to the guidelines presented by the Swedish Geotechnical Society in 1993.

4. INTERPRETATION OF TEST RESULTS

In Norway, research on interpretation of CPTU results has mainly been carried out at the Norwegian Institute of Technology (NTH), Norwegian Geotechnical Institute (NGI) and the Norwegian Road Research Laboratory (NRRL). Since the first piezocone tests were carried out in Norway in the beginning of the 1970's (Janbu and Senneset (1974)), research has concentrated on the development of rational interpretation methods for various soil types, based on well-known, basic theoretical principles. Systematic correlations between reference parameter values from other in situ and laboratory tests have also been made. The aim has been to add empirical modifications to the interpretation methods, or establish direct empirical correlations between CPTU results and various soil parameters. Research has also focused on the use of CPTU data directly in geotechnical design (e.g. Lunne et al (1989)).

A brief description of interpretation methods used in Norway is given in the following sections.

4.1 Soil stratification and identification

Stratification

To elucidate soil stratification, the following measured and derived parameters are evaluated:

Corrected cone resistance: q_t
Net corrected cone resistance: $q_n=q_t-\gamma z$
Recorded skin friction: f_s
Total or excess pore pressures: u, Δu
Friction ratio: R_f
Cone resistance number: $N_m = q_n/(\sigma_{vo}'+a)$
Pore pressure ratio: $B_q = \Delta u/q_n$

The parameters are usually plotted versus the penetration depth z.

Soil identification

In Norway, a soil identification chart presented by Senneset et al (1982) and later modified by Senneset et al (1989) is used quite a lot. This chart uses q_t and B_q as input parameters, and has been modified to accomodate negative excess pore pressures often recorded in stiff, dilative soils.

For offshore application the chart proposed by Robertson et al (1986) is used. This classification diagram incorporates q_t, B_q and R_f. The diagram has later been modified by Robertson et al (1990), using normalized input parameters.

Even if the proposed classification charts have general application, local soil conditions such as fissuring, stress history, mineralogy and organic content may influence the soil behaviour considerably, even if the composition may be similar. Soil identification charts hence provide an indication of soil behaviour, rather than the actual grain size distribution of the penetrated soils.

4.2 Interpretation of soil parameters

Relative density

The correlations between q_c, σ_{vo} and D_r used in Norway are mainly based on calibration chamber tests carried out at NGI (Lunne and Christophersen (1983)) and in Italy (Baldi et al (1986)). Values of D_r may subsequently be used to interprete friction and liquefaction potential by utilizing empirical relationships. The interpreted relative density is used to reconstitute samples in the laboratory. Static and cyclic soil design parameters can then be found from these laboratory tests.

Evaluation of stress history

Knowledge of the preconsolidation stress σ_c' is important in Norway, since a large part of our soil deposits are overconsolidated (see Section 1.5). Oedometer tests on undisturbed soil samples are still the best way to determine the preconsolidation stress. However, undisturbed samples may be difficult to get in many soil types. In this case, in situ methods such as CPTU may be a prudent alternative for evaluation of stress history.

For a normally consolidated clay, the theoretical cone resistance may be written (Sandven (1990)):

$$q_T = K_c \gamma z \tag{4.1}$$

where:
q_T: theoretical cone resistance
K_c: $N_c \alpha_u \gamma'/(\gamma+1)$ ($\approx$ 1,75-2,25)
N_c: bearing capacity factor ($\approx$ 6-12)
α_u: 0,25-0,30 ($\approx \frac{1}{2}\sin\phi$)
γ: unit weight of soil
z: penetration depth

If the corrected cone resistance q_t is considerably larger than the theoretical value given by Eq.4.1, the clay may be overconsolidated.

Several approaches are used in Norway to assess values of the preconsolidation stress directly from CPTU data:

- approaches based on empirical relationships between preconsolidation stress and recorded pore pressures, cone resistance and skin friction, see summary in Lunne et al (1989). Based on experience, no unique correlations seem to be valid for all soils since the parameters are influenced by many other factors than the stress history. However, good local correlations have been obtained.

- interpretation methods introducing semi-theoretical approaches (e.g. Senneset et al (1989), Sandven (1990)).

Undrained shear strength (clays)
Assessing values of the undrained shear strength is not a simple task since this parameter is influenced by factors such as anisotropy, stress history, rate of straining and mode of failure.

In the interpretation of s_u from CPTU records, empirical correlations are still the most popular approach, see for example Lunne et al (1985, 1989). These correlations are carried out by first assessing the empirical cone factor N_{KT} from the expression:

$$N_{KT} = (q_t - \gamma z)/s_u \tag{4.2}$$

A reference s_u- value should then be found from other relevant in situ or laboratory tests in selected profiles at the site. After assessing a local value of N_{KT}, this value is used in subsequent interpretation of the CPTU results. If no reference values for s_u exist, $N_{KT} = 15$ is usually recommended.

Various analytical approaches, such as bearing capacity and cavity expansion theories, have also been used to determine a theoretical bearing capacity factor N_c, replacing N_{kT} in Eq.4.2. A range of N_c-values between 6 and 12, depending on type of approach and boundary conditions have been reported.

By the use of cavity expansion theory, the undrained shear strength can also be expressed by:

$$s_u = \Delta u/N_{\Delta u} \tag{4.3}$$

where the bearing capacity factor $N_{\Delta u}$ theoretically may vary between 2 and 20, with 6 - 10 being an estimated range for most Norwegian clays (Lunne et al (1989)). This approach has gained increasing popularity in Norway, particularly in soft clays.

In any correlations, it is very important to use consistent reference values of s_u. It is recommended that CAU_C triaxial tests should be used on high quality samples. Recent experience shows that block samples (Sherbrooke type) give superior quality, particularly for silty clays. A new generation of correlations based on laboratory tests on these samples are presently being developed at NGI.

Effective stress strength parameters
The framework for interpretation of effective stress strength parameters has been the conventional bearing capacity approach. This approach has mainly been used for drained penetration in sands (e.g. Janbu & Senneset (1974), Lunne & Christophersen (1983)), but the interpretation method has also been extended to represent undrained penetration (Senneset et al (1982, 1989), Sandven (1990)).

The interpretation method is based on the relationship:

$$q_n = N_m(\sigma_{vo}' + a) \tag{4.4}$$

where the friction $\tan\phi$ is found numerically when the cone resistance number N_m and the pore pressure ratio B_q are known from the obtained CPTU records. The values of attraction a (= $c/\tan\phi$) and the angle of plastification β influence the interpretation and have to be assessed (e.g. Sandven (1990)). The friction angle interpreted from this method refer to values obtained by CIU_c or CAU_c triaxial tests. This method is used frequently in Norway, but further research is necessary to validate the method in various soil types.

Constrained modulus
Most correlations between CPT results and the constrained deformation modulus refer to the tangent modulus M found from an oedometer test. However, due to the large strains imposed on the soil during penetration, there is no established analytical solution relating CPT data to any kind of deformation modulus. In the interpretation, values of M are hence usually estimated from empirically based relationship of the type $M = mq_n^{\alpha}$ (Senneset et al (1982), Sandven (1990)). The stress exponent α equals

1,0 for clays and fine silts, whereas 0,5 (square-root adaption) is more appropriate for sands. In clays, m may vary between 5 and 15, depending on the stress level the modulus value is referring to, whereas 20 - 60 is a more relevant range in sands and coarse silts.

Based on calibration chamber tests, Lunne and Christophersen (1983) proposed some simple correlations for a conservative estimate of M in sands, which are much used in practice.

Small strain shear modulus, G_{max}

The small strain shear modulus G_{max} can be found from CPTU, either by finding the shear wave velocity from a seismic cone test or by empirical relationships between the initial shear modulus G_{max} and factors such as cone resistance and relative density (e.g. Baldi et al (1989)). Some work has been done within this field in Norway, particularly in sandy and silty soils. Lunne et al (1989) showed results from Holmen sand in Drammen, where G_{max} was determined from measurements by seismic cone and crosshole methods and compared to estimated values from empirically based interpretation charts.

Coefficient of consolidation

The most used interpretation approaches are summarized by Senneset et al (1989) and Lunne et al (1989). A recent simplified chart given by Robertson et al (1992) is also much used in practice. The various approaches often give one order of magnitude difference in interpreted values. This discrepancy may be explained by the complexity of the cone penetration process, and the many factors influencing the results.

5. USE OF CPT DATA IN PRACTICAL DESIGN

Over the last 20 years, CPT/CPTU results have been an essential input for foundation design offshore. For onshore applications it is mainly over the last 4-5 years that this test has played a significant role. In the following, two examples from offshore applications will be presented.

Pipeline engineering

In connection with transport of petroleum from the Norwegian sector of the North Sea to UK, Germany and Holland, several pipelines have been installed. Some of them are more than 800 km long, covering soil conditions from very dense sand to very soft clays.

In the design of these pipelines, it is necessary to obtain soil parameters in the upper 2-3 m for evaluation of pipe-soil interaction. To provide information about the soil conditions, CPTUs are carried out with internal distances from about 50 m to 4 km, depending on factors such as soil conditions, closeness to the shore line and trenching requirements. In addition, samples are taken by vibro- and gravity corers for every 2-4 CPTU. Information of soil layering and design parameters are then mainly selected from the CPTU results, following the procedures outlined in Chapter 4. This approach has proved to be a very efficient and economical way of assessing the soil conditions.

Foundation of platforms

Foundation of large offshore gravity base structures (GBS) has required particularly detailed knowledge of stratigraphy and soil behaviour under static and cyclic loading. Within this context, CPTU results have provided essential information for the following purposes:

- detailed layering across the platform area.
- penetration resistance of skirts beneath the GBS during installation.
- undrained shear strength of clays and drained shear strength profiles in sand have been established from a combination of CPTU and laboratory test results.
- for sands, samples have been reconstituted in the laboratory to densities found from CPTU results. Advanced cyclic/dynamic triaxial and direct simple shear tests have been carried out on these samples.
- settlement parameters in sand.
- distance between draining layers in clay
- evaluation of liquefaction potential in loose, sandy deposits.

For piled platforms, bearing capacity has been evaluated directly from CPTU records in addition to other methods.

6. COMPARISON AND CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

The overview below summarizes the status of onshore field investigation methods used in Norway:

Sounding methods

The simpler sounding methods such as rotary pressure sounding, rotary weight sounding and the more complex total sounding dominate (see Chapter 3). None of these methods provide the same soil mapping ability as CPTU, but they are cheaper and represent less challenge to the operator. Fig.6.1 shows an example of the difference between a profile obtained by CPTU and a neighbouring profile from a rotary pressure sounding. The two profiles clearly demonstrate the superior ability of CPTU to reveal even thin layers of contrasting materials.

In situ tests

The field vane test is a a cheap and rational method to find undrained shear strength and sensitivity in clays. Despite several drawbacks, vane testing is a better tool than CPTU for evaluation of soil sensitivity. The method has been widely used in Norway, and will continue to be an important in situ method. However, the use of vane testing is believed to take a declining curve in future Norwegian practice as the use of CPTU is increasing.

Soil sampling

The aim of soil sampling is to obtain undisturbed or representative samples for soil classification and determination of engineering parameters. The following sampling methods are normally used:

Undisturbed sampling (clays, silts):
- ϕ54 or ϕ95 mm piston samplers
- block samplers

Representative sampling (all soils):
- simple samplers (e.g.augers)
- tube samplers
- driven samplers

Laboratory investigations

The results from laboratory investigations on undisturbed soil samples often provide reference parameters for local or general correlations to CPTU interpretations. The need of relevant laboratory tests as a background for evaluation and calibration of in situ test results is obvious. The number of laboratory tests may however be reduced in a project, through carefully planned in situ testing.

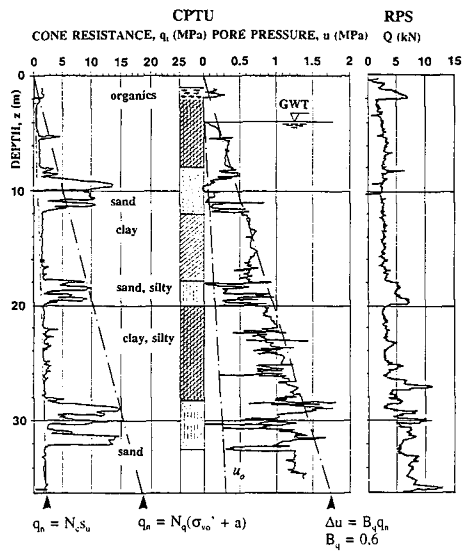

Fig.6.1. Comparison between profiles obtained by CPTU and rotary pressure sounding in a stratified deposit.

In Norway, a common laboratory test program includes the traditional index tests for identification and classification of soil type. Laboratory tests for determination of design parameters usually involve incremental and/or continuous loading oedometer tests, compression or extension triaxial tests, direct simple shear and shear box tests. Advanced tests for analyzes of special problems may include piezoceremic bender element and resonant column test (shear modulus G_{max}), oedotriaxial and split-ring oedometer tests (at rest coefficient K_0') and permeameter tests (coefficient of permeability k).

When comparing results, one should keep in mind the often considerable differences between the test conditions for an advancing cone, and the procedures followed for a reference laboratory test. The laboratory test procedures are often simplified and idealized to obtain rational and smooth test routines, and may not adequately resemble the soil behaviour around a penetrating cone.

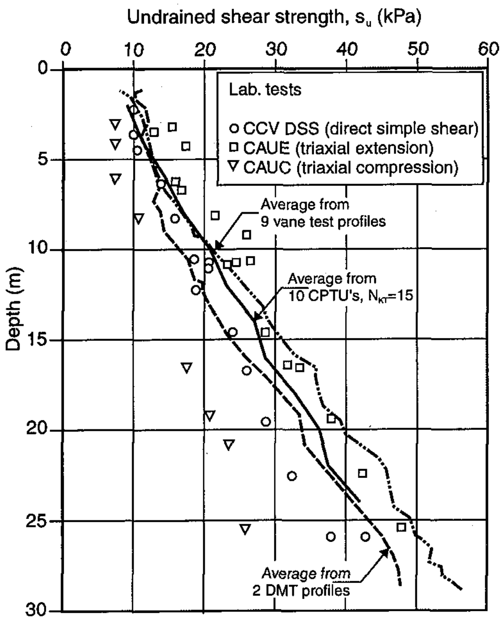

Fig.6.2. Comparison between different methods of s_u-determination in Onsøy clay.

In Fig.6.2, different methods of s_u-determination in a clay have been compared.

7. FURTHER DEVELOPMENTS AND TRENDS

In the years to come, further development of CPTU is believed to take place in Norway. The most important trends seem to be:

- economical restrictions will lead to more use of rational in situ methods like CPTU also in onshore projects, reducing the need for sampling and laboratory testing.
- recently published national guidelines (NGF (1994)) will lead to an improved and more uniform field performance.
- more emphasis on formal education of field personnel will improve the quality of CPTU
- improved and recommended procedures for interpretation and evaluation of CPTU data is expected to develop the applicability of the test results even further.
- courses and workshops for practicing engineers on use and interpretation of CPTU results will be arranged.
- increasing use of data methods for presentation (GIS) and interpretation of CPTU results.

The extended use of CPTU in a variety of soil types, creates a very good basis for evaluation of present knowledge on interpretation and use of CPTU results. Future research efforts should concentrate on further improvements of todays framework of interpretation, along with further refinement of equipment and test procedures. Some topics where future research acivities are desired are presented below:

- further development of multi-purpose CPTU equipment with a variety of different sensors incorporated in the penetrometer (e.g. seismic element, earth pressure cell, pore pressures at different locations, chemical sensors etc.).
- improvement of existing interpretation methods, boosted by extended use and an increasing database.
- consolidate national research sites where testing of new (CPTU) equipment can be carried out under controlled and well-documented conditions:
  * establish reference CPTU profiles for comparison and testing of new equipment/procedures.
  * reference background for correlations to data from laboratory or other in situ tests.
  * establish more consistent reference parameters for validation of CPTU interpretations, through improved field and laboratory methods.
- improve accuracy for available CPTU equipment.
- documentation of new test procedures, such as alternative saturation techniques (e.g. Envicone system).
- increased use of refined numerical tools (FEM) along with relevant soil models for more fundamental analyses of the cone penetration process.
- development of "expert systems" based on advanced data methods, geo-statistics and presentation techniques.
- use of miniature piezocones in various soil types.
- use of cone penetrometers for environmental soil investigations.

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STATE-OF-PRACTICE ON CPT IN ROMANIA (NATIONAL REPORT)

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SYNOPSIS: The 30 years or more of CPT use allowed to set up some statistic correlations between the penetration data and the geotechnical characteristics of the normally consolidated formations, on the basis of comparisons with the laboratory tests and some in situ tests (PLT), as well as with the measured settlements of constructions. The most stable correlations are the ones determined for the density index I_D and the modulus of linear deformation E for sands, with E and E_u (undrained modulus) respectively for clays and for the bearing capacity of the driven piles. The CPT proved to be useful when checking the improving (the increase of the dry density ρ_d) of loessial, collapsible soils.

1. BRIEF GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF THE REGION

Mountain area excluded, the ground conditions on the Romanian territory, up to the depth influenced by construction works, are characterized by the presence of material of recent age (Pliocen and Quaternary). As a result, terrace and alluvium deposits are frequently met in built areas: normally consolidated or slightly overconsolidated clays, medium dense or dense sands and gravels. Beside this good soil conditions, there are several types of soils which pose special problems (Fig.1):

- silt-sized,collapsible soils (loess) which cover about 40,000 km^2 (17% of the country's territory) mostly in the plain and plateau zones, with great density of buildings, transportation works, irrigation systems etc.;
- unconsolidated recent alluvium deposits, in the lower course of rivers and along the Danube;
- recent, unconsolidated fills, especially around large cities and industrial developments.

2. TYPE OF PENETRATION TESTING AND OTHER INVESTIGATION METHODS USED IN ROMANIA

The SPT and the Plate Loading Test (PLT) were for many years the only methods of in situ investigation of the geotechnical characteristics in Romania.

Since 1960, the CPT, the Dynamic Penetration Test (DPT) as well as the Ménard Pressuremeter Test and the Vane-Test have been introduced.

Due to the rather low cost, the dynamic penetration devices have extended ever more of late. In Romania, 3 types of Dynamic Probing are used: Light (DPTL), Medium (DPTM) and Heavy (DPTH), whose operation characteristics are identical to the ones recommended in the DIN 4094 (December 1990). With a view of reducing the friction effect on inner pressure rods, outer sounding tubes were placed on the dynamic penetrometer rods. Furthermore, in the soft soils, the vibropenetrometer is used (which is a dynamic penetrometer equipped with a vibrator) measuring the penetration velocity of the cone into the soil on 20 cm sections.

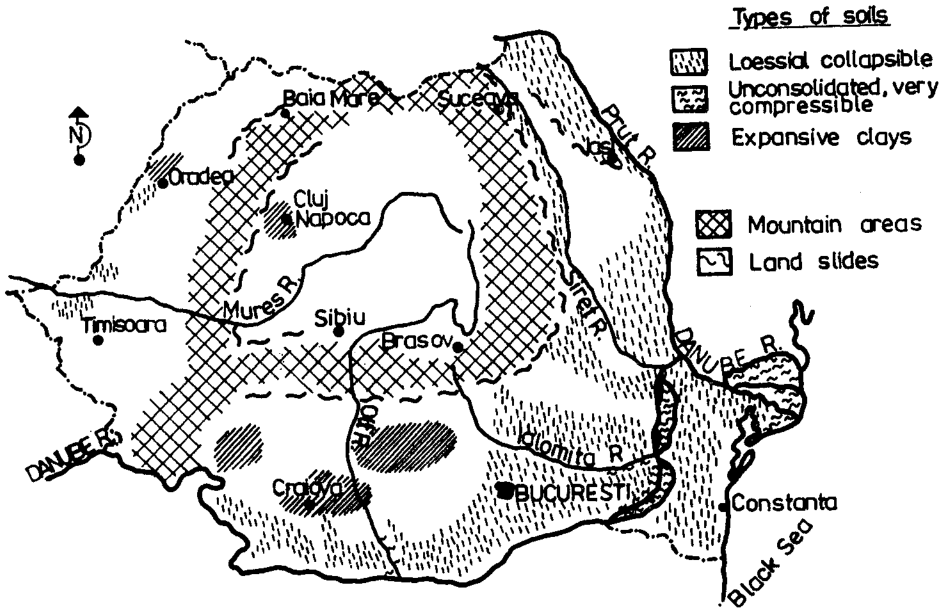


Figure 1. Difficult soil conditions encountered on Romanian territory

3. TYPE OF CPT EQUIPMENT USED IN ROMANIA

Only the mechanical Dutch cone penetrometer is used, in 2 variations: mantle cone and friction jacket cone. The maximum loading capacity of the penetrometers commonly used is 100 kN; however, there is a limited number of devices with the maximum thrust of 200 kN.

The devices use, as a rule, screw pickets; there are also 100 kN penetrometers on heavy trucks, whose weight serves as kentledge during the test. Since the force measuring devices (the hydraulic load cells) are placed in the upper part of the apparatus, the penetrometer types with continuous penetration are comfronted with problems on evaluating the frictions on the inner pressure rod that activates the cone. That is why, the clearest results are the ones obtained on penetrometers provided with friction jacket and on which the alternative advance of the cone, friction jacket and sounding tube is achieved. By comparing the forces necessary to separately thrust the cone, the friction jacket and the sounding tube, with the force in the whole device thrusting stage, the corrections related to the device internal friction are deduced.

The main operation parameters of the static penetrometers are regulated by the Romanian Standard STAS 1242/6-76, and they are identical to the ones used around the world for the Dutch cone penetrometer, and namely:

- cone base diameter, $d_c = 35.6$ mm
- angle at cone peak, $\alpha = 60^{\circ}$
- sounding tubes diameter, $d_t = 36$ mm
- rate of penetration, $v = 0.5 \div 2$ cm/s.

In the Romanian Code of Practice C159 (1989), the standard provisions are detailed and recommendations are made regarding the interpretation of the static penetration results, which are presented in the next chapter.

4. INTERPRETATION OF TEST RESULTS

4.1. Tests in sands

Because of the difficulties in obtaining undisturbed samples from sand, the CPT was frequently used to determine the relative density and the mechanical characteristics (especially the compressibility ones).

Based on the comparison with direct tests carried out in several types of quartzitic or limey

Table 1. Evaluation of the density index I_D based on the q_c values

Sand	Depth (m)	$I_D < 0.33$	I_D 0.33...0.67	$I_D > 0.67$
		q_c (MPa)		
Coarse	5	< 10	10 ... 15	> 15
	10	< 15	15 ... 22	> 22
Medium	5	< 6	6 ... 10	> 10
	10	< 9	9 ... 15	> 15
Fine	5	< 3	3 ... 6	> 6
	10	< 4	4 ... 9	> 9

sands, the intervals of q_c values were established for different intervals of density index, as shown in Table 1 and which are recommended in Code C159 (1989).

The modulus of linear deformation E was determined by PLT (Marcu & al., 1982) as well as by back computation using effęctively measured settlements on various structures.

Many CPT carried out in parallel with the PLT have shown a dependence close to the one recommended by Vesiç (1970):

$$E = 2(1 + I_D^2)\,q_c \qquad (1)$$

The settlements measured in sand layers under test loadings at large scale or under real constructions led, by back computation, to values of the E modulus closer to the correlation re- commended by Trofimenkov & Vorobkov (1974):

$$E = 3.4\,q + 13 \qquad \text{(MPa)} \qquad (2)$$

The test achieved by a large-sized loading (a reservoir), measuring the settlements into the depth of the sand layer illustrated in figure 2 allowed to determine, by back computation, the mean value of the modulus E=50 kPa (Stănculescu et al., 1980) which is close to the E value calculated by using the relation (2) for q_c^{mean} = = 12 MPa.

Similar results were obtained when interpreting the settlement measurements at *Intercontinental* Hotel of Bucharest (21 storeys, raft foundation on a medium, homogeneous, medium dense sand layer of 32m thickness) as shown in figure 3. When introducing in the settlement computation (by *« adjusted elasticity method »*) the E values based on the CPT and the relation (2), a 7.5 cm settlement resulted. Nevertheless, it should be mentioned that after the earthquake of March 4, 1977 (8.5 degrees on Mercalli scale, in Bucharest) the sand, in medium dense state suffered a supplementary settlement of more than 2 cm.

The possibility that supplementary settlements occur due to shoks and seismic actions led to the recommendation of Code C159 (1989) to use the conservative relation (1) for sands of loose or medium density ($I_D < 0.67$).

Such correlations which have a rather general validity, as well as a number of relations established at regional scale were used very much to the control of the compaction works in sands and sandy-silty soils carried out by using heavy hammers, or gravel columns (Stănculescu & al., 1980), or by deep vibration or vibroflotation (Păunescu & al., 1985).

4.2. Tests in cohesive soils

Strong statistic correlations were obtained between the q_c values and the *drained* modulus E determined by the PLT, with settlement stabilization under every loading step (Marcu, 1977). These correlations are recommended in the Code C159 (1989) in the following simplified forms:

$$E = 3.8\,q_c - 0.55\,n + 26 \qquad \text{(MPa)} \qquad (3)$$

$$E \approx 4.8\,q_c \qquad (3')$$

where:

E, q_c - in MPa

n - porosity (in %)

A high value of the correlation coefficient (r= = 0.869) was obtained between the values E_u of the undrained deformation modulus determined

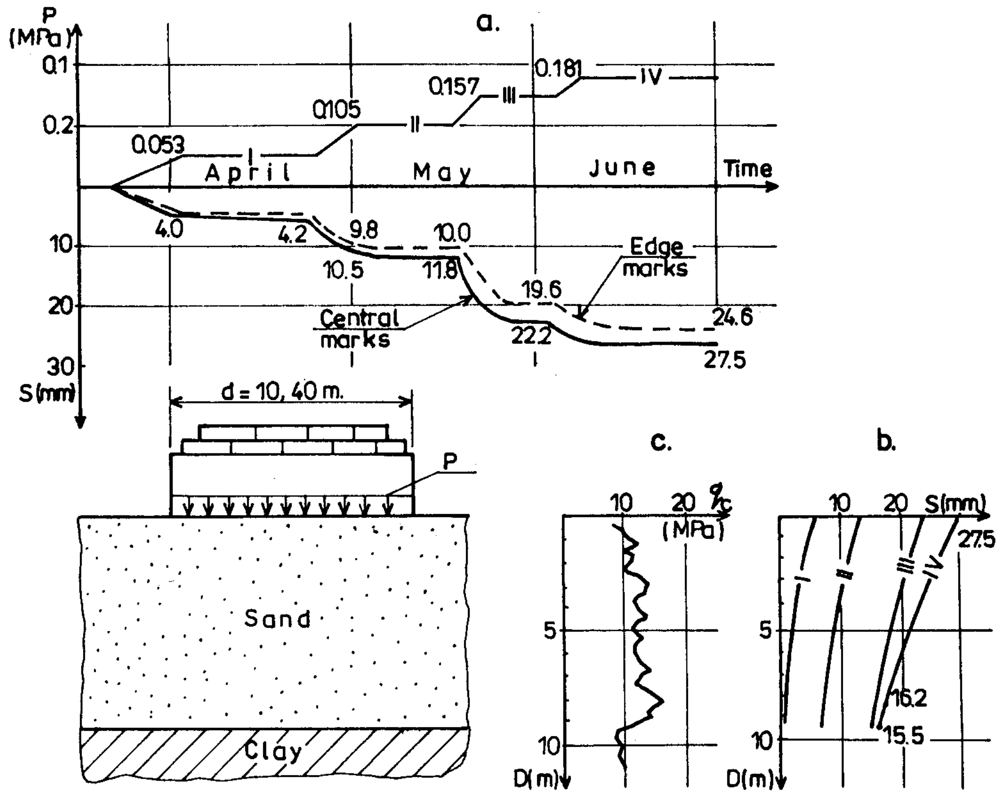


Figure 2. Measured settlements (a;b) and CPT results (c) on a large scale loading test in sand

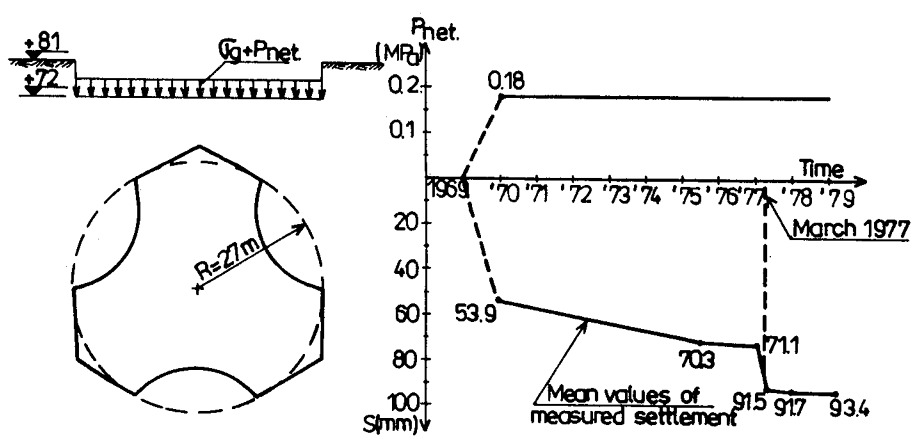


Figure 3. Settlement evolution of the raft foundation of the ***Intercontinental*** **Hotel in Bucharest**

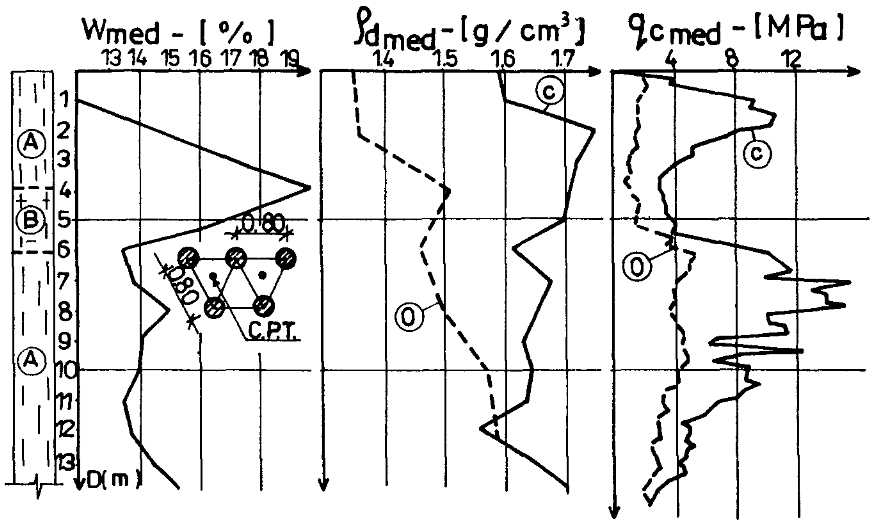


Figure 4. Compaction in the depth of the loessial layers by using stiffening columns Φ 300 mm. A - loessial silt; B - loessial clay; O - natural soil; C - compacted soil

by rapid plate tests and the q_c (Privighetoriță & al., 1995).

The linear regression:

$$E_u = 12.95q_c + 1.38 \quad (4)$$

can be used conservatively:

$$E_u \approx 11q_c \quad (4')$$

4.3. Tests in loessial collapsible soils

In natural conditions loess porosity is n = = 47...52% and water content w = 8...15%. The specific characteristic is its collapsivity, expressed by the sudden additional settlement induced by structural collapse when the soil is flooded until practical saturation. The supplementary strain for a given pressure p (usually p= = 300 kPa) may reach 10% or more (Botea & al., 1969).

In order to render possible shallow foundations upon saturated loess (because rising of underground water level and local water losses are unavoidable on building sites) the compaction procedures are utilised.

It was found out that the loess, whose porosity decreased by compaction below n = 40% (corresponding to the dry density ρ_d > 1.65 g/cm³) no longer exhibits supplementary strains

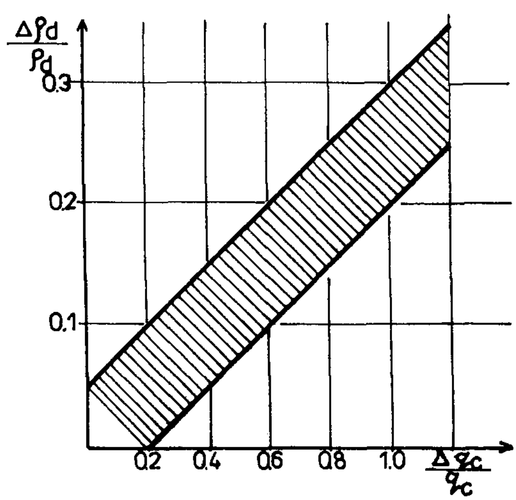


Figure 5. Variation range of the experimental correlations between the increase of dry density ρ_d and cone resistance q_c for loessial compacted layers

after humidification.

The control of the deep compaction of the compacted loess layers by using the ground stiffening columns can be accomplished by comparing the CPT diagrams obtained before and after compaction (Fig.4).

The test carried out on many sites where the loess compaction was achieved by 3...10 t hammers launched from 8...30 m heights, allowed the establishing of correlations between the increase Δq_c of the initial cone resistance q_c and the increase $\Delta\rho_d$ of the initial density ρ_d. In the case of loess compaction without humidification, the correlations obtained on different sites were situated in a relatively narrow variation range (Fig.5).

5. USE OF CPT FOR PILE CAPACITY CALCULATION

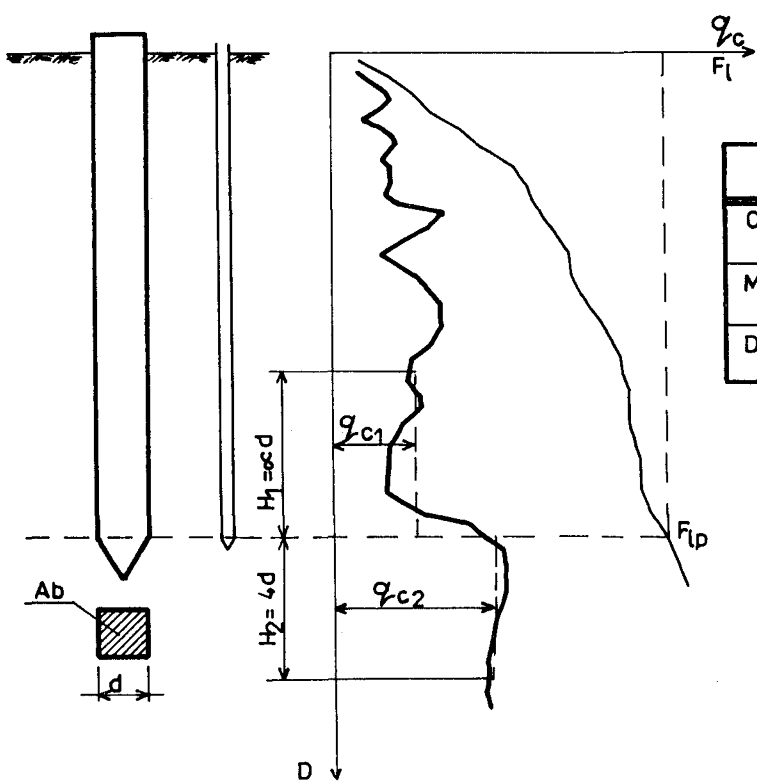
The best results in the determination of the bearing capacity of the driven piles were obtained by the direct use of the cone resistance values q_c and the total friction force on the sounding tubes F_ℓ (Boian & al., 1976). By comparing the CPT data with more than 100 loading tests on driven piles with cross sections ranging between 30x30 cm and 40x40 cm and lengths of 6...18 m, introduced into cohesive layers, sands and gravel, the following expression resulted for the determination of the ultimate pile capacity:

$$Q_u = \frac{q_c}{2} A_b + F_\ell \frac{U}{u} \qquad (5)$$

where:

A_b - the area of the pile base;
U - the perimeter of the pile cross sction;
u - the perimeter of the penetrometer, cross section.

The value F_ℓ is determined for the penetrometer thrusting to the level of the pile end. The value q_c is calculated by the relation:



SOIL	α
Clay loose sand	3
Medium dense sand	8
Dense sand	15

Figure 6. Determination of the design values q_c and F_ℓ for the evaluation of the bearing capacity for driven piles

$$q_c = \frac{q_{c1} + q_{c2}}{2} \qquad (5')$$

in which q_{c1} and q_{c2} respectively are the mean values of the cone resistance calculated over and under the level of the pile end, as shown in figure 6.

The relation (5) is recommended in Code C159 (1989) for the driven piles. For the bored piles, similar correlations were obtained and corrected with coefficients that depend on the boring and concreting technologies.

6. CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

Most of the comparisons were drawn between the values q_c and the number of blows necessary to penetrate 20 cm (N_{20}) recorded by the DPTH in sands.

The mean correlations established by one of the authors are represented in figure 7 and were

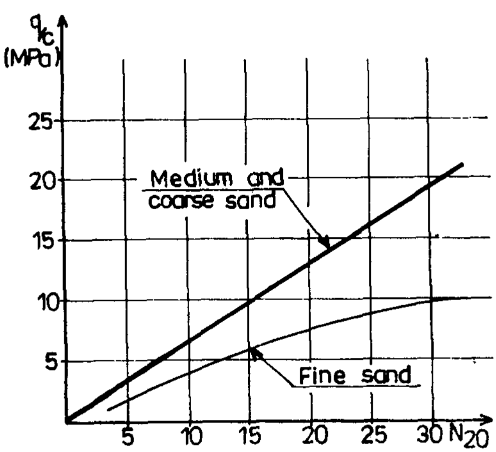


Figure 7. Correlations between CPT and DPTH results in sands

included in the Code C159 (1989).

The same code recommends, in case of using the DPTL in sands, the following relation:

$$q_c = 0.2\,N_{10} \qquad \text{(MPa)} \qquad (6)$$

In the case of cohesive soils, the correlations between the CPT and the DPT proved to be less stable.

7. CONCLUDING REMARKS

The CPT has proved its efficiency in solving a lot of the problems related the site investigation or to the foundation design.

However, due to the high costs, the spreading of the up-to-date penetrometers (which include pore-pressure measuring devices, inclinometers etc.) is limited nowadays in Romania.

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CONE PENETRATION TESTING IN RUSSIA

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Synopsis: The report consists of the following parts: a brief abstract of the surface geology in Russia, description of the equipment used, methods of calculating the soil characteristics and bearing capacity of piles, some trends and needs in future development of CPT.

1. Brief geological and geotechnical description of the region.

Construction in European part of Russia and in the South-East of the country is conducted mainly on continental deposits of Quarternary period. They are predominantly of glacial origin. In the North-East of the country we have permafrost grounds. Deposits of glacial origin are formed as the result of action of glacial sheet which crushed massif rocks, transported products of their erosion to a new territories and partly densified them. Deposits of glacial origin constitute the main part of different deposits.

Principal soils of glacial origin are the following: moraine clay and silty clay; fluvioglacial sands and gravels; lake-glacial deposits (varved clays, silty clays and silty sands) which are characterized by marked lamellar composition. These soils often have low bearing capacity.

Besides the soils of glacial origin we have on our territory different soils of erosion origin: residual soils having uneven compression; talus material, heterogeneous, often unstable; alluvial deposits which form big thickness of laminated clay and sand. From the construction point of view all Quarternary soils depending on composition, conditions of formation and others may have good as well as very low bearing capacity.

It were not soil conditions only what promoted the introduction of CPT in our country. In the 60-th the mass construction of apartement buildings from prefabricated reinforced concrete units began in our country. These buildings are very sensitive to uneven settlements, and they needed pile foundations in many cases. The wide use of the pile foundations required numerous investigations for the determining of pile sizes and their bearing capacity. The problem was solved to a great extend due to wide use of the static sounding apparatus which were devolopted at that time. In 1967 design of pile foundation on driven piles in accordance with CPT was included in our Building Code (SNiP II-B.5-67).

Data on the use of static sounding in the middle of the 70-th were given in the state-of-the-art report on ESOPT, Stockholm, Trofimenkov (1974).

2. Type of penetration testing and other investigation methods used in our country.

Field methods of soil test, which have a number of positive sides, are being widely used in our country. The principal advantages of the field methods are the following:

- possibility to receive data on characteristics of big massive of the ground, especially as compared with soil samples tested in a laboratory;
- in most cases they give a continuous geological profile;
- it is possible to test soils from which there is no hope to receive undisterbed samples;
- soil is tested in its natural condition;
- economy (except for plate load tests). At the same time on trials there rest some vaguenes: boundary conditions for stresess and deformations are unknown as well as conditions of drainage; the speed of deformations is considerably higher than in laboratory tests.

Because of these for reliable determination of soil characteristics both methods are as a rule used.

In our country the following methods of field test, all having State Standards, are used:

- static sounding with cone diameter 36 mm;
- dynamic sounding with direct driving of cone, diameter 74 mm, weight of hammer 30, 60 and 120 kg., height of fall accordingly 40, 80 and 100 cm;
- vane test with plate diameters 60, 80 and 100 mm;
- plate load tests in pits with plates of 5000 and 2500 cm^2 in area, and in boreholes with plate of 600 cm^2 in area;
- screw plates of 600 cm^2 in area;
- pressuremeter tests (diameters 76-127 mm);
- standard pile load tests (pile diameter 127 mm);

The equipment used and methods of tests are described in corresponding State Standards as well as in publications: Bondarik, Komarov, Ferronsky (1967); Trofimenkov, Vorobkov (1981); Mariupolsky (1989); Kolesnik, Ryzhkov (1976); Razorenov (1980); Rubinshtein, Kulachkin (1981), Dudler (1979).

3. Type of CPT-equipment used in our country.

There are two ways in the progress of the CPT-equipment in Russia: development in the whole (working out of sounding rigs) and development of new measuring devices and sensors. At present there are several sounding rigs in use, the main of them are S-979 and SP-59, developed by "Fundamentproject", S-832, developed by "BashNIIstroy", SPK and PSPK (underwater station for radioactive logging), developed by VSEGINGEO.

All of these rigs are hydraulically operated, have mobile bases and devices for measuring q_c and f_s. All the rigs, exept SPK and PSPK, correspond to the "Recommended European Standard on penetration testing" (1977).

The SPK and PSPK rigs differ from others known to us rigs in that there is possibility, besides traditional determination of q_c and f_s, to measure the natural gamma-background of soil, volumetric moisture content, and density of soils. It is possible because the probes are equipped with scintillation scanner of gam ma-rays, plutonium-beryllium source of fast neutrons, and source of gamma-rays. Addition of these detector and sources gives possibility to determine physical characteristics of soils (moisture content, density, porosity).

The rig SPK is mounted on 6-wheel drive truck. It has two probes 62 mm in diameter, the rig is completely automatized: the process of pushing in, pulling out and registering is fulfilled automatically.

On the acting rigs results of tests are presented, as a rule, in digital form, in physical units (resistance q_c, skin friction f_s, temperature t, pore pressure- if a sensor is present- u). On the present apparatus the information may be accumulated and stored in hard data medium on the truck with subsequent processing in personal computer.

There is also available the analog write operation for recording results of sounding on bend printer.

Appropiate attention is being given to calibration of apparatus. It is fulfilled either in laboratories or in the field. The main error of instruments does not exceed 5% over a practical range of measure of traditional parameters. Another important problem is the evaluation of errors in difficult geological and climatic conditions. Here constructional features are needed which protect the probe and instruments against extreme actions by use of electronic and materials of special design. Instruments developed by Institute NIIOSP have undergone tests in conditions of severe climate (Chita, Ulhan-Bator) and tropical weather (Salif, Yemen).

In "BashNIIStroy" from the end of the 60-th when working with apparatus S-832M a special procedure is sometimes used. This method called "sounding with stabilization" has the following distinctive features. At the predetermined depth of sounding the oil supply to a hydraulic jack is cut off, the motion of the probe falls and resistance of soil is balanced out. For increasing the duration of this process and decreasing the sensitiveness of hydrosystem to leakage of oil a special air reservoir is joined in parallel to the jack. The oil and air in this reservoir is compressed too. The pressure in the hydraulic jack and in the reservoir will always be equal, but owing to presence of air in the reservoir the damping capacity of hydrosystem markedly increased.

This system may be successfully used for testing of permafrost as well as other soils having visco-plastic characteristics, Manual (1977).

The field of application of all types of rigs is governed by their technical possibilities: pushing force, stability and strength of probes. In 1969, in order to widen the field of application of CPT, the new trend was started aiming to develop the apparatus suitable for use with different rigs, including foreign, and boring machines. This apparatus received the name PIKA (Field measuring set of apparatus). All in all it has been manufactured more than a thousand sets. They became the main part of all sounding rigs, Ilyichev, Kulachkin, Van den Berg et all (1986).

The greatest effect from the use of PIKA was received when these sets were used with boring machines UG B-50, PBU, URB and others. On this base B. I. Kulachkin developed the method of combined sounding. The essence of the method is in combining sounding and boring as a one process on the basis of boring. At the same time all the requirements of the Standard are fulfilled. All these enlarge the field of use of CPT and allow to receive results of sounding and boring simultaneously.

This method and its technical realization have some advantage in comparison with the rig Hyson-30Tf of the A. P. Van den Berg firm and the Huges (1989) version.

In further development PIKA will be instrumented with additional sensors, detectors and sources of fast neutrons and gamma-rays to measure pore pressure, moisture content and density of soils.

4. Interpretation of test results.

The determination of physical and mechanical properties of soils by means of static sounding is made according to acting. Building Code on site investigations (SNiP 1.02.07-87). For determining characteristic values of soils the following indices of CPT are used: unit cone resistance (q_c) and unit skin friction on the sleeve (f_s) according to State Standard-GOST 20069-81.

Given in norms data may be accepted directly for selection of the type of foundation, for design of soil base and foundations of buildings and structures of average responsibility (class III), as well as for computing the strength and deformation characteristics of soils for design deformations of soil base for structures of class II responsibility (according to norms structures of high responsibility have class I).

Data presented in norms relate to quartz and quartzo-felsphatic sandy soils of Quarternary period with small value of specific cohesion (less than 0.01MPa) and to Quarternary clays and silty clays with less than 10% of organic matter.

In norms there are the following data for determining different soil characteristics, given in abridged form.

Table 1. Density of sands.

Sands	q_c, MPa
Coarse and medium sands irrespective of water content	5-15
Fine, irrespective of water content	4-12
Silty of low water content	3-10
Silty, water saturated	2-7

Table 2. Angle of internal friction of sands versus q_c and h.

h m	q_c, MPa						
	1.5	3	5	8	12	18	26
2	28	30	32	34	36	38	40
5	26	32	30	32	34	36	38

Table 3. Modulus of deformation of sands versus q_c.

Sands	q_i, MPa				
	4	8	12	16	20
All kinds, except alluvial and fluvio-glacial	12	24	36	48	60
Alluvial and fluvio-glacial	20	30	40	50	60

Table 4. Modulus of deformation, cohesion and angle of internal friction of clays (exept moraine and lake-glacial) versus q_c.

q_c, MPa	E, MPa	C, MPa	φ, degrees
1	7	0.024	17
2	14	0.036	19
3	21	0.047	22
4	28	0.058	24
5	35	0.070	25
6	42	0.082	28

There are a lot of experimental data including that of Italian experiments in calibration chambers which show that given in our norms values of φ for sands are considerably understated. In the work of Trofimenkov (1988) an approximate relationship between φ and q_c was derived on the basis of the theory developed by Vesic (1970) and solution of Van Wieringen (1982) on the relationship between the cone resistance q_c and the limit pressure P_L of a relationship between φ and q_c was displayed in the graph.

The data of the graph show that at overburden of 5 m and q_c shown in Table 2 the values of φ are as follows:

Table 5. Liquidity index I_L of clays and silty clays versus unit side friction f_s.

q_c, MPa	f_s, Mpa					
	0.02	0.04	0.08	0.12	0.20	0.40
1	0.50	0.39	0.29	0.23	0.16	
2	0.37	0.27	0.16	0.10	0.02	
3	0.22	0.16	0.09	0.05	0.01	-0.06
5	0.09	0.04	0.04	-0.03	-0.07	-0.11
10		-0.05	-0.05	-0.10	-0.13	-0.16

q_c (MPa)	5	8	12	18
φ =	35	37	38.5	41

Angles of internal friction (φ) determined by the graph are about 5^0 larger than shown in Table 2. They differ from that given by Bellotti, Crippa et al (1985) by one-two degrees.

In the years past after publication of this norms extencive investigations were carried out in evaluation of mechanical characteristics of clayey soils by data of CPT. Generalization of these investigations was performed by Ignatova, Mariupolsky, Guster (1990).

In norms the mechanical characteristics of soils are given against q_c. In perfomed research the following data were collected: a) results of shear tests in laboratories in direct shear apparatus (consolidated test) and of modulus of deformation tests in the field by plates (5000 cm^2 in area) and screw piles (600 cm^2 in area); b) results of CPT test with measurement of q_c and f_s.

Silty clay and clay soils in five regions were investigated. These were soils of different genesis: alluvial, deluvial, lake-alluvial and others. Sheared in laboratories soils have degree of saturation S_r>0.8; values of modulus of deformation hold true for soils of different water content. In all, the number of test for φ and c amounted to more than 1500, for E-91 tests.

Generalized correlation- relation for φ and c of alluvial, deluvial, lake-alluvial silty clays and clays have the form.

$$\varphi=22.67 + 1.40q_c - 8.40f_s/q_c - 0.23I_p \quad (1)$$

$$c=-3.95 + 3.22q_c + 240f_s/q_c + 0.95I_p \quad (2)$$

In these formulae q_c and f_s are in MPa, I_p (plasticity index)-in %. We receive φ in degrees, c in kPa.

The following range of parameters was used in the derivation of formulae (1) and (2): q_c=1-5 Mpa, f_s/q_c=0.01-0.07, I_p=7-35.

Given relations allow to substantially accurately make the prognosis of φ and c as compared with SNiP 1,02.07-87 because of taking into account plasticity index I_p, as well as relation f_s/q_c for cohesion c.

Correlation ratio for these soils is about 0.7 for φ and 0.68 for c.

For determination of modulus of deformation the following relationship was received for alluvial and deluvial silty clays and days:

$$E=-3.4+6.4q_c+95f_s/q_c \quad (3)$$

Correlation ratio is about 0.82.

It should be noted that the values of E given by relation (3) are very close to taken in norms $E=7q_c$.

The conducted investigations will be used when SNiP 1.02.07-87 is being refined.

5. Use of CPT in geotechnical design.

The use of the static sounding for the determining of the bearing capacity of piles is one of the main ways of this method due to the direct analogy between penetrating of a sounding tube and a pile at the moment of failure, i.e. the sounding tube is taken as a model pile. In our country the determination of the bearing capacity of driven piles by CPT was first introduced in Building Code (SNiP II-B.5-67) in 1967. The ultimate resistance of a pile was determined by taking the unit resistance of the pile point to be equal to one half of the q_c, and the unit skin friction on the pile-to be equal to that on a sounding tube.

This method has undergone many changes during the ensuing years. At present we have the Building Code (SNiP 2.02.03-85) which allows the bearing capacity of a driven pile to be determined highly reliably.

In this norms provision is made for use of penetrometers with a friction sleeve and without it. The point resistance of a pile (R_s) is determined by the same way for both type of the penetrometers depending on cone resistance q_c, which is taken as a mean on the length 4d below and Id above a pile point. The unit skin friction on a pile (f) is taken differently for both types of penetrometers and for sandy and clayey soils.

According to norms the particular value of the ultimate resistance of a driven pile (F_u) is determined by the equation:

$$F_u = R_s A + fhU, \quad (4)$$

where in R_s is the ultimate resistance of the pile point, f-the mean value of the pile skin friction, h-the depth of pile driving, U-the pile perimeter:

$$R_s = \beta_1 q_c \quad (5)$$

where β_1 is the coefficient of transition from unit cone resistance q_c to unit pile point resistance taken according to Table 6.

The mean value of the pile skin friction is taken:

a) for penetrometers without a friction sleeve:

$$f = \beta_2 f_s \quad (6)$$

b) for penetrometers with a friction sleeve:

$$f = \frac{\Sigma \beta_i f_{si} h_i}{h} \quad (7)$$

In equations (6) and (7):

β_2, β_i are the coefficients taken by Table 6;

f_s- the mean varlue of friction on the sounding tube on the length of the pile;

f_{si}-the mean value of the sleeve friction in the layer h_i;

h_i-the thickness of a layer i.

When determining coefficients β_1, β_2 and β_i the ultimate bearing capacity of staticaly tested piles was taken at the settlement of 16 mm (0.2 of the ultimate allowable settlement of ordinary dwellings).

The allowable pile load is determined by the equation,

$$N = \frac{\bar{F}_u}{1.25\gamma_g} \quad (8)$$

where $\bar{F}_u$ is the mean ultimite resistance of piles in n sounding tests (no less than 6 tests), γ_g the safety ratio determined by statistical method at confidence level α=0.95.

Numerous concurrent tests in different Quaternary soils have shown good agreement in bearing capacity of piles determined by static loading and by sounding. On the results of 7S such tests it is indicated, Mariupol-

Table 6. Coefficients β_1, β_2 and β_i.

q_c kPa	β_1	f_s, f_{si}	β_2		β_i	
			sands	clayey soils	sands	clayey soils
<1000	0.90	<20	2.40	1.50	0.75	1.00
2500	0.80	40	1.65	1.00	0.60	0.75
5000	0.65	60	1.20	0.75	0.55	0.60
7500	0.55	80	1.00	0.60	0.50	0.45
10,000	0.45	100	0.85	0.50	0.45	0.40
15,000	0.35	>120	0.75	0.40	0.40	0.30
20,000	0.30	-	-	-	-	-
>30,000	0.20	-	-	-	-	-

sky, Seskov, Rodkevich (1986), that the mean relative error in the ultimite resistance of piles ranged between +25% and -21%.

In regard to the use of CPT for the determining the bearing capacity of cast-in-place piles there is no norms in force. Accumulation of experimental data is being carried out.

Of interest is the methodology for determining the bearing capacity of piles when at the field site there are simultaneously results of cone penetration testing and a few pile load tests. This methodology was developed by Ryzhkov (1988).

In this method results of every sounding is converted into the pile bearing capacity at the points of sounding. The developed methodology is based on the use of Bayes' formula which allows to revalue the probability of different hyprothesis after some new event has taken place. In this solution the following data are used: the past experience on the accuracy of pile resistance determination by method of CPT and results of pile load tests of one or more piles at the points of sounding. This method gives possibility to calculate the bearing capacity of piles more reliably.

In Russia a big construction programme is being developed on the permafrost grounds which occupy more than 50% of the country territory. This is made urgent to investigate the possibility to use CPT for determining the characteristics of frozen soils and designing of pile foundations in this region.

The first investigations which were made have shown prospect in the use of this method. For this goal an extensive programme of investigation has been fulfilled in the last 10-15 years. These investigations were conducted mainly in regions with plastic frozen soils. Results of some investigations are given by Isaev, Shvarev, Tikhomirov et al (1991).

These investigations were carried out with the use of the S-832M. The mean depth of sounding was 10 to 12 m, maximal- 20 m. For the research the special temperature probe was developed which allowed to measure the soil temperature at the probe point and along its length. Sounding was made at more than 100 points at 10 experimantal building sites consisted of plastic frozen clayey soils of glacial origin at temperature of 0 to -2^0C.

In these investigations so-called "with stabilization" method of sounding was used. In this method, as it said earlier when the cone is at a prescribed depth the oil discharge by jack pumpe is stopped, the speed of the cone movement is quickly decreasing and soil pressure on the cone comes into equilibrium. It was taken that this pressure of soil at equilibrium is close to the long-term strength limit of the soil.

In the process of experiments three different resistances of cone penetration q and skin friction f were measured: "rapid" $q_{\dot{c}}$ and $f_{\dot{s}}$ (under uniform pushing with the speed of 0.5m/min), "stabilized" q_c and f_s (under the balanced out state of the system cone-soil) and "peaks" q_c' and f_s' (at the first moment of the cone movement after freezing).

In comparative analysis of sounding data with the physical-mechanical properties and the long-term strength limit of soils there were used: more than 200 bearing ball tests, more than 80 consolidation tests, 34 pile load tests.

On the basis of the correlation-regression analysis the following formulae were received.

1. Long-term equivalent cohesion C_{eq} (MPa).

$$C_{eq}=B_c(q_{\dot{c}})^{n_c}, \qquad (9)$$

where B_c; n_c=empirical coefficients determined in concurrent CPT and bearing ball tests.

For investigated clayey soils of Vorkuta of glacial origin B_c=0.0031; n_c=1.49. Correlation ratio is r=0.87.

2. Oedometric modulus E (MPa).

$$E=B_e(q_c)^{n_e}, \qquad (10)$$

where B_e; n_e= empirical coefficients determined in concurrent CPT and oedometer tests. From these tests B_e=7.39; n_e=0.49; r=0.75.

3. Long-term limit strength of frozen soil under pile base R(MPa).

$$R=B_r(q_c)^{n_r}, \qquad (11)$$

where B_r; n_r=empirical coefficients determined in concurrent CPT and pile load tests. From these tests B_r=0.87; n_r=0.54; r=0.93.

4. Long-term shear strength of frozen soil on pile shaft f_p (MPa).

$$f_p=B_0(q_c)^{n_0}, \qquad (12)$$

where B_0, n_0=empirical coefficients determined in concurrent CPT and pile load tests. From these tests B_0=0.95; n_0=1.56; r=0.94.

Comparison of long-term limit resistances of piles determined by 23 load tests with those determined by CPT results and by Tables of Building Code 2.02.04-88 has shown that corralation ratio for the former was r=0.94, and for the latter r=0.64

6. Comparison and correlation of CPT with other investigation methods.

Comparison of the modulus of deformation determined by the results of cone penetration tests and standard plate load tests are given below. The plate was of 5000cm², the depth of testing-from 2 to 15 m.

On the bases of 50 tests in alluvial sands the following formula was received, Trofimenkov, Vorobkov (1981):

$$E=3.4q_c+130; \qquad (13)$$
$$E \text{ and } q_c \text{ in kg/cm}^2$$

Correlation ratio was 0.8.

For clayey soils results of 97 tests in Quaternary soils are given in the same paper. The following formula was received:

$$E=7.8q_c+20 \quad (14)$$

Correlation ratio was 0.88.

Data of that experiments show that the values of the modulus of deformation in our norms are taken with the proper safety-both, for sands ($E=3q_c$) and for clayey-soils ($E=7q_c$).

7. Major areas for research activities.

The technical progress in cone penetration testing is presumed to depend mainly on design of the cone, as all other components of rigs are used from industry ready made. This is expected to be the main direction of our activity.

It is well known that there is no close relation between one of the sounding parameters (q_c and f_s) and different soil characteristics. Nevertheless it was shown, Rubinstain, Kulachkin (1981), that these parameters contain information on strength and deformability of soils. The problem is how to extract these strength and deformability data from integral characteristics q_c and f_s. The solving of this problem may be reached by the complex consideration of all interacting factors of sounding process: soil-cone interactions, correlation between inertia of instruments and processes in soil during cone penetration, characteristics of driving devices, and so on. We are working on this problem.

8. Future trends and new development.

At the present the Union Rosstroyiziskaniya has created the experimental prototype of a new sounding-boring complex on 6-wheel drive truck KamAZ. The equipment is oriented on the use of PIKA, but a version is also considered of creating the probe with elements for recording and storage of information. On the vehicle the data processing system is situated.

The main progress lately has been in installing different additional sensors in cones. Cones were created (modification of PIKA) with regulated discharge of water at the bottom of a borehole, with the piezometer sensor which allows the measurement of pore pressure, with temperature sensors, detectors and sources of gamma and neutrons rays.

In the last years a new direction receives development in geotechnics and agronomy-ecology. In connection with this, new apparatus have appeared on the base of CPT, for example, "Radon", Kulachkin, Ilychev, Van den Berg (1994). The use of ionic selective detectors allows to measure pH, Redax potential, the presence of heavy metals. These works are being fulfilled practically in parallel with A.P. Van den Berg firm.

The immediate practical task is the accumulation of experimental data for developing reliable method of the bearing capacity determination of cast-in-place piles on the basis of cone penetration testing.

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Cone Penetration Testing in Singapore and Malaysia

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SYNPOSIS: The use of cone penetration testing in site investigation has gained popularity only in the last five to ten year in Singapore and Malaysia. The test is primarily used for sounding purposes to detect the subsoil stratigraphy and to evaluate the depth variation of soil properties such as shear strength and overconsolidation ratio and to provide a basis for field control of sandfill. The test is also used for the assessment of the coefficient of consolidation in projects involving reclamation and/or ground modification. A research emphasis has been placed on the application of cone penetration test to the characterization of recently reclaimed land in Singapore.

1. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION

Singapore and Malaysia are located in Southeast Asia. Geographically, Malaysia (specifically West Malaysia) covers the entire Peninsular Malaysia. Singapore, separated from the West Malaysia by a narrow strait, is located right at the southern tip of the peninsular.

The geology of Singapore and that of Malaysia are similar. There exists a spectrum of geological formations from the north to the south of Peninsular Malaysia. The dominant geological formations are the granite formation in western-central Peninsular Malaysia and central Singapore Island, the sedimentary rock formation in central Malaysia and western Singapore, the calcareous formation in north and southeast Malaysia, the old alluvium in eastern Singapore, and the Recent (Quaternary) alluvial deposits, dominated by soft to medium stiff marine clays and occasionally peaty clays, which cover the coastal plains. In addition, recently reclaimed land forms a significant part of the total landspace in Singapore. In certain areas of Peninsular Malaysia, tailing ponds from tin mining contain special slime deposits.

Significant fraction of construction activities and geotechnical works in Singapore and Malaysia are centred around the coastlines where Recent alluvial deposits consisting predominantly of marine clays prevail. The main use of cone penetration test in this region is for the characterization of Recent deposits and sandfill at reclaimed sites and in ground modification projects.

2. PENETRATION TESTING AND OTHER INVESTIGATION METHODS

Site investigations in Singapore and Malaysia are traditionally based on direct boring and sampling followed by laboratory testing. Shelby tubes are often used for undisturbed sampling in soft to medium stiff clays. In the case of investigation in stiff to hard residual soils, a Mazier triple tube core barrel is sometimes used to obtain high quality undisturbed samples. Field vane tests are commonly incorporated in direct boring in soft to medium stiff clay deposits. Standard penetration tests are used instead when sands and stiff to hard clays or highly weathered rocks are encountered. The cone penetration test (CPT) is presently the most commonly used field investigation method other than direct boring. However, the use of CPT is restricted mainly to Recent deposits and reclaimed land. Other in-situ tests used in Singapore and Malaysia include the pre-boring pressuremeter test and the dilatometer test. Occasionally, the plate load test, the self-boring pressuremeter test, the Swedish ram sounding test, the Swedish weight sounding test, and the Mackintosh probe are used.

3. CPT EQUIPMENT, TEST PROCEDURE AND RESULT PRESENTATION

CPT is now widely used in many parts of the world. CPT practice in Singapore and Malaysia, in term of equipment, test procedure, and result presentation, follows closely to that adopted elsewhere.

3.1 Equipment

A CPT system generally comprises a jacking rig, a data acquisition unit, and the cone itself. Several types of jacking rig are used in Singapore and Malaysia. The hydraulic ram is either a single cylinder of 10 ton capacity or a twin cylinder of 20 ton capacity. The hydraulic jacking unit is either trailer mounted, crawler mounted, or truck mounted. The truck-mounted rig is not that popular due to the hot and humid climatic condition, and the frequently found soft compressible water-logged terrain in coastal plains. For near-shore investigation, hydraulic rigs erected directly on jack-up or piled platform, have also been employed. Fugro Singapore has also used a hydraulic penetration machine mounted on a seabed frame, a wheel drive Seacalf, in recent off-shore cone penetration testing.

A few types of cones are used in Singapore and Malaysia. Traditionally, mechanical cones were used, but they had been replaced with electric cones and subsequently with piezocones of different makes in the past few years. Electrical cone and piezcone systems used in this region include the GMF Gouda cone, the Fugro cone, the Hogentogler (USA) cone, the AB van der Berg cone, the Kiso-Jiban (Japanese) cone and the Swedish Envi Menocone. The design of these cones are similar. Most cones are subtraction type compression cones. A tension type electric cone has also been used occasionally when there is a need for accurate determination of unit sleeve friction in soft soils. The standard version of these cones is generally 10 cm^2 in cross-sectional area, 150 cm^2 in sleeve area, and having an apex angle of 60°. The filter is usually located just behind the conical tip for most piezocones. These cones generally have friction sleeves with equal end area to avoid necessary correction for measured friction values. The net area bearing ratio of these cones, however, varies from 0.62 to typically 0.8. Piezocones with filter located on the cone face or with dual filters at the base and on the face have also been used occasionally.

High precision pressure gauges are used for cone resistance and friction measurements in a mechanical cone test. For electric cone and piezocone tests, the data acquisition system used in the region usually consists of a computer which displays key outputs from the cone as the test is in progress and at the same time stores the output data in hard disks. The data are generally recorded at every 50mm penetration during sounding, although occasionally the recording depth interval is reduced to 25mm for some special needs. A printer is often connected to the computer for obtaining on-line printouts of key results. A floppy disk drive can also be attached to down load CPT data at the site or in the laboratory.

3.2 Test procedure

The common test procedure follows either British Standard Codes of Practice B.S. 5930:1981 or ASTM D3441-79. Dutch Standard NEN3680 : Soil Investigation - Static Cone Penetration Tests has also been adopted.

For most projects, the cone penetration test is carried out for sounding purposes. The penetrometer is pushed down into the soil at a constant penetration rate of 2 cm/s. The parameters measured by a piezocone often include cone resistance (q_c), unit sleeve friction (f_s), porewater pressure (u), inclination, and temperature. Signals from the penetrometers are generally transmitted by a special electric cable through the sounding rods to the surface data acquisition system. In the case of the cordless Menocone, however, the signals are stored in a memory and processor unit located immediately behind the cone sleeve during the test.

For piezocone tests, a careful preparation of the cone prior to a test is crucial. De-aired silicon oil or glycerin is often used for saturation of the filter element (s), although a filter element saturated with water and protected with a rubber tube section has also been used with success. A waiting period of between 20 and 30 minutes, with the saturated cone submerged in water, is usually practised to facilitate the stabilization of the measuring instruments.

Due to the extensive presence of soft clay in the region, the accuracy of the measuring parameters is critical in a cone penetration test. A specification which calls for the following resolution of the measuring system for piezocone tests in soft soils has been drawn up by the practising engineers in Malaysia: cone resistance (q_c) - 0.01Mpa, unit friction (f_s) - 0.1 kPa, and pore pressure (u) - 1 kPa.

The test is normally terminated when the force of cone reaches its safe capacity of 5 tons or 10 tons or when the inclination of the cone exceeds 15^o from the vertical.

In some special projects, pore pressure dissipation tests are also incorporated in the regular sounding for the estimation of coefficient of consolidation. Occasionally, long duration dissipation tests are used for the establishment of the in-situ pore pressure in a consolidating clay such as that presents below the sandfill at recently reclaimed sites.

Calibration of cones is usually done by the cone manufacturers at their technical support centres as the calibration constants are often built into the system. Some frequent cone users return their cones to the manufacturers for checking and calibration once every six months or in the event that something is suspected with the cone. Local calibration is only carried out for the cone resistance measurement by means of a hydraulic jack and a load cell on a somewhat random basis. Both the friction measurement and the net area bearing ratio, which depends on the detail of cone design, are normally not calibrated locally.

3.3 Result presentation

The test results are usually presented in profiles of corrected cone resistance (q_t), sleeve friction (f_s) or friction ratio (f_s/q_c or f_s/q_t), and penetration porewater pressure (u) versus depth. Corrections of results are usually made for the unequal end area effect for the cone resistance so that corrected cone resistance (q_t) instead of the direct-measured cone resistance (q_c) is presented. In Malaysia, additional profiles such as ($q_t - \sigma_v$) and ($u - u_o$) versus depth, where σ_v is the vertical total stress and u_o is the in-situ porewater pressure, are sometimes presented.

For piezocones with the filter located at the cone base, the correction is usually based on $q_t = q_c + (1-\alpha)\, u_{bt}$, where u_{bt} is the pore pressure measured at the cone base and α is the net area bearing ratio. The value of α, supplied by the cone manufacturers, ranges from 0.62 to 0.8 in general, although one manufacturer claimed that $\alpha = 1$ could be used for his cones.

For piezocones with the filter located on the cone face, Fugro Singapore uses the following expression: $q_t = q_c + (1-\alpha)\,[\,B\,(u_t - u_{bt}) + u_o\,]$, where u_t is the pore pressure measured on the cone face, u_o is the in-situ pore pressure, and B is the ratio of u_{bt}/u_t. Both inclination and temperature effects on the cone resistance are seldom accounted for. No correction is generally made for sleeve friction for most cones.

4. INTERPRETATION OF TEST RESULTS

The interpretation of CPT results in Singapore and Malaysia is based on correlations in the literature that are commonly referred. Most of these correlations are empirical and semi-empirical in nature. Efforts are therefore often made whenever possible to verify the validity before the application of these correlations to local soils in the region.

4.1 Soil classification and stratigraphy

In a CPT sounding, not only that the data is continuous with depth but also that more than one parameter can be measured in a single pushing. The inter-relation between these parameters is significantly influenced by the soil type and the associated soil properties. As such, one main objective of CPT sounding has been to provide additional data for the verification of soil stratification.

Traditionally, with the use of a mechanical cone for which two parameters, the cone resistance (q_c) and the sleeve friction (f_s) are measured, soil classification, such as the classification chart proposed by Schmertmann (1978), has been primarily based on the friction ratio (f_s/q_c). There has been not much improvement of the accuracy of classification with the introduction of electric cones. With the birth of the piezocone for which an additional important measurement, the penetration pore pressure (u) is added, a new dimension has been created in the use of CPT results for soil classification and determination of soil stratigrapy. This is attributed to the fact that the penetration pore pressure is extremely sensitive to the drainage and the shear characteristics, as well as the compressibility, of the soil. Significant improvements have been made in soil classification with the incorporation of normalized parameters derived from the three basic measurements in a piezcone sounding (Robertson, 1990). Nevertheless, the classification chart has also become more complicated as a result.

In Singapore and Malaysia, where the current site investigation practice is still inclined toward direct boring methods, piezocone soundings are very often introduced as a supplement to boreholes. A direct reliance of CPT on soil classification is normally not required. Various soil classification charts available are used for the estimation of soil types for comparison with boring results. The use of CPT results in this aspect is therefore primarily to help on the verification of the detailed soil stratigraphy, such as the existence of sand seams in the marine clay, which can be a crucial factor in a soil improvement project involving preloading.

Due to the difficulty of using a complicated soil classification chart in practice, some simple rules have been adopted on the basis of observation of the penetration pore pressure to utilize the CPT results for the verification of certain soil types in Malaysia. Examples of these rules include (a) the excess pore pressure generally cannot dissipate quickly for penetration in normally to lightly overconsolidated clays, (b) the penetration pore pressure can be negative in heavily overconsolidated clays, as well as in dense fine sands, silty sands, and silts, due to the tendency for these materials to dilate in shear during a cone penetration, and (c) penetration in a mixed soil often results in rapid varying plots or spikes in the pore pressure profile.

4.2 Soil parameters

Although CPT sounding basically involves a series of large strain shear tests, the penetration resistance (q_t) and the sleeve friction (f_s) are affected not only by the shear strength of the soil alone, but also by many other soil parameters such as the overconsolidation ratio and the modulus or compressibility of the soil. The pore pressure response in a piezocone test is also affected by these factors and others related to the drainage characteristic of the soil. The rate of dissipation of excess pore pressure in a piezocone dissipation test is direct reflection of the consolidation characteristics of the soil. The CPT results have therefore been used very extensively, in addition to soil classification, for the evaluation of engineering soil parameters. For investigation in predominantly clayey Recent alluvial deposits in Singapore and Malaysia, the main parameters that are evaluated from CPTs include the undrained shear strength (s_u), the overconsolidation ratio (OCR), and the coefficient of consolidation due to horizontal drainage (c_h).

4.2.1 Undrained shear strength

The undrained shear strength (s_u) is usually evaluated from CPT using the cone resistance (q_t) and a cone factor N_{kt} as follows:

$$s_u = (q_t - \sigma_{vo}) / N_{kt} \qquad (1)$$

A comparison of the undrained shear strength from field vane tests with parallel CPT tests from 12 recent project sites in Malaysia involving the Malaysian marine clay by Wong (1995) showed that the N_{kt} ranged from 5 to 13 and averages 10. A cone factor of 10 has been recommended for the Malaysian marine clay. Studies carried out by Dobie (1988) in Singapore showed that the N_{kt} ranged from 9 to 12 and averaged 10.5 when the uncorrected vane strength was used for the Singapore marine clay, which is geologically similarly to the Malaysian marine clay. The corresponding N_{kt} was between 15 and 20 and averaged 17.5 when Bjerrum's correction factor was applied to the vane strength based on the plasticity index. A recent study by Orihara et al. (1993) showed that the N_{kt} ranged from 12 to 14 and averaged 13 for a slightly overconsolidated seabed marine clay near the Singapore Island. Interestingly, the corresponding N_{kt} was between 5 and 12 and averaged 8 for the same marine clay that was still undergoing consolidation under the influence of a newly reclaimed sandfill.

Recommendations have also been made to use the excess penetration pore pressure (Δu) in the evaluation of undrained shear strength (e.g. Robertson et al., 1986) as follows:

$$s_u = \Delta u / N_{\Delta u} \qquad (2)$$

Orihara et al. (1993) showed that $N_{\Delta u}$ was between 7 and 8 for a slightly overconsolidated seabed marine clay near the Singapore Island. Lunne et al. (1985) pointed that $N_{\Delta u}$ could be normalized with respect to B_q and the ratio would become N_{kt}, where $B_q = \Delta u /(q_t - \sigma_{vo})$ is the penetration pore pressure ratio. From $N_{\Delta u}$ versus B_q plots for 12 project sites involving the Malaysia marine clay, Wong (1995) found that the N_{kt} based on the penetration pore pressure averaged 10, remarkably similar to that based on the cone resistance.

4.2.2 Overconsolidation ratio

Overconsolidation ratio (OCR) affects all the three measurements in a CPT. Estimation of OCR profiles from the CPT results is therefore possible. According to Wroth (1984), B_q which resembles the pore pressure parameter A in a triaxial test should decrease as OCR increases. Both Chang (1988) and Dobie (1988) have verified this trend for the Singapore marine clay. A correlation has been proposed by Chang (1991) for the estimation of OCR from B_q for clays with both OCR and sensitivity less than 8. Chang's correlation, which is believed to provide a slightly conservative estimate of OCR for the Singapore and the Malaysia marine clays, is as follows:

$$OCR = 2.3\ B_q / (\ 3.7 B_q - 1) \qquad (3)$$

A verification of Eq. (3) by Orihara et al. (1993) reflected that Chang's correlation was suitable for practical use within a confidence limit of 25 to 35% for the Singapore marine clay with the OCR ranging from 0.6 to 1.6. It is interesting that the correlation appears applicable also to "under-consolidated" clays which are commonly found at reclaimed sites in Singapore.

A more recent approach toward estimation of OCR from CPT is to employ the cavity expansion theory and the critical state concept. Equations relating OCR to CPT measurements q_t and Δu and critical state parameter M (Mayne, 1991; Chang, 1992; Chen, 1994). These equations are occasionally used for the evaluation of OCR profiles in Singapore and Malaysia. Both Chang's (1992) and Chen's (1994) correlations have been found to produce OCR-profiles which compared well with odeometer results based on CPTs at three project sites involving the Malaysian marine clay (Wong, 1995).

4.2.3 Coefficient of consolidation

CPT dissipation tests have only recently been used in Singapore and Malaysia for the evaluation of the coefficient of consolidation c_h mainly in conjunction with ground improvement and reclamation projects. The methods proposed by Torstensson (1977) and Baligh and Levadoux (1986) are often used for the interpretation of c_h from a pore pressure dissipation curve. Occasionally, Teh and Houlsby's (1991) method is used. A recent study reported by Wong (1995) showed that both Torstensson's and Teh and Houlsby's methods provided reasonable estimates of c_h when compared with laboratory values at three Malaysian test sites. A series of CPTs were conducted recently at four well-apart locations in a near-shore seabed marine clay in Changi East area in Singapore. Results of these tests showed that the c_h values interpreted using the method proposed by Baligh and Levadoux (1986) agreed well with laboratory data obtained from hydraulic consolidometer tests. The CPT deduced c_h values also compared well with those estimated from parallel dilatometer dissipation tests, but were consistently lower than those interpreted from self-boring pressuremeter holding tests.

5. USE OF CPT IN GEOTECHNICAL DESIGN

CPT has become widely used in Singapore and Malaysia only in the past 5 to 10 years. In Singapore, the tests are currently used primarily for projects involving massive earth works that cover large areas such as land reclamation and ground improvement works. The test has also used basically as a supplementary investigation method along routes of proposed major highways and a Mass Rapid Transit line. In Malaysia, CPTs have been used extensively in site investigation along the newly constructed North-South Expressway, particularly in areas involving trial embankments. The test has also been extensively used in an airport development project.

Basically, the two main current uses of CPT in Singapore and Malaysia are for site characterization of predominantly clay deposits in projects involving extensive earthwork and for field control of sandfill at recently reclaimed or ground improvement sites. CPT results have not yet been used directly in the design of piles or other geotechnical works in this region.

For CPTs in clays, results have been used primarily for the evaluation of the undrained shear strength, the overconsolidation ratio and the coefficient of consolidation, which are critical for the design of earthworks and ground improvement works. For CPTs in sandfill, the cone penetration resistance (q_t) is used as an index for gauging the suitability of the sandfill for support of designed surface loads and for judging the extent of improvement from various ground modification schemes. In a ground improvement project at a reclaimed site in Singapore, for example, a q_t of no less than 12 MPa or 15 MPa has been specified for diffent areas at the site as the acceptance criterion for the reclaimed sandfill in an airfield development.

6. MAJOR AREAS OF RESEARCH ACTIVITIES

The major areas of research on CPT in Singapore and Malaysia have been centred around the interpretation of results of piezocone tests in marine clays. A particular emphasis is placed on the assessment of preconsolidation pressure profiles for clay layers underlying recently reclaimed sandfill through the estimation of in-situ vertical yield stress or in-situ porewater pressure.

A study involving review of methods of evaluating OCRs or preconsolidation pressures of clay layers from normal piezocone sounding tests has been undertaken at the Nanyang Technological University (NTU) in Singapore. Validity of these methods are verified with data collected from project sites all over Malaysia and Singapore. Methods developed based on the cavity expansion theory and the critical state concept, including those of Konard (1989), Mayne (1991), Chang (1992), and Chen (1994), have been selected for the study. Comparisons of the predicted OCRs or preconsolidation pressures with laboratory oedometer data over three sites in Malaysia (Wong, 1995) showed that Chang's and Chen's correlations produced results that are the closest to the oedometer data.

A study is also currently being conducted to measure the in-situ porewater pressure using long-duration piezocone dissipation tests at recently reclaimed sites. A special piezocone with two filters, one located at the base and the other located on the cone face, is employed in some of these tests. Figures 1 and 2, respectively, shows the CPT profiles and the pore pressure dissipation curves obtained at 20 m depth at a reclaimed site in Singapore. The dissipation curves for the excess penetration pore pressure, which was obtained by subtracting the hydrostatic pore pressure (u_s) from the measured penetration pore pressure (u) are also shown. It is interesting that the two pore pressure readings (u_t and u_{bt}), which differed only by 12 kPa initially, merged as the tail of the s-shape dissipation curve became established. This merger of two pore pressure readings appears to have provided a clear basis of justification for the determination of point of full dissipation of the penetration pore pressure, which is crucial and yet difficult from the dissipation curve. With the dual pore pressure readings, an extrapolation of full dissipation appears possible if the dissipation test is stopped after around 80% dissipation. The use of a dual-filter cone should prove worthwhile in this particular area of application.

A theoretical study which aims at the development of interpretation methods for CPTs in saturated clays, particularly at reclaimed sites, are also being undertaken at NTU. A theoretical model is being developed primarily for the evaluation of OCRs of clays on the basis of the cavity expansion theory and the Cam Clay model. A comparison of results from a preliminary version of this model with data from several published case studies indicated that the model which incorporated the strain

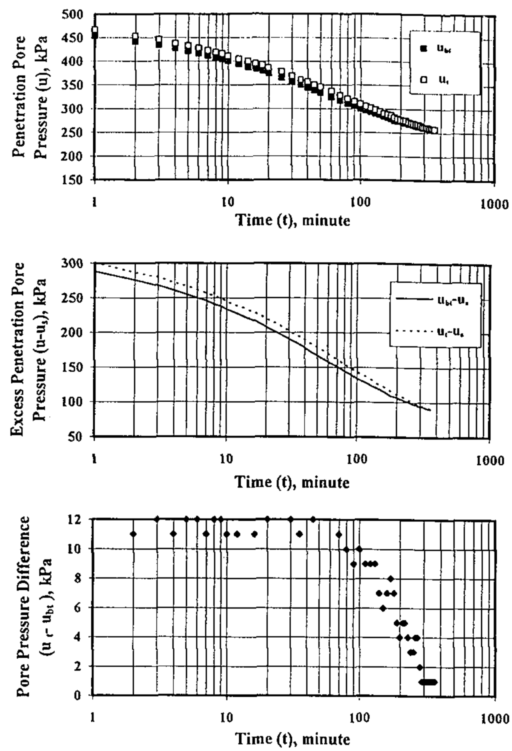

Fig. 2 Pore pressure dissipation curves from a dissipation test

rate effect on undrained shear strength was promising for interpretation of piezocone data. A model for the direct assessment of in-situ vertical yield stresses from measurements made in normal piezocone penetration tests for reclaimed sites is also currently being developed.

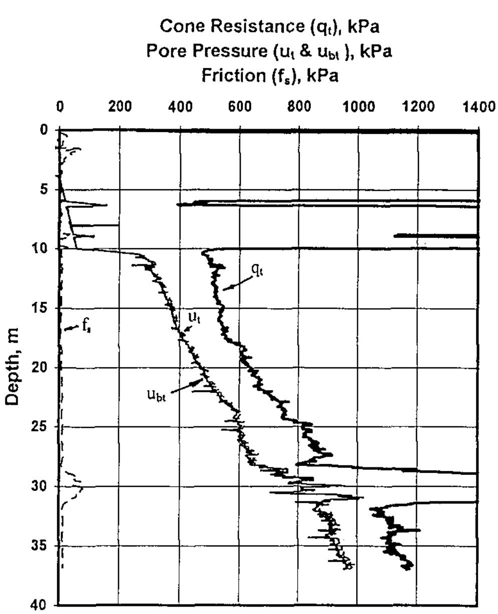

Fig. 1 CPT profiles at a reclaimed site in Singapore

7. FUTURE TREND AND NEW DEVELOPMENT

The use of CPT in construction projects in Singapore and Malaysia is on the increase. The demand is high especially for projects involving massive earthworks mainly for the characterization of both the sandfill and the underlying clayey foundation soils. There is a trend toward the use of CPT as a standard field control test for compacted sandfill. Direct use of CPT in foundation designs is a possible development yet to be seen.

8. ACKNOWLEDGMENTS

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National Report on Cone Penetration Testing in Slovenia

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SYNOPSIS: Within the Republic of Slovenia, geotechnical conditions are, in some places, relatively unfavourable due to various geological and morphological reasons. Many depressions, river valleys and the hinterland of the Adriatic coast are covered by different deposits, which are heterogeneous and, at many locations highly compressible and of low bearing capacity. In these areas CPT has proved to be a very useful tool for determining classification and soil properties such as strength, deformability and permeability. Over the last decade CPT has been used for the design of both shallow and deep foundations, e.g. for buildings, road structures and embankments. CPT equipment development is followed, and efforts are made to obtain as much information as possible regarding the determination of correlation between CPT results and geotechnical parameters.

1. BRIEF GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF THE REGION

The Republic of Slovenia which has two million inhabitants and a surface area of 20.926 km^2, is situated in Central Europe. Its geographical position in relation to neighboring states is presented in Fig. 1.

Geotectonically, Slovenia belongs to the Southern Alps, Eastern Alps, Dinarides and the Pannonian basin. Due to the very complex geology of the region, geotechnical conditions are often very demanding.

The oldest formations (the metamorphic complex of the Eastern Alps) belong to the Lower Paleozoic. On the other hand, soft soils, where CPT is mostly used, have been deposited since the Pleistocene. These soils are mostly normally consolidated, fine grained, of low to high plasticity, and of moderate sensitivity. In some marshy districts peat and organic clays can be found. The most important areas of soft subsoils, where geotechnical investigation by CPT is commonly used, are shown in Figure 1. The most significant and vast areas are the Ljubljana Marshlands and the hinterland of the Adriatic coast.

The tectonic depression of the Ljubljana Marshlands is filled with Pleistocene fluvial and marsh sediments, and with Holocene lake deposits. The total thickness of these sediments is up to 150 m, whereas the thickness of the very soft upper layers is up to 20 m.

The coastal region of Slovenia, as well as the north-western part of Croatian Istria, is covered by very soft marine and fluvial clayey deposits, with intercalations of cohesionless layers. Deposits of this type are particularly thick in the lower parts of the Mirna and Dragonja valleys.

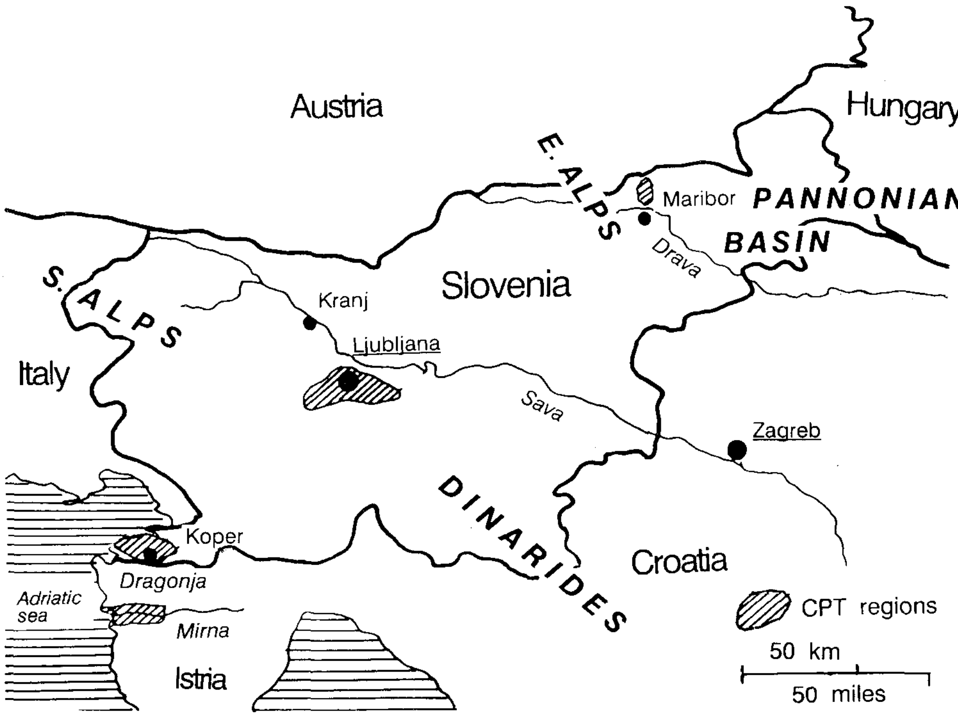


Fig. 1. Geographic units in Slovenia, and the main regions of CPT investigations

2. TYPE OF PENETRATION TESTING AND OTHER INVESTIGATION METHODS USED IN SLOVENIA

In Slovenia, the Cone Penetration Test /CPT/, the Standard Penetration Test /SPT/ and Dynamic Probing /DP/ are used. Among other investigation methods, test boring with sampling and laboratory investigation of the samples, various geophysical investigations, the vane test and pressiometer measurements in rock are used.

3. TYPE OF CPT EQUIPMENT USED IN SLOVENIA

3.1 Equipment

In Slovenia there are two static cone penetrometer assemblies. The older assembly was constructed in 1984. The mechanical-hydraulic part of the equipment, with a maximum thrust of 200 kN, was manufactured in Slovenia, whereas the electronic part was bought from the company Gouda. The electronic part consists of an analogue graphic recorder, a measuring unit and two piezocones. The cones/piezocones can measure cone resistances q_c of up to 50 MPa or 100 MPa, respectively, maximum sleeve friction f_s of up to 0.5 MPa, and pore pressures u of up to 1.4 MPa. The analogue signals from the transducers are plotted on strip chart records. The results can also be read out from the unit, simultaneously.

Since 1988, cone penetration tests have also been carried out using considerably up-to-dated equipment, produced by the company ISMES. In principle, this penetrometer consists of a penetration system, a measuring device and a data acquisition system. The equipment is mounted on a truck of domestic production. The total weight of the equipment is 136 kN.

The penetration system enables penetration at the prescribed rate of 2 cm/s, with a possible maximum total thrust of 200 kN. A maximum depth of 78 m has been reached.

The measuring system enables measurements of q_c up to 80 MPa, f_s up to 0.7 MPa, pore pressures up to 3.5 MPa, and of inclinations within the range $\pm 20^\circ$. The porous element is located on shaft, between cone and sleeve.

The data acquisition system enables recording of the results on a analogue graphic recorder, and at the same time in digital format on a computer. The results are recorded at depth intervals of approximately 1.5 cm.

Some details regarding this type of penetrometer are evident from Bruzzi (1982) and Bruzzi (1987).

The newer assembly has been additionally equipped with a "Mostap 35" type soil sampler, produced by A.P. v.d.Berg. With this device practically undisturbed samples of fine grained soils, with a diameter 35 mm and a length of up to 1 m can be obtained. In the opinion by the authors, this kind of soil sampling is a very useful supplement to traditional penetration testing.

3.2 Test procedure

The main operating phases of the newer equipment, where a relatively skilled operator is needed, are the following:

- activation of the hydraulic system (from the motor of the truck)
- stabilizing of the truck
- switching on the current converter, the analogue graphic recorder and the computer
- arrangement of the cone or piezocone
- connection of the cone to the cable and to the first rod
- input of the main data about the test into the computer
- determination of the first offset
- penetration

With this equipment a dissipation test can be executed after interruption of steady cone penetration. The course of this test can be inspected in graphic and digitized format on the computer. When the desired depth of penetration has been reached, pulling out of the rods and measurement of the final offset follows.

The cones of the older equipment are calibrated in Slovenia, whereas the newer cones are regularly calibrated by the manufacturer. On these occasions the whole equipment is inspected and serviced as necessary.

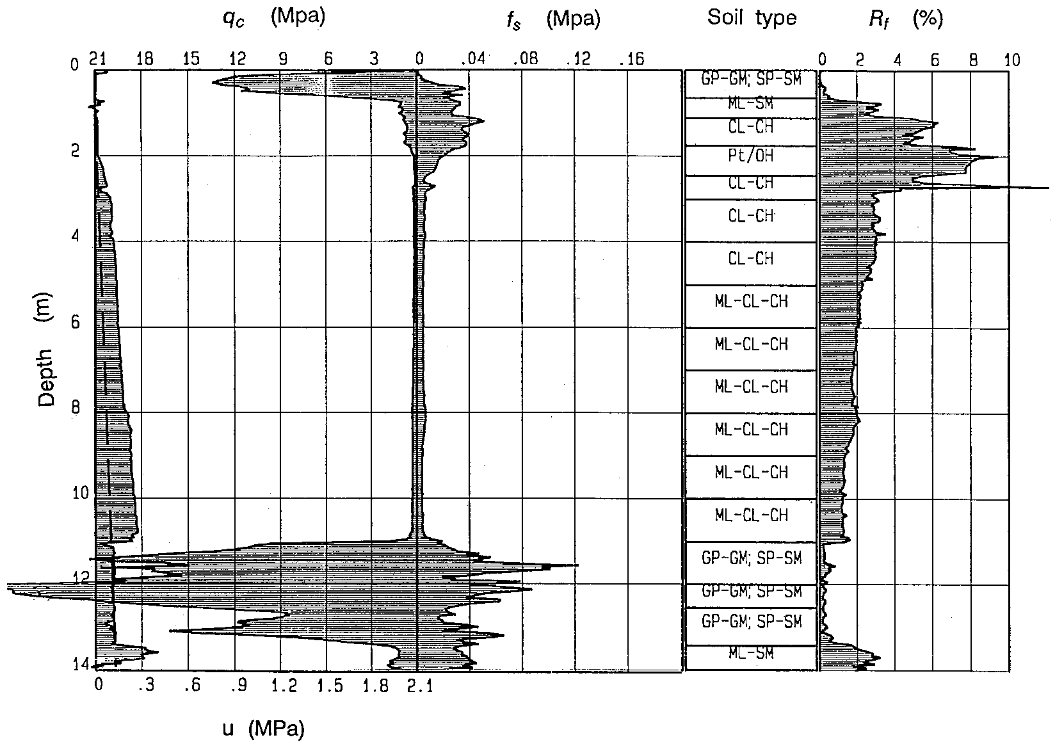
From the calibration certificates of the newer equipment, it can be seen that calibration is executed in three step-by-step loading and unloading operations from which sensitivity and the parameters showing accuracy are calculated. The typical deviations in laboratory conditions expressed as percentage of the full scale are: $q_c \sim 0.1\%$, $f_s \sim 0.8\%$, $u \sim 0.3\%$ and inclination $\sim 1.4\%$.

3.3 Correction and presentation of the test results

As already mentioned, the results obtained using the older assembly can be read during penetration both in the digitized format of the analogue values, and graphically from the recorder.

The newer penetrometer is equipped with software developed in Slovenia for digitalization of the analogue values, taking into account the offsets. It enables a graphical presentation of the test and has the following main elements:

- correction of the depth of penetration taking into account the inclination values
- shifting of the measured f_s on the location of the measured q_c



Depth (m)		q_c (MPa)	var (%)	f_s (MPa)	var (%)	U (MPa)	var (%)	R_f (%)	S_u (KPa)	S_t	φ (o)	Dr (%)	M (MPa)	Soil type (ASTM D 2487)
0.00	0.65	9.63	39	0.0155	89	0.00	48	0.2			37	100	32.1	GP-GM;SP-SM
0.65	1.12	1.30	55	0.0291	15	0.01	190	2.2	77.4	2.7			2.6	ML-SM
1.12	1.76	0.74	12	0.0394	14	0.02	12	5.3	43.9	1.1			3.0	CL-CH
1.76	2.45	0.26	44	0.0197	39	0.03	49	7.6	14.3	0.7			0.3	Pt/OH
2.45	3.01	0.18	42	0.0079	37	0.06	31	4.4	9.0	1.2			0.9	CL-CH
3.01	4.01	0.19	8	0.0058	9	0.10	7	3.0	9.5	1.6			1.0	CL-CH
4.01	5.02	0.18	8	0.0050	6	0.13	5	2.8	8.1	1.6			0.9	CL-CH
5.02	6.00	0.21	6	0.0046	5	0.14	3	2.1	9.6	2.1			1.1	ML-CL-CH
6.00	7.00	0.24	4	0.0046	4	0.16	3	1.9	10.8	2.4			1.2	ML-CL-CH
7.00	8.01	0.29	8	0.0051	13	0.18	4	1.8	12.8	2.5			1.4	ML-CL-CH
8.01	9.00	0.30	6	0.0051	18	0.22	4	1.7	12.7	2.5			1.4	ML-CL-CH
9.00	10.01	0.30	4	0.0038	6	0.24	3	1.3	12.6	3.3			1.5	ML-CL-CH
10.01	11.02	0.39	38	0.0049	50	0.26	5	1.3	16.8	3.4			1.8	ML-CL-CH
11.02	12.01	16.42	25	0.0517	63	0.11	24	0.3			39	75	49.0	GP-GM;SP-SM
12.01	12.54	23.88	17	0.0539	36	0.11	13	0.2			41	84	67.7	GP-GM;SP-SM
12.54	13.43	10.21	33	0.0327	52	0.11	22	0.3			37	61	33.5	GP-GM;SP-SM
13.43	14.00	1.46	23	0.0350	19	0.23	59	2.4	79.6	2.3			3.2	ML-SM

Note: var=(st.dev./mean)·100%

Fig. 2. CPTU interpretation of a typical record from the Ljubljana Marshlands

- correction of q_c due to the pore pressures, if measured
- graphical presentation of the test
- soil classification and estimation of the basic geotechnical parameters for individual depth sections of the profile.

The same software is also used for the interpretation of the CPTU obtained using the older assembly, but after previous digitalization of the results.

In Fig. 2 an example of our interpretation of CPTU is given.

As a young state, Slovenia is presently preparing its own standards. Since October 1991 a special bureau within Ministry of Science and Technology has been concerned with the replacement of ex Yugoslav standards by Slovenian standards. The last Yugoslav standard regarding CPT was published in 1989, with the designation: JUS U.B1.031.

At the moment the ASTM standard D 3441-86, and the Report of ISSMFE (1989), are used in Slovenia.

4. INTERPRETATION OF TEST RESULTS

4.1 Soil classification and stratigraphy

On the basis of the measured tip resistance q_c and the friction ratio $R_f=f_s/q_c \cdot 100\%$ soil classification is made by using our own computer program. Up until now it has been based on the charts developed by Robertson and Campanella (1983). We can estimate individual depth intervals or layers as they are expected, or we can divide the whole profile into any number of equidistant sections. The suitability of the layer distribution and its homogeneity is verified by statistical indices. In every layer the total volume mass is determined with regard to classification and its expected values, e.g. Hunt (1986, Tables 3.5 and 3.7). These values are utilized in the calculation of the overburden pressure σ_{vo}.

4.2 Soil parameters and other data

In order to obtain the average soil

Table 1. Equations used in our interpretations of CPTU data

SOIL PARAMETER	RELATIONSHIP	REFERENCE
Undrained Shear Strength S_u	Cohesive soil: $S_u=(q_t-\sigma_{vo})/N_c=q_{net}/N_c$ $N_c=16$ (normally consolidated)	Meigh (1987)
Angle of friction φ'	Cohesionless soil: $\varphi'=28+2.5\cdot(q_t)^{0.5}$ /q_t in MPa/	Mayerhof (1976)
Constrained modulus M	Cohesive soil: $M=\alpha\cdot q_t$ $(\alpha=1\div8)$ Clean sands: $M=2.5(q_t+3.2)$ Clayey sands: $M=1.7(q_t+1.6)$ /q_t in MPa/	Sanglerat (1979) Mitchell (1975) Webb et al. (1982)
Sensitivity S_t	Cohesive soil: $S_t=S_u/f_s$	
Relative density D_r	Cohesionless soil: $D_r=0.98+66\cdot log(q_t/(\sigma'_{vo})^{0.5}))$ q_t and σ'_{vo} in t/m^2	Jamiolkowsky et al. (1985)

Note: q_t is the corrected value of q_c due to the effect of pore pressure properties of the individual layers or depth intervals the above mentioned computer program takes into account the empirical relationships which are listed in Table 1.

The pore pressures generated during penetration in CPTU, and the rate of excess pore pressure dissipation after interruption of penetration, are good qualitative measures of relative permeability and the drainage conditions in the soils. The coefficient of consolidation c_h and the coefficient of permeability k_h can be satisfactorily deduced from dissipation curves according to Levadoux and Baligh (1986).

5. THE USE OF CPT IN GEOTECHNICAL DESIGN

5.1 Direct methods

In our country this approach is used in the prediction of pile bearing capacity for driven piles. For this purpose we have adopted the method after Zhou et al (1982), which in principle includes corrected values of q_t and f_s. We have also developed our own software, based on comparisons between CPT data and the results of dynamic tests (Strniša and Ajdič, 1991). The latter method proved to be helpful for predicting final pile bearing capacity and bearing capacity at the end of driving.

5.2 Indirect methods

CPT and CPTU are mainly used to determine the stratigraphy of soft to stiff soil layers, and the depth of bedrock. CPTU is an unsurpassed method for obtaining substantial information about drainage conditions and equilibrium ground water conditions. In situ penetration testing reduces the need for conventional boreholes but preferably both methods should be combined. CPTU can considerably supplement, and partially replace, longer-lasting laboratory tests.

The cone penetration tests are very important in the determination of undrained shear strength for ordinary design projects, too. We have had good experience in using this method to investigate the increase of subsoil bearing capacity due to consolidation.

In stability problems the use of CPTU has become an indispensable tool. All the above mentioned properties (stratigraphy, strength properties and drainage conditions) are of great interest in the stability analysis of naturally and artificially shaped slopes, and in bearing capacity predictions beneath embankments and other structures.

In our practice there is also interest in the assessment of deformation and consolidation properties from CPTU results. The approximation of settlements has been based on constrained moduli, obtained using the relations presented in Table 1. For soft, normally consolidated cohesive soils, in future the correlations which are more related to the stress conditions will be used. This matter is presented in section 6, and in more detail in (Gaberc et al, 1995-a).

The information about the drainage conditions, gained during steady penetration are also, according to our experience, of great importance for the prediction of the consolidation process. Furthermore, the evaluation of the consolidation properties c_h and k_h provides valuable additional information for geotechnical design, i.e. of vertical drains and other modes of ground improvement.

A case of the successful use of CPTU results was demonstrated during the construction of a breakwater embankment in the Izola Marina on the Adriatic coast. This rather extensive structure was designed on marine clayey deposits of low bearing capacity. The results of some previous field and laboratory tests of the bearing capacity of the subsoils were inapplicable due to unusual large scatter. In the first study of the foundation design we took advantage of an older, general statistical study of the different results of the undrained shear strength S_u of marine sediments. Before a final decision was taken there was insufficient time for longer-lasting drilling and laboratory works, so CPTU provided the only "emergency exit". The results of S_u confirmed the average values from the general statistical study, and they demonstrated a rather favourable increase in S_u with depth. On this basis, the final stability analysis was carried out, and a shallow foundation was chosen with a prescribed rate of filling. Control of S_u values made 6 months after the beginning of intensive loading, showed a significant increase in the bearing capacity beneath the embankment, which was within the range of expected values. After that construction works were continued in steps, and have recently been completed. The observed settlements are also in good agreement with the predicted values, taking into account the constrained moduli from CPTU results according to Gaberc et al (1995-a).

6. COMPARISON AND CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

6.1 Undrained shear strength

CPT results (q_c or q_t respectively) and the undrained shear strength of soft cohesive soils from vane tests were correlated. In Fig. 3 to 5 some examples from Ajdič (1993) are given. In Fig. 3 the determination of the "cone factor" N_c for soft soil up to a depth of 30 m in the Mirna valley (Istria-Croatia), is presented. It is based on correlation between CPT's $q_{net} = q_t\text{-}\sigma_{vo}$ and vane test's shear strength S_u. The statistical relation is expressed in the form $S_u = a + b \cdot q_{net}$. Since the regression coefficient a is close to zero, the "cone factor" N_c, defined as $N_c = q_{net}/S_u$, is evidently the reciprocal value of the regression coefficient b: $N_c = 1/b$. In the given case, N_c equals 16.7. Results of the same order can be demonstrated by statistics, in which the field vane tests as

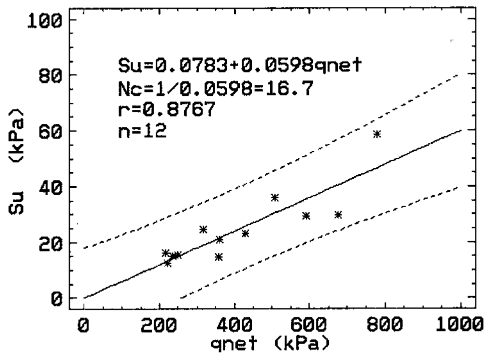


Fig.3. Determination of the cone factor Nc; Mirna valley, 3 m - 30 m (vane tests)

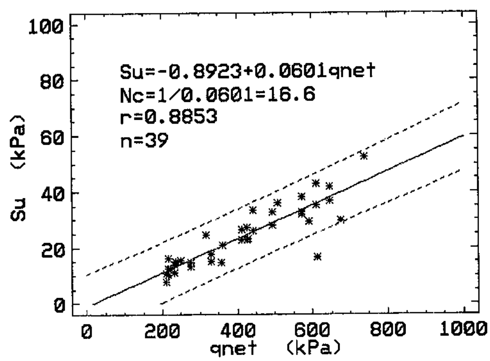


Fig.4. Determination of the cone factor Nc; Mirna valley, 3 m - 30 m (all tests)

well as laboratory tests of undrained shear strength are included (Fig. 4). In Fig. 5 a comparison between the results of determination of S_u by means of CPT (for N_c=16.5) and three different vane tests (points) at a typical location on the Ljubljana Marshlands (soft, normally consolidated, silty to clayey soils) is presented. It can be seen that the estimation of undrained shear strength from CPT is within the 95% confidence region due to vane tests, but the range of prediction is relatively wide because of the deviation in the results of the vane tests.

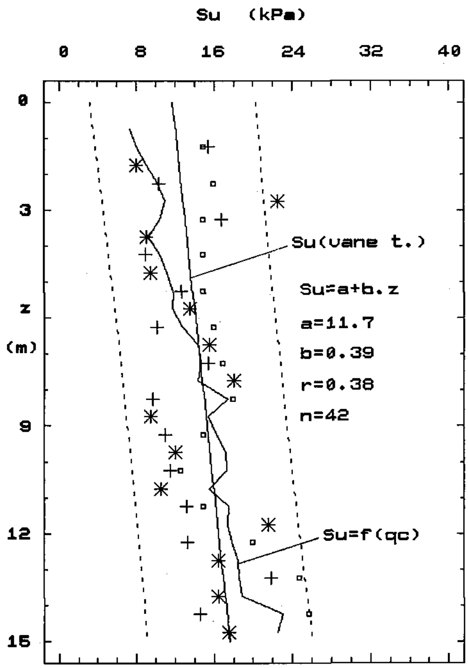


Fig.5. Su from vane tests and CPT, on the Ljubljana Marshlands

6.2 Constrained modulus

In an experimental study of compression moduli obtained by CPT in soft, normally consolidated, silty to clayey sediments (Gaberc et al, 1995-a) a linear correlation between cone resistance (q_c or q_t) and the tangent modulus of compressibility M_o from the oedometer curve at the adequate effective overburden stress σ_{vo}' was demonstrated: $M_o = \alpha_o \cdot q_t$. The size of the coefficient α_o depends on the kind of soil, and it increases with decreasing plasticity $(1.5 < \alpha_0 < 5)$. In the examined soft subsoils the measured cone resistance did not exceed 1 MPa. The modulus of compressibility M at any given stress step $\Delta\sigma_v'$ is determined according to Senneset et al, (1988): $M = M_o((\sigma'_{vo} + \Delta\sigma'_v/2)/\sigma'_{vo})^{0.5}$. The results of some recent settlement measurements have confirmed the validity of the proposed correlations. However, the need for further experimental verification is clear.

6.3 Excess pore pressure dissipation tests

In an experimental study of the vertical and horizontal permeability of Ljubljana Marshland subsoils (Gaberc et al, 1995-b) the reliability of permeability determination based on dissipation tests was studied. Three methods were compared: the laboratory permeability tests in oedometers; permeability, back-calculated from field measurements of excess pore pressures during radial consolidation around vertical drains by piezometers; and in situ dissipation tests by CPTU. In both of the latter cases horizontal permeability dominates. A rather wide range of k_h values from the dissipation tests arises from different approaches given by authors: Levadoux and Baligh (1986), Tumay et al (1982), Houlsby and Teh (1988), Gupta and Davidson (1986). In all the interpretations presented, a 50% dissipation level was taken into account. Nevertheless, the results of k_h values from both in situ measurements have shown satisfactory agreement. A comparison with laboratory tests has proved some degree of disagreement in relation to the results of in situ measurements, which in our opinion results from the non-homogeneity of the tested soils. The average values of the laboratory permeability obtained by the increased number of the laboratory tests has shown a better agreement between all three methods.

7. MAJOR AREAS FOR RESEARCH ACTIVITIES

In order to make as full use as possible of the data obtained during penetration into the ground, our research activity is oriented towards the relationship between the measured values and the strength and deformation parameters. The major areas of recent studies have been presented in previous sections. They deal with deep foundations, the proper interpretation of undrained shear strength, and with attempts to approximate compression moduli and settlements together with their time development of normally consolidated soft subsoils.

In analyses of undrained shear strength and its increase due to consolidation beneath embankments, the importance of the horizontal stress in the soil has been demonstrated. So the problem of proper normalization also represents an interesting subject for research activity.

8. FUTURE TRENDS AND NEW DEVELOPMENTS

In order to improve the quality of CPT, the following advances in development of equipment should be made:

- replacing the existing measuring system with a cordless one
- improving mobility of the truck by adding tracks
- completion of the equipment with the possibility of measuring both CPT and some basic environmental parameters.

With the aim of improving the reliability of interpretation, the following subjects of experimental research should be further investigated:

- completion of the existing soil classification procedure with some new soil behavior type systems; both normalized CPT parameters (e.g. Olsen et al, 1988) and excess pore pressures (e.g. Robertson, 1990) should be included
- introduction of statistical tests for equivalence of several CPT records (e.g. Eduardo et al, 1975)
- evaluation of undrained shear strength with respect to excess pore pressure
- correlation between cone resistance and the tangent compression modulus at the effective overburden pressure M_0 should be continued and extended over the actually examined range of cone resistance (q_c > 1 Mpa)
- further investigation of soil permeability by dissipation tests and comparisons with the results of laboratory test.

In order to improve the interpretation of the presented soil classification and geotechnical parameters, an essentially higher percentage of pore pressure measurements is needed. Up until now only 10% of measurements of this kind have been realized.

New solutions should be implemented to the routine interpretations of CPT measurements, in order to increase the applicability and reliability of this in situ investigation tool.

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National Report - South Africa

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1 INTRODUCTION

Penetration testing has changed little in South Africa since the previous national report and the current report should therefore be seen as an update of the information which was recorded at ISOPT-1.

The following sections describe the various tests, assess adherence to standards, the use of the results and an estimate of the quantity of testing carried out in South Africa annually.

1.1 Dynamic Probe - Light

The equipment is as described at ISOPT-1, ie 8 kg weight falling through 580mm. Measurement is either by counting number of blows for 100mm penetration for softer materials or direct measurement of penetration in mm per blow.

The former tends to be for general foundation work in natural and compacted soils and the latter for road pavements. Interpretation is generally through well established correlations of penetration rate (mm/blow) with California Bearing Ratios; for foundations the CBR values are correlated with bearing capacity for footings through empirical equations.

Pavement engineers use the method as a direct measure of pavement conditions and extensive work has been done and reported on this.

The method is usually designed as the DCP in South Africa - Dynamic Cone Penetrometer and will probably remain as this. The penetration depth is usually confined to 1m although extension rods are occasionally used to add a further metre. It impossible to estimate the total number of "DCP"'s carried out annually in South Africa - every road authority and every consultant uses the method and most general civil consultants use it for checking insitu densities of natural and compacted soils.

Probably the number of tests is in the order of tens of thousands per annum.

1.2 Dynamic Probe - Super Heavy

The South African equipment was described for ISOPT-1 and essentially comprises an SPT weight and anvil assembly with the standard fall distance. There is generally no automatic weight release mechanism ie a hand controlled winch is used. The loose point is a 60°, 50mm diameter cone. The method is often designated as continuous SPT or simply CPT, although latterly DPSH is becoming recognized as correct.

DPSH testing is generally carried out at the inland areas rather than at the coastal zones and is considered as a very economical substitute for SPT's.

It is usually conducted during site investigations when there is a possibility of piling being used as the foundation method. Interpretation is generally on the basis of conventional

SPT methods with some devaluation of the measured N values.

The method is not standardised in South Africa although the equipment used by all practitioners is virtually identical. In general the main users follow the ISOPT-1 published recommended method.

The total quantity of DPSH testing per annum is 6000m; typically depths at each test position are about 20m.

1.3 Standard Penetration Test

The SPT is carried out generally according to ASTM D1586-64T but not with strict regard to casing size. Practically without exception automatic trip hammer mechanisms are used. No penetration depth recording systems are in use and depth measurement usually comprises simply temporary felt tip pen markings on the drill rods at 75mm intervals; the test is always carried out for 450mm if penetration permits.

Interpretation methods are generally as in the international literature ie there are no peculiarly South African methods. SPT's are carried out, as elsewhere, during geotechnical investigations for all civil engineering purposes in materials varying from recent alluvial deposits to deeply weathered residual rocks.

It is estimated that approximately 10000 individual SPT's are carried out each year.

2 CPT

Conventional CPT's using the reference test equipment and procedures are carried out for geotechnical investigations generally in the coastal zones. It is often called Dutch Probing and the standard cone and friction sleeve equipment is used with measurements by two pressure guages. The machines do not usually have sophisticated mechanisms to ensure constant rates of penetration, but the rates are normally fairly well controlled by experienced operators.

It is estimated that about 2000m of probing is carried out each year.

3 CPTU

All CPTU work is carried out according to the ISOPT-1 reference test procedures. The cones are manufactured in South Africa and without exception have a single filter element at the base of the cone. No friction sleeves are utilized. The measuring system operates through a laptop computer and sometimes a chart recorder is used in parallel.

The rigs are standard 10 ton Delft machines since these are most suitable for the conditions. The sites are either at recent alluvial deposits or at mine tailings dams. The latter now dominates the CPTU work particularly since the safety of many existing tailings dams is being re evaluated.

Approximately 3000m of CPTU work is carried out each year.

The use of cone penetration testing in Spain (NR4)

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SYNOPSIS: In Spain, due to its particular geological evolution there are not many geographical zones composed by the so called soft soils; nevertheless, since some years ago it has been necessary to undertake many constructions on such type of soils, specially road and railway works.

As a consequence, during the last ten years it has become evident an important increase in the use of exploratory techniques specially designed for this kind of soils as the CPT and CPTU.

In this paper the equipments and techniques most frequently used for this exploration work as well as its application in actual cases and the interpretation models of the results are included. Particularly, their use in roadworks on soft soils in Spain and its application in techniques of ground improvement are pointed out.

1. BRIEF GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF THE REGION

Soft soils are associated with low-energy levelpoorly drained sedimentary environments, which are conducive to the accumulation ofvery finely-graded sediments, the neoformation of clayey minerals or the genesis of soils that are organic in nature. Furthermore, these materials are not very compact, which means subsurface (or underwater) deposits and a short geological life. The geological evolution of Spain during the Quaternary period has determined that this type of formation is clearly defined in space and time, being restricted to environ-ments in which the above-mentioned circumstances concurred.

The sedimentogenic factors that determine the appearance of soft soils, can be found in both continental and marine environments. However, the existence of this type of soil in Spain, is mainly linked to environments associated with a continental-marine transition. Characteristic examples of such areas include coastal plains near river estuaries, deltas and lagoons.

Aerial sediment (of which very little exists in Spain) is characteristic of continental environments, as is sediment associated with lacustrine environments, and these can contain soft materials. Lacustrine environments are not very common and restricted to specific areas, but they are varied in type (ranging from glacial lakes in the Pyrenees to volcanic lakes on the Canary Isles). Reservoir basins, where the sediments are frequently clayey, would have to be included in this category. (It was possible to carry out detailed research into soft soils of this type, after the emptying of the Proserpina Reservoir, near Mérida, which dates back to Roman times).

In endorheic or semi-endorheic morphological depressions, which because of the scarcity of precipitation in most of Spain only occasional have a water surface, intense edaphic activity often takes place, and thisgives rise to the formation of clayey material, but such soils of edaphic origin are not very hard. However, these soils are fairly common in the southern half of the Iberian Peninsula, but from a geotechnical viewpoint, they are rarely regarded as soft soils.

In mountainous areas of Northern Spain, and at high elevations in the mountain ranges of the south, both of which were affected by periglacial conditions, peat bogs are occasionally to be found, in which the mud is rich in organic material. However, there are not many zones of this type.

The same applies to soft materials that are associated with karstic phenomena. Although outcrops of limestone and gypsum cover extensive areas of Spain, and both these materials are susceptible to karstification, the geomorphological evolution of those rock masses has hardly ever led to a generalised formation of soft soils. They are associated with punch-bowls (poljes).

River floodplains deposit fine-graded material, but the formation of large extensions of soft soils only occurred in upper or middle reaches of rivers, when geomorphological alterations to the river valley gave rise to lacustrine or marshy environments, and this only happened occasionally. One such example is the polje at Medinaceli (Soria), in the upper valley of the River Jalón which, although it is narrow, stretches for several kilometres; it eveloped in clayey and gypsum formations dating back to the Triassic period, upon which up to 40 m of soft soils have accumulated.

Fine sediments are sometimes abundant in the lower reaches of Spanish rivers that flow into the Mediterranean. Owing to the frequent sea level changes during the Quaternary period (because of neotectonic and eustatic activity), it is often the case that marine materials are found close to those of continental origin, as occurs in the coastal area of the Spanish Levante, especially near the estuary of the River Segura. Lagoons, such as La Albufera (Valencia), *exist in this* region, as well as the salt pans of Santa Pola and Torrevieja (Alicante) and the Mar Menor (Murcia), and there are also silted up lagoons such as the "marjal" de Pego (Valencia).

Marine intrusion, through the tides, plays an important role in the genesis of soft soils on the South Atlantic Coast of Spain. There is a degree of development in the Bay of Cádiz, in the form of creeks. A significant yield of continental sediment enhances the development of soft soils that, in some cases, can cover extensive areas to a considerable depth. Such cases include the marshland of the Rivers Guadalquivir, Odiel and Tinto (Huelva).

Finally, the soft sediments associated with deltas, are only highly developed in the River Ebro, whereas in the marine influence of other open-estuary type rivers, (the rias of Galicia and the rivers of the Cantabric Coast), only small amounts of soft soils have accumulated.

2. TYPE OF PENETRATION TESTING AND OTHER INVESTIGATION METHODS USED

Where penetration tests in Spain are concerned, a clear distinction is made between the dynamic and static types used.

The dynamic group includes the classic S.P.T. (Standard Penetration Test), continuous dynamic penetration tests also widespread; depending on the degree of energy transmitted, the latter type are broken down into semiheavy, heavy and very-heavy. The latter two are currently the ones which are most extensively used, because they are best suited to Spanish soil characteristics (often encrusted on the surface, semi-saturated and having a consistency above that of soft soils in general).

However, static penetration tests, with or without pore-water pressure measurement, are reserved for cases of works of major importance that are being carried out on soft soils, in which attempts must be made to design ground improvement techniques, construction of embankments in stages, piles, etc.

Apart from other specific techniques, such as geophysical ones (a field in which a considerable amount of research is taking place, including the perfecting of electric tomography tests, surface waves, georadar, etc, (especially at the Laboratorio de Geotecnia of the CEDEX), and the laboratory tests themselves (in which case, and especially where dynamic tests are concerned (the CEDEX is also involved in installation, development and research into high-technology equipment), pressiometer tests could be mentioned as yet another technique that is currently being applied extensively in Spain in geotechnical investigation.

In any case, it ought to be pointed out, that for quantitatively determining parameters and geotechnical calculations, only S.P.T. and static penetration test data are used. Continuous dynamic penetration tests are only used for qualitative purposes (to determine the presence of fills, soft soils, ground defects such as hollows and cavities, etc).

3. TYPES OF EQUIPMENT USED

The latest C.P.T. equipment to be used in Spain, manufactured by the companies FUGRO and HOGLEN TOGLER, is the 10 cm^2 and 15 cm^2 electric piezocone versions with a conical tip.

The equipment concerned is a 10 Ton electronic piezocone, consisting of a 10 cm^2 60° drill with a conical tip, whose lateral friction sleeve is 10 cm from the tip. It is inserted into the ground by means of a mechanical thruster consisting of a thrusting head that slides along two endless screws driven by a hydraulic motor. Two sensors have been incorporated with a view to controlling operation and the testing one of which is of the encode type, to measure the driving velocity, and the other of the pressure type, situated at the upper inlet for the motor so that the force being exerted upon the rods can be measured. Driving takes place at a constant velocity, depending of type of soil. The data is automatically recorded on a computer especially equipped for fieldwork. The assembly has been mounted on an 18 Ton ballasted lorry, to ensure a 10 ton reaction. Rigs of the flexifloat type have been used for marine works.

The pore pressure is recorded with a calibrated pressure transductor. With a view to ensuring that the measurements are not affected by the presence of air, the circuit is deaerated by injecting glycerine fluid. The process used to saturate the porous stones, manufactured with highly porous material of the "bion" type, consists of injecting liquid glycerine under pressure, after vacuuming. The stones can be placed in two positions: either at the base of the conical tip or halfway along it, but the equipment appears to respond better and show a greater degree of sensitivity in the latter position.

The test procedure consists of inserting a drill into the ground, ensuring that the porous stone does not lose its saturation (for which it is covered with a flexible membrane, sometimes resorting to predrilling techniques) and maintaining a constant velocity. The process is automatically halted so that successive rods can be added and for the programmed dissipation to be conducted.

The equipment makes it possible to simultaneously record the tip resistance, sleeve friction, pore pressure and drill inclination. The tip resistance values have to be corrected, as a function of the pore pressure, so the relationship between the area of the cone base and net thrust area must be taken into account. The drill has an area ratio of 0.5.

4. INTERPRETATION OF THE RESULTS

The use of this type of test in Spain, has been restricted to soft and relatively soft soils of the clayey type (marsh, Quaternary deposits of both freshwater and marine origin, etc) and fine grain soils, with fairly low densities (recent deposits, etc). The results obtained have mainly been used to define the stratigraphic profiles of the ground, the geotechnical properties and especially the consolidation velocity.

Classification of Soil Types

The empirical classifications published in the technical bibliography are used for estimating soil profiles, and such classifications are based on the relationships that exist between the friction coefficient (sleeve friction/tip resistance in per cent) and the tip resistance (Robertson et al. (1986) and Meigh (1987)).

It is also normal, especially in the case of cohesive soil surveys, to use the pore pressure parameter $B_q = (u_{max} - u_0/q_t - \sigma_{vo})$. It has been demonstrated that the values that can be obtained with the pore pressure equipment are extremely accurate, and show the stratigraphical and interlayering changes much better than by using parameters that are exclusively concerned with resistance.

The abaci currently used in Spain are: the one proposed by Senneset et al. (1989), which classifies the soil into seven different groups as a function of parameter B_q and the tip resistance (q_T) and that proposed by Robertson et al. (1986).

Cohesive Soils

The undrained shear strength is calculated from the tip resistance, taking into account Sanglerat's Theory (1972), according to which: $q_T = S_u N_k + \sigma_{vo}$, where N_k is the cone factor that must be determined for each soil type. In practice, factor N_k is calculated from correlations between q_T and S_u, obtained using "in situ" vane tests and correlations with the pressiometer test. $N_k = 15$ is the average value generally taken, but values of around 16 and 17 have also been used. On occasions, undrained strength values (S_u) have been estimated in normally consolidated cohesive soils, on the basis of the overpressure measured at the tip, using the expression: $S_u = \Delta u/16$.

Granular Soils

The correlations proposed by Baldi et al. (1986) for normally consolidated moderately compressible soils, are normally used to estimate the relative density (D_r) and the angle of internal friction for granular type soils.

Interpretation of the Consolidation Parameters

The pore pressure dissipation tests that can be carried out when the piezocone tests are being conducted at the different depths programmed, make it possible to obtain information concerning the drainage conditions of a soil and its consolidation parameters.

Work in Spain is being carried out following the lines set out by Houlsby and Teh (1988), to calculate the coefficients of horizontal consolidation of the ground deduced from the dissipation curves that are plotted by the equipment.

The expression that is used to obtain the coefficient of horizontal consolidation C_h as a function of the dissipation time of the excess pore pressure generated during the driving process, is as follows:

$$C_h = \frac{T* \, R^2 \sqrt{I_r}}{t}$$

where:
T* : time factor
R : radius of the conical tip
I_r : stiffness coefficient
t : time

In accordance with the theoretical curves of dissipation, which is the solution proposed by these authors, time factors of T* = 0.245 were adopted for 50% of the consolidation when the porous stone was placed at the base of the conical tip, and T* = 0.118 when the porous stone was placed halfway down the conical tip.

Work is also being conducted on the consolidation model proposed by Baligh and Levadoux, (1986), based on the unconnected and linear consolidation theory, applied for the normalised initial distribution of the excess pore pressure in the normally consolidation clay in Boston. The method consists of comparing the overpressure dissipation measurements which are obtained depending on the different positions of the porous stone.

The coefficient of horizontal consolidation for any degree of consolidation, is expressed as follows:

$$C_h = \frac{T \cdot R^2}{t}$$

where:
T : time factor
R : radius of the conical tip
t : time

and the following time factors are being used: T = 5.6 for 50% of the consolidation when the porous stone was placed at the base of the conical tip, and T = 3.7 when the porous stone was placed halfway down the conical tip.

Modulus of Deformation, E_u

The modulus of deformation E_u is usually estimated in accordance with the empirical correlation:

$$E_u = \alpha\ Q_c$$

where:

Q_c : resistance at the tip

α : experimental coefficient

Some values of α have been deduced experimentally (Oteo, C. 1995), for the specific case of muddy clay of marine origin (lower reaches of the River Segura, in Alicante-Spain) and these range from 0.8 in the zones were fine soils are predominant, to 3 in the zones were sand is abundant.

5. USE OF C.P.T. IN GEOTECHNICAL DESIGN

Where most applications are concerned, it is common practice to use tests to determine the corresponding geotechnical parameters (shear strength, deformability, etc), through which the relevant calculations will later be made (pile bearing capacity, embankment settlement, etc). It is far less common that the data obtained from the tests is directly applied in the calculations and dimensioning concerned.

However, it is customary for these test results to be compared with data from other techniques, above all S.P.T. and laboratory tests, as well as pressiometer test data if such data is available.

Most applications are used in soft soil sites, most commonly in either in pile design and embankment or overburdening study upon these. They are also used a great deal in determining the lithological profile of the subsoil, to find out the constituents of the different layers, whether they are sandier, i.e. permeable, or whether they are clayey, i.e. impermeable.

In recent years, they have been extensively used in the soft soil and roadworks fields, where they have been mainly put to use in the foundations of the structures, as in the case of embankments overlying soft soils. In this latter case, a special reference should be made to ground improvement techniques based on stone columns and prefabricated drains.

6. COMPARISON AND CORRELATION OF C.P.T. WITH OTHER INVESTIGATIONS

When C.P.T. type tests are used for site investigation in Spain, it is common practice to conduct other more classic and widely used tests simultaneously, such as: the Standard Penetration Test and the continuous dynamic penetration tests (dynamic probing). In the case of soft soils, the Vane Test is often used. These types of test are performed for back up and comparison purposes when interpreting the results of the C.P.T. as well as allowing for the establishment of correlations, both general and specific, for the soil types tested. It is also important for the purpose of comparison, to take undisturbed samples for laboratory tests. These two ways of testing have made it possible to deduce some specific correlations and coefficients for our soils.

Fig. 1, shows a comparison between the undrained shear strength (S_u) values obtained from both a piezocone test and from triaxial UU and CU type tests, respectively, for undisturbed samples taken from a borehole close to the test site. Furthermore, Vane-type Tests and S.P.T. tests were performed in this borehole, and these are also included in the figure.

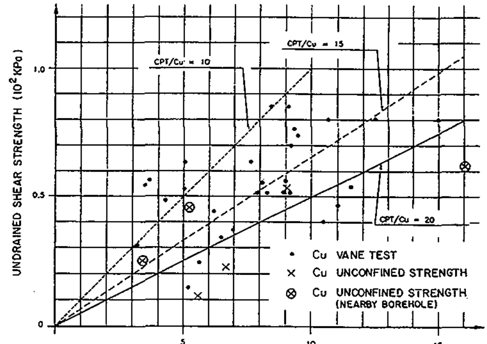


Fig. 1

Although much more information can be deduced from the piezocone test, it is possible to detect that there is a certain tendency for the short term strength derived from the UU triaxial tests to be somewhat greater than that shown by the static penetration test (about 70-80% lower in the latter case). This observation can likewise be applied to the "in situ" Vane Test" (once the Bjerrun correction has been made).

As far as S.P.T. is concerned, if a direct correspondence is to be made between the coefficient of drive resistance and the undrained shear strength deduced from the penetration tests:

$$\left(S_u = \frac{N_{SPT}}{\alpha}\right)$$

the correlation coefficients (α) should be somewhat greater (α = 8-10) than those which are normal for clayey or silty soils (amongst other factors, the real energy transmitted in the S.P.T. would have to be taken into account, and this could turn out to be rather lower than the theoretical energy transmitted).

In the figure 2, an attempt has been made, for this particular site, to evaluate the cone factor using laboratory tests so a better adjustment can be obtained between the correlation and the soil concerned. As can be observed, these factors are equivalent to those deducted from the correlations normally applied to the penetration tests.

The figure 3 shows the results obtained for the coefficients of consolidation deduced from the edometric tests (C_v) performed in the laboratory and the dissipation in the piezocone (C_h) and, it can be deduced that the relationship between the coefficient of vertical relation of the edometer and the horizontal of the piezocone, is about C_h = (10-20)C_v. This relationship is consistent with the published data and, amongst other data, it includes the known anisotropy of the ground for this purpose.

The figure 4, shows the results of some of the dissipation curves that correspond to a specific test for muddy clay at Puerto de Santa María (Cádiz). The coefficients of consolidation have been deduced using the Houlsby and Teh (C_{hpz}) and Baligh and Levadoux (C_h) methods, for 50% consolidation, the values of the latter being more conservative than the former ones (frequently the Houlsby and Teh values have proved to be more representative of the real behaviour of the ground).

7. MAJOR AREAS FOR RESEARCH ACTIVITY

Some of the lines of research that have been developed in Spain recently, concern the determination of the consolidation parameters by means of dissipation tests performed with the piezocone. The anisotropy factor in the ground is specifically analysed, together with the relationship between this and the permeability data obtained depending on whether the porous stone is placed at the tip, etc. Work is also being carried out on evaluating the deformability of the subsoil, using static penetration data, and comparing this with data from instrumented and auscultated works.

Another area in which investigation is currently being developed, concerns the determination of the degree of consolidation of hydraulic fills, on the basis of data obtained with the piezocone, in which a relationship is being established between the geomechanical parameters of tip resistance with the permeability determined in the test and the pore pressures.

8. FUTURE TRENDS AND NEW DEVELOPMENTS

In addition to the basic aspects that are inherent to both the test technique and the interpretation thereof, and given the high degree of application that is carried out in linear constructions, it is of primordial importance to work in the future, towards perfecting the use of the technique and its interpretation in specific questions such as:

- Evaluation of the subsoil characteristics in relation to the dimensioning of systems of ground improvement, such as stone columns and drains.
- Assessment of the geomechanical characteristics of the subsoil underlying

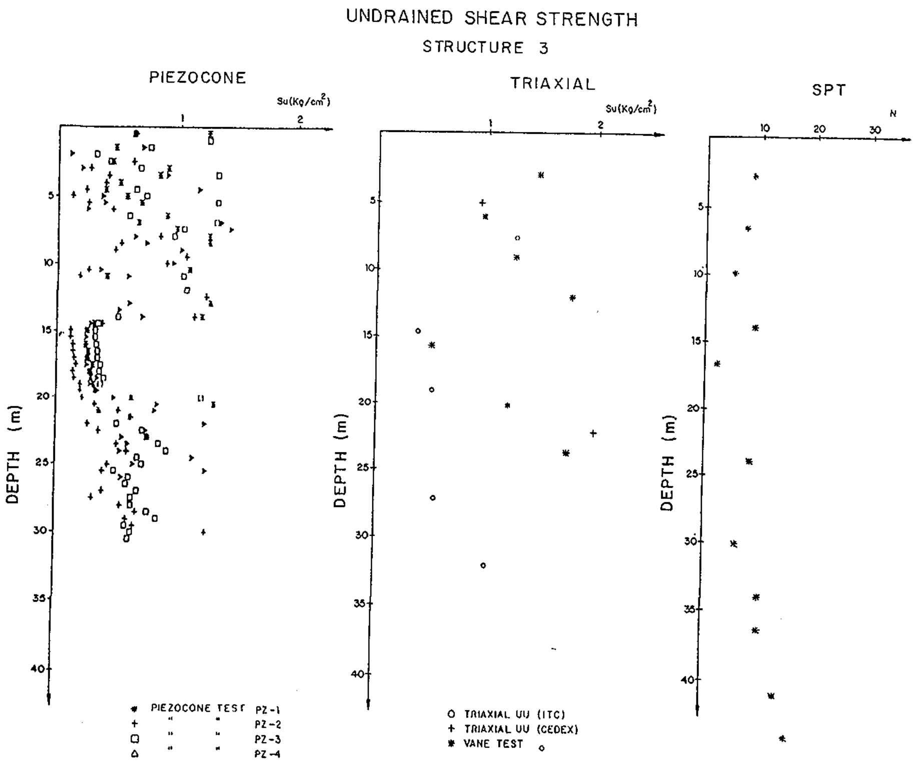

UNDRAINED SHEAR STRENGTH
STRUCTURE 3
PIEZOCONE
TRIAXIAL
SPT
Su(Kg/cm2)
N
DEPTH (m)
PIEZOCONE TEST PZ-1
PZ-2
PZ-3
PZ-4
TRIAXIAL UU (ITC)
TRIAXIAL UU (CEDEX)
VANE TEST

Fig. 2

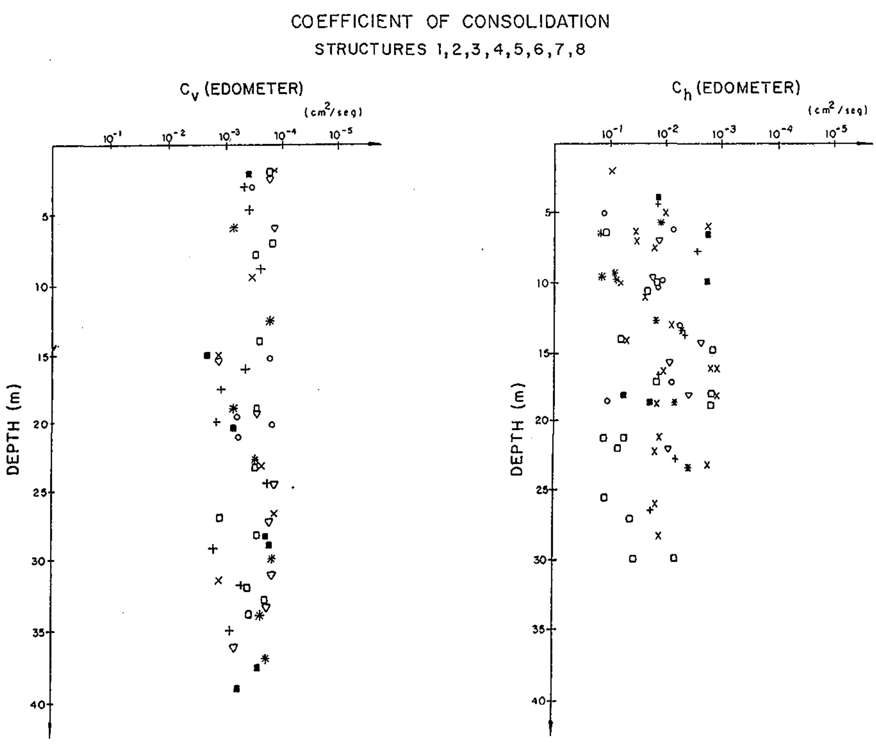
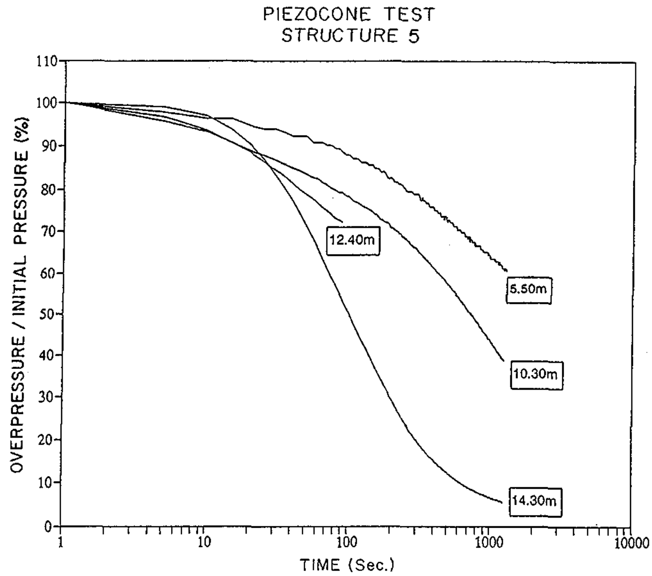

COEFFICIENT OF CONSOLIDATION
STRUCTURES 1,2,3,4,5,6,7,8
Cv (EDOMETER)
Ch (EDOMETER)
(cm2/seg)
DEPTH (m)
X E-1, + E-2, □ E-3, ■ E-4, ✱ E-5, ▽ E-6, o E-7, ● E-8

Fig. 3



COEFFICIENTS OF CONSOLIDATION

DEPTH (m)	C_{hpz} (cm^2/s)	C_h (cm^2/s)
5.50	$2 \cdot 10^{-3}$	$8 \cdot 10^{-4}$
10.30	$1 \cdot 10^{-2}$	$3 \cdot 10^{-3}$
12.40	$2 \cdot 10^{-2}$	$2 \cdot 10^{-2}$
14.30	$1 \cdot 10^{-1}$	$1 \cdot 10^{-1}$

Fig. 4

embankments, including consolidation, and their evolution in time.

Evaluation of the effects of improvements made to the geotechnical characteristics of the subsoil and the behaviour of the works constructed upon it.

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Geotechnical study of Huelva Ring Road, (Spain) Lab. Geotecnia. CEDEX, Madrid 1995.

Cone Penetration Testing - CPT´95 National Report for Sweden

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SYNOPSIS: During the last five years, the CPT test has been widely used as a tool in site investigations. Comprehensive studies have been carried out in Sweden in soft and fine-grained soils, which have resulted in a recommended standard for CPT tests. Swedish manufacturers of CPT equipment have developed cordless CPT systems with high accuracy suitable for soft soils. Software for correction and interpretation of CPT data has been developed and education has been provided for both field and geotechnical engineers. Today, research is in progress into using a slot filter instead of a porous filter and evaluating the properties of silts, sulphide soils and boulder clays. Many of these activities are being performed in large scale field tests.

1. INTRODUCTION

In Sweden, there is a long tradition in geotechnical engineering of using sounding to determine soil stratigraphy and soil properties. During the last 80 years, static sounding as well as dynamic probing have been used and developed for Swedish soil conditions. Combined with sampling, this has provided sufficient information for geotechical design in the typical case.

The first types of CPT tests with measurement of cone resistance were introduced around 1935 using a mechanical measuring principle. Later, developments were made to introduce electronic measurement and record additional parameters. However, the use of the method in geotechnical practice was very limited. Pore pressure sounding was introduced by Torstensson (1975) in Sweden in the mid-70s and was standardised in 1984.

In Sweden, CPT tests and pore pressure soundings have been performed usually as separate tests, but sometimes parallel. Nowadays, the two methods have been integrated into CPT tests where cone resistance, sleeve friction and pore pressure are measured simultaneously.

In recent years, comprehensive investigations have been carried out in Sweden concerning soft and fine-grained soils (Larsson and Mulabdic, 1991). In 1992, a new recommended standard for CPT tests was approved by the Swedish Geotechnical Society. The standard takes into account the Swedish soil conditions, especially soft soils. This standard replaces both the old CPT standard and the standard for pore pressure sounding.

Over the last five years, CPT tests have been widely used as a tool in both routine and more advanced field investigations. During this period, a computer programme has been developed for correction of data from CPT tests, interpretation of stratigraphy and soil parameters and presentation of the results.

2. BRIEF GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF SWEDEN

2.1 Topography

Sweden is a long, narrow country, 1,500 km long and 300 - 400 km wide, covering a total of 450,000 km^2. The topography of Sweden is varied. Almost all the land area has an elevation between 200 and 1,000 m above the sea level. The highest peaks reach about 2,100 m.

2.2 Soil formation

During the last glaciation, the whole of Sweden was covered by the Continental ice sheet . The ice started to retreat from southern Sweden about 14,000 years ago and about 8,500 years ago the inland ice had practically disappeared.

In connection with the retreat of the ice, Sweden was covered by alternating fresh and brackish water. The highest water level - the highest shoreline - occurred at different times in different parts of the country. As the pressure of the inland ice decreased during and after the melting of the ice, the land was uplifted from the water due to the elastic rebound of the depressed crust of the Earth - isostatic uplift - a process that continues today. The absolute maximum uplift is calculated at about 800 metres. The maximum uplift between the highest shoreline and the recent sea level is about 285 m above sea level in the north of Sweden. In the south, the highest shoreline is uplifted about 50 m above sea level.

Together with the shorelines of the former ice lakes, the highest shoreline mentioned above is very important from the geotechnical point of view. Fine grained sediments, especially clay, have a very limited extent above these levels and below them there are only wave washed materials. Thus the soil profiles are principally different above and below these levels.

2.3 Soil and rock composition

The bedrock in Sweden is dominated by crystalline basement, pre-Cambrian hard rock, such as gneisses and granites. There are a few areas with Cambro-Silurian sedimentary rock, such as sandstones, limestone and clay shales. In the mountain area, the Cambro-Silurian rock covers a very large area.

The soils are geologically young and belong with a few exceptions to the Quaternary period. The soils were formed in connection with the movements and melting of the last Continental land ice (glacial soils) and subsequent processes (post-glacial soils).

The **glacial soils** can be divided into tills and glacial sediments. *Till material* - as uppermost layer - covers about 75 % of the land area of Sweden and normally underlies other soils. The composition of the tills is highly varied, ranging from fine grained boulder clay to coarse grained gravel till. The *glacial sediments* consist of coarse grained sediments - sand, gravel and cobbles - in eskers and deltas, and of fine grained sediments - clay and silt - deposited outside the edge of the ice.

The **postglacial soils** can be divided into reworked and redeposited soils and organic soils. Most of the postglacial processes are still in progress, although with rather small magnitude. The glacial soils on slopes exposed to waves were washed during the land uplift. These *wave washed sediments* were transported down the slopes, coarse grained material a short distance and clay particles long distances, and deposited upon glacial soils. *Post-glacial silts and clays* are normally found as a relatively thin top cover on glacial clay. Lower lying glacial material was eroded by large rivers during the uplift process. Significant volumes of *fluvially eroded material,* mainly silt and sand, were thus transported downstream to form new (post-glacial) sediments covering glacial soils. Of special geotechnical interest is that wave washed and eroded gravels and sands in many places cover clay and silt in the lower parts of slopes and valley floors. The distribution of coarse and fine-grained sediments is shown in Figures 1a and 1b (it should be observed that the bedrock in the mountain range in the north-western part of Sweden is also shown on the maps).

After the glacial period, there was a strong increase in the production of ***organic material.*** Organic material was mixed into the fine grained material to form more or less organic fine grained soils, e.g. organic clay (gyttja clay) and gyttja. Peat bogs and fens were formed in many places in Sweden.

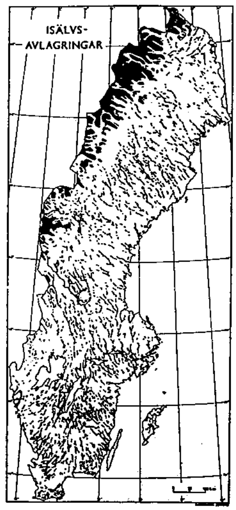

Figure 1a. Distribution of sand and gravel (glacio-fluvial sediment).
(From Magnusson & al, 1963)

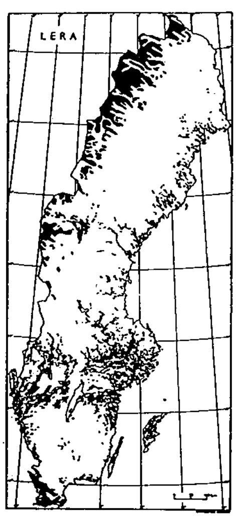

Figure 1b. Distribution of clay, silt and boulder clay
(From Magnusson & al, 1963)

2.4 Thickness of soil cover

Rock outcrops are common in Sweden and the thickness of the soil cover is therefore in general relatively modest, normally a few metres to some tens of metres. The thickness may change rapidly within a short distance. The greatest known soil depth is about 200 m.

The till is in general a few metres thick, but may in some places reach depths of several tens of metres.The thickness of sand and gravel in eskers and large deltas is often a few tens of meters and not infrequently up to 50 m or more. The thickness of clay deposits is in general 5-10 m, but thick layers of clay, up to 100 m, are encountered on the west coast (Gothenburg area) and north of Stockholm (city of Uppsala). Sedimentary layers consisting of sand and silt and some clay with thicknesses up to 50 m or more occur in river valleys in Värmland, Dalarna and in river valleys along the eastern part of northern Sweden.
A typical soil profile in the middle of southern Sweden is shown in Figure 2. The profile illustrates the soil layering principles below the highest shoreline.

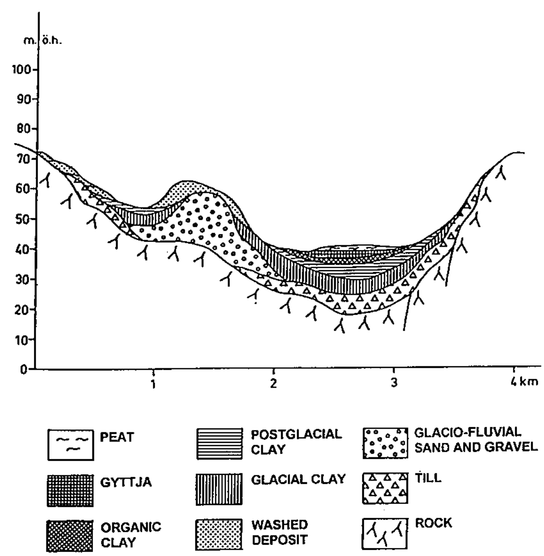

Figure 2. Schematic profile from the middle of southern Sweden below the highest shoreline.
(From Ahlberg, 1979)

3. TRADITIONAL SITE INVESTIGATION TECHNIQUES

The investigation of soil strata by penetration testing has a long tradition in Sweden. As far back as 1914-1922, the Geotechnical Commission of the Swedish State Railways developed and standardised the **Swedish Weight Sounding Method**. (Note.: In the following, the term "sounding" is used in the same sense as "penetration testing"). With the aid of this equipment, in combination with sampling, it became possible to obtain reliable information about the stratification of soil deposits. Over the years, the weight sounding method has become the dominant soil investigation tool in Sweden. Even today, this method is the most commonly used soil investigation technique in Sweden. During the last 20-30 years, the testing equipment has been mechanised. The screw-shaped penetration point, however, has maintained its initial shape as designed by the Geotechnical Commission of the Swedish State Railways. The standard for the Swedish Weight Sounding Method was approved by the Swedish Geotechnical Society in 1974.

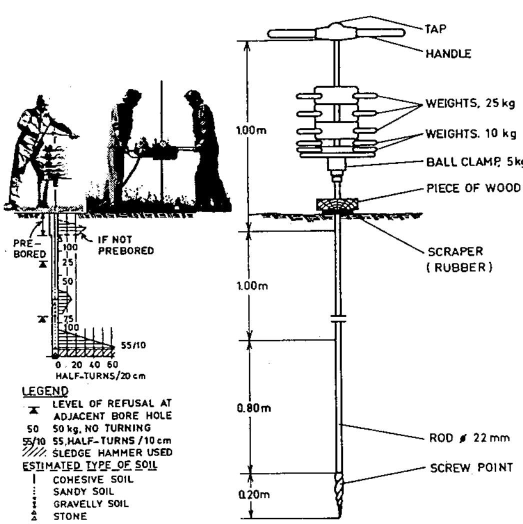

Figure 3. The Swedish Weight Sounding Method. Equipment, principle and presentation of test data.
(From Dahlberg, 1974)

Figure 3 shows the original testing equipment and the presentation of penetration data. The method is predominantly used in clay, peat and other organic soils and in loose to medium dense silt and sand.

Today, weight soundings are almost exclusively carried out with the aid of mechanical and/or hydraulic drill rigs.

It should be emphasised that the accuracy of the weight sounding method, as well as other penetration methods, is largely dependent on the experience and care of the operator. This is especially true when mechanical penetration equipment is being used.

Parallel with the use of the weight sounding method, a number of mechanical, static penetrometers have been developed in Sweden. The *Geotech* (formerly *Nilcon*) *Static Penetrometer* is the most commonly used equipment of this kind. This equipment utilises a pyramidal tip with an area of 10 cm² which is pushed down into the soil at a constant speed. By using a special slip coupling, the friction along the push rods can be separated from the total penetration force, Figure 4.

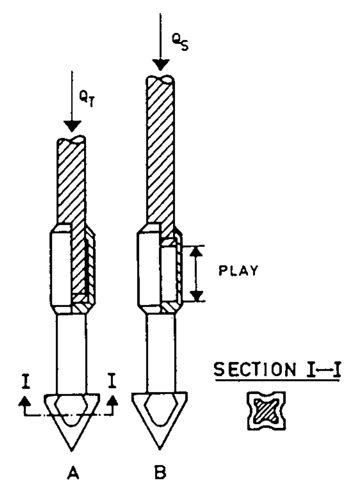

Figure 4. Tip of Geotech Static Penetrometer.
(From Dahlberg, 1974)

With the aid of a mechanical recorder, the total penetration force is automatically registered as a function of the depth. The Geotech Static Penetrometer provides detailed information on the thickness and relative strength of the penetrated soil layers.

Different dynamic probing methods were used very early in Sweden. Around 1940, the Swedish firm Borros designed and developed the **Swedish Ram Sounding Method**. This method has been commonly used for predicting of the length of end-bearing, driven piles.

Based on extensive investigations by Bergdahl and Dahlberg (1974) the traditional ram sounding method was revised and an improved method and procedure - Method A - was standardised in 1973. The basic elements of the Swedish ram sounding test are described in Figure 5. A free-falling hammer (weight 63.5 kg) strikes a fixed anvil, equipped with a rubber cushion. The height of the fall is 0.5 m In order to reduce the skin friction, the extension rods (32 mm) are rotated two turns every 0.2 m of penetration.

A more sophisticated testing procedure includes measurement of the torque required to turn the extension rods. Based on such measurements, the skin friction can be separated from the total penetration resistance, which in turn enables a more detailed interpretation of the test results.

The Swedish Ram Sounding Method is useful tool for predicting of the length of both end-bearing and friction piles driven in sand and gravel.

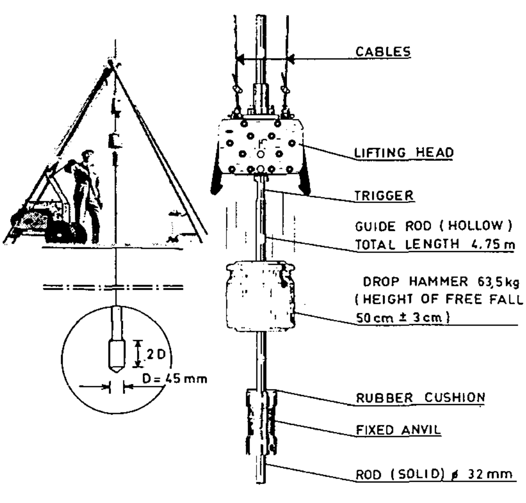

Figure 5. Swedish Ram Sounding Device, Method A.
(From Dahlberg, 1974)

Since around 1950, heavy rock drilling machines have been utilised for **Soil and Rock Drilling**. The main purpose of the Soil and Rock Sounding method is to determine both the depth to bedrock and the quality of the underlying rock. In principle, this type of investigation is carried out with the same equipment as that used for conventional rockdrilling. Light, medium or heavy rock drilling machines are used, depending on both the soil/rock conditions and the required maximum depth of investigation.
The time in seconds required for 20 cm of penetration is recorded. This parameter gives an indication of the hardness of the rock or the compactness/composition of the penetrated soil. In accordance with the existing recommendations, the soundings shall extend 3 to 5 m into assumed rock in order to ensure that the borehole does not terminate in a boulder.

The Swedish Geotechnical Society has proposed a standard for the Soil and Rock Sounding method, which includes items such as the outside/inside diameter of the drill rod and casing, size and shape of drill bits, rotation rate, torque and axial force applied to the drill rods, pressure and flow rate of the circulation fluid.

Today the use of in situ methods, such as pressometer and dilatometer are steadily increasing in Sweden.

Parallel to the testing described above, sampling is also carried out by routine in order to obtain additional information about soil types and characteristic soil parameters.

4. TYPE OF CPT EQUIPMENT USED IN SWEDEN

The Static Penetrometer described in the preceding section may be called a mechanical CPT. Improvement of electronic and computer technologies has facilitated the development of various CPT systems. In Sweden, normal and fairly small drill rigs are used for all types of field investigation. This equipments is relatively compact and can easily reach difficult areas. This has forced the Swedish manufacturers to produce more flexible CPT systems, which can easily be installed on any type of drill rig with capacity to push the probe. Today, there are two manufacturers of CPT equipment in Sweden: ENVI AB and GEOTECH AB. Both produce cordless CPT probes, but using different measuring and recording principles. Almost all

CPT tests in Sweden are performed with one of these systems. Other equipments, such as Hogentogler, Van den Berg or GMF, has occasionally been used, mainly for comparison in research projects or measurement of additional parameters.

4.1 Equipment

The **GEOTECH CPT** system is cordless. Instead of a cable, a sound transmitter on top of the CPT converts analogue signals (three or four channels) into a digital coded signal. The signal is transmitted through the rods up to the drill rig or pusher at ground level. A microphone receiving the signal is installed on the feeder of the drill rig. The signal then goes by cable to an interface connected to a PC or some other logger. The system provides full information in real time during the test. Since the hole in the centre of the rods is not used for a cable, it is possible to use it for pumping down Bentonite mud and discharging it just above the probe.This reduces the friction along the rods dramatically, and consequently the total force needed to push the probe can be reduced by more than 50%. GEOTECH also manufactures multipurpose mobile drill rigs, from which the CPT is operated.

ENVI MEMOCONE is a CPT system which works with or without a cable. During the test, all data is stored in the internal memory of the probe. The readings are stored once a second for a maximum period of 4.5 hours. On the drill rig, time and depth are recorded. The probe is pushed until refusal, retracted, and the data is transfered into a PC interface or a GEO-PRINTER. The probe can be stopped for a period in order to monitor porepressure dissipation.This operating principle makes it possible to use CPT at great depths, and the results cannot be affected by disturbance in data transfer.The cone can be operated using the same drill rods as are used for other purposes on the drill rig.

BORRO is the manufacturer of a continuous 20 tonne CPT pusher. The rig contains two chucks operating in sequence in such a way that the penetration is uninterrupted. The electronic control panel provides information on rate of penetration, depth, total load in kN and the setting for maximum load cut-off limit. The rig has also been used underwater.

4.2 National Codes and Standards

On June 15 1992, the Swedish Geotechnical Society presented a new recommended standard for CPT testing. This standard replaced a standard from 1979. The earlier standard did not include pore pressure measurement and was mainly intended for mechanical CPT testing.

The probe consists of a cylindrical cone with a cross sectional area of 1,000 mm² and an apex angle of 60°. This is pushed vertically into the ground at a constant speed of 20 mm/sec.

The Swedish recommended standard has three different classes depending on factors such as the precision requirements CPT1, CPT2 and CPT3, where class 3 is the highest requirements. The ranges for application of the various test classes are shown in Table 1.

In general, the accuracy, taking into account all possible sources of error (parasitic frictions, errors of measuring devices, eccentricity of the loads on the cone and the sleeve, temperature effects, etc.), shall be better than

- 2 % of the typical measured value (the average value) for any of the soil layers* in which the results are to be interpreted in terms of classification and soil properties
- 1 % of the measured values for static pore pressures

*) *The term "soil layer" in this context refers to separate soil layers or, in case of thick homogeneous layers, depth intervals of one metre.*

For the different test classes, there are lower limits for the required precision in terms of generally accepted inaccuracies according to Table 1. The precision of a cone must be verified by regular calibration. Calibration procedures have been worked out by Mulabdic et al (1990)

4.3 Temperature stability

All measuring elements and all other electronic devices shall be stable to temperature changes. When handling the equipment, care shall be taken to minimise temperature changes in the cone.

Table 1. Range of application for different test classes

Class	Type of soil						Accepted inaccuracies		
	Coarsegrained		Silt		Fine-grained		Cone resistance kPa	Sleeve friction (kPa)	Pore pressure kPa
	Stratigraphy	Properties	Stratigraphy	Properties	Stratigraphy	Properties			
CPT1 A	•[1]	•[1]	°	°	-	-	100	10	-
CPT1 B	•	•	▲	▲	°[2]	°[2]	100	10	10
CPT2	(•)[1]	(•)[1]	•	•	▲	▲	40	4	5
CPT3	((•))[4]	((•))[4]	(•)[5]	(•)[5]	•	•	20	2	1

A) Without pore pressure measurement
B) With pore pressure measurement

Estimation:
- • Good
- ▲ Relatively good
- ° Coarse
- \- Not possible

Remarks:
1) The ground water level has to be determined separately. At large water depths the tests should be performed with measurement of pore pressure.
2) Only in stiff fine-grained soil.
3) Somewhat limited application in coarse-grained soil.
4) Very limited application in coarse-grained soil.
5) Limited application in stiff silt.

The required stability expressed as maximum zero shifts is

- 2.0 kPa/°C for cone resistance
- 0.1 kPa/°C for side friction
- 0.05 - 0.1 kPa/°C for pore pressure (transducers with measuring ranges of 10 -20 bars)

This demand on stability, which applies to 5-tonne cones, shall be verified. For cones with higher measuring ranges, a proportional instability is accepted.

4.4 Data format standard

The need for a modern method of interchanging geotechnical information has stimulated the development of a data format standard for geotechnical measurements. This also applies to CPT, and since 1990 there exists a standard recommended by the Swedish Geotechnical Society. Since the 1994 revision, the standard covers the interchange between field systems and office as well as between offices. All geotechnical software used in Sweden today conforms this file standard.

5. INTERPRETATION OF TEST RESULTS

The most commonly used method of interpretation was worked out by Larsson (1993). Generally, the interpretation of tests results in coarse materials is based on international experience, after some minor modifications. For tests performed in soft fine grained soils, the interpretation relies on semi-empirical data obtained from research carried out in Sweden.

5.1 Stratigraphy

To obtain information on soil stratigraphy, the following parameters, measured or derived, are evaluated:

- Total cone resistance, q_T
- Total sleeve friction, f_T
- Total pore pressure, u

The basic parameters q_T and f_T are obtained after correction of the measured values for pore pressure effects.

For interpretation of the results, the following basic parameters are also required:

- Initial in situ pore pressure, u_0
- Initial vertical stress in situ, σ_{v0} (calculated from the density of the soil)

The initial pore pressure may be obtained from supplementary pore pressure measurements made at a number of levels, or by observations of the ground water level in more permeable layers measured during temporary stops in the penetration test.

The initial vertical stress in situ is estimated by using the density of the soil. This estimate can often be by interaction using the classification of soil type and stiffness obtained from the test results to estimate an approximate density.

Different relations between the basic parameters are used for interpretation of the test results. For a preliminary interpretation, the following parameters are often used:

- $\Delta u = u - u_0$
- Friction ratio, $R_f = (f_T/q_T) \cdot 100$, %
- Differential pore pressure ratio,

$$DPPR = \Delta u/q_T$$

All the parameters mentioned above (measured, corrected and calculated) are usually plotted against the penetration depth. From this plot, there are excellent possibilities for evaluating the stratigraphy.

5.2 Soil classification

The properties of the soil affect the magnitude of the cone resistance, the sleeve friction and the generated pore pressure measured during CPT tests in different ways.

A ***preliminary classification*** of the soil can be made using the relations $(q_T - \sigma_{v0})/\sigma'_{v0}$ and $f_T/(q_T - \sigma_{v0})$. The soil density used for calculating the parameters included in these relations may be obtained from sampling, dilatometer tests or by an iteration process in the interpretation of the CPT test. The soil is usually divided into layers of a given thickness in order to perform the calculation. The principal character of the soil (***sand, silt or clay/organic soil***) is thereafter estimated by using the diagram presented in Figure 6.

The preliminary soil classification based on CPT tests is thus mainly made on the basis of the relation between the cone resistance, the sleeve friction and the normal in situ stress conditions. In soft clay, the measured sleeve friction is very small and relatively unreliable but in overconsolidated clay, where the cone

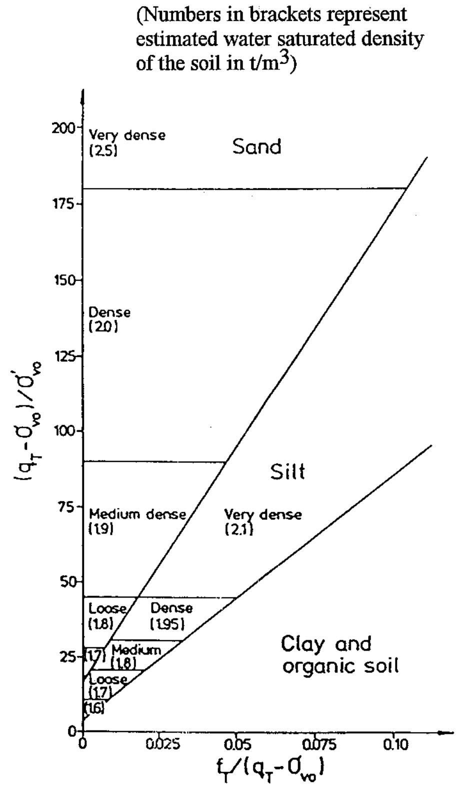

Figure 6. Diagram for evaluation of silt and sand and separation of cohesive soil.

resistance may be of the same size as for soft, coarser soil, the measured values normally become larger and more reliable. Possible uncertainties in the measurements of sleeve friction normally have a relatively small influence on this division. The main exceptions are highly sensitive clays and/or silty clays. In these soils, the sleeve friction may be very low, at the same time as the measured stiffness in relation to the overburden pressure places the soil in the region for silt according to the diagram in Figure 6. However, in these soils very high pore pressures are often developed in the tests and a check on whether the factor B_q $[=\Delta u/(q_T - \sigma_{v0})]$ is higher or lower than 0.6 can be used to judge whether the soil should be classified as silt or clay.

In those cases where the soil has been classified as "***clay/ organic soil***" in the first diagram or after further checks, or if this has been specified

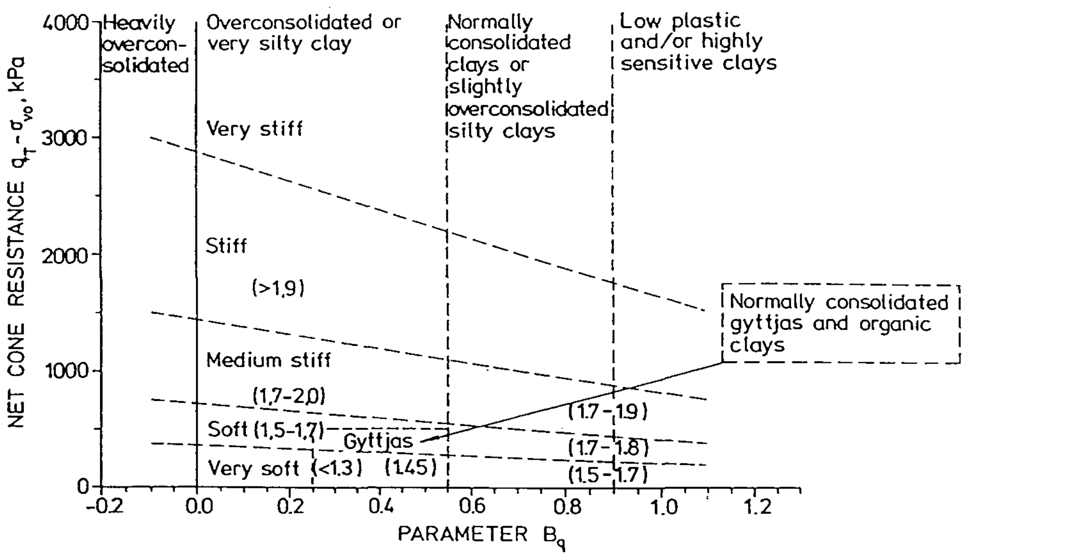

Figure 7. Classification chart for clay and organic soil.

at an earlier stage, the classification process passes over to the special classification chart developed for this type of soil, Figure 7. This chart is based on the parameters net cone resistance (q_T - σ_{v0}) and $B_q = \Delta u/(q_T - \sigma_{v0})$.

The soil in the groups is also summarily classified with respect to the undrained shear strength as very soft, soft, medium stiff, stiff or very stiff. This division is preliminary because the relation between the net cone resistance and the shear strength among other things depends on the consistency limits of the soil. After a later, more careful evaluation of the undrained shear strength, the sub-designations with respect to the undrained shear strength should be adjusted according to Table 2.

Table 2. Classification of clay

Undrained shear strength, kPa	Designation
<12.5	Very soft
12.5 - 25	Soft
25 - 50	Medium stiff
50 - 100	Stiff
> 100	Very stiff

5.3 Soil parameters and other data

Correlation between results from the CPT tests and in situ test or qualified laboratory tests have provided relations for evaluation of ***undrained shear strength*** in fine-grained soils. The relation between the undrained shear strength and the net cone resistance is sensitive to the liquid limit of the soil, w_L

$$\tau_{fu} = \frac{q_T - \sigma_{v0}}{13.4 + 6.65 w_L}$$

The undrained shear strength evaluated in this way corresponds directly to the undrained shear strength obtained from corrected field vane and fall cone tests, direct simple shear tests and dilatometer tests.

If values of the liquid limit of the soil are missing, a coarser estimate of the undrained shear strength can be made according to

$$\tau_{fu} = \frac{q_T - \sigma_{vo}}{16.3} \quad \text{or} \quad \tau_{fu} = \frac{q_T - \sigma_{vo}}{24}$$

(for clay) (for gyttja)

The accuracy of the evaluation of the undrained shear strength depends mainly on the accuracy achieved in the test. When very accurate estimates are required, the results from the CPT tests should be calibrated locally against field vane tests and preferably also direct simple shear tests.

A preliminary estimate of the ***preconsolidation pressure*** in cohesive soils can be made from the net cone resistance:

$$\sigma'_c \approx (\frac{q_T - \sigma_{v0}}{1.21 + 4.4 w_L}) / (1.07 - 0.54 \log OCR)$$

The relation is sensitive to both the plasticity and the overconsolidation ratio of the soil. The equation is solved by iteration involing the insertion of the estimated in situ effective vertical stress σ'_{v0} and OCR = σ'_c / σ'_{v0}.

The ***relative density*** of relatively even-graded sand with a mineral composition of quartz and feldspar may be obtained from the relation between the cone resistance and the effective over burden pressure (Lancelotta 1983). For coarser soils, the relative density can be calculated using the same relation and applying a reduction of 10 - 15 % according to Lunne and Christoffersen (1983).

Also the ***friction angle*** in friction material can be evaluated from the relation between the cone resistance and the in situ effective vertical stress. In reality, it is mainly the horizontal stress that governs the relation and, before the evaluation is made, some kind of estimate of the coefficient of earth pressure should be made (Marchetti 1985).

The evaluation of ***deformation properties*** is normally not made directly from the CPT test results. In ***friction soils***, some of the well established empirical calculation methods specially produced for the CPT test are used. However, these methods are only suited for calculation of settlements and bearing capacity in normal cases of shallow foundations on sand. For other cases of loading on sand, compression moduli and moduli of elasticity may be estimated according to Robertson and Campanella 1988).

Compression characteristics in ***cohesive soil*** should not be evaluated directly from the results of CPT tests, which in this type of soil are performed under almost fully undrained conditions. For evaluation of these properties, undisturbed sampling and oedometer tests are required.

5.4 Evaluating software

The use of CPT in Sweden has increased after the Geotechnical Institute developed the CONRAD programme for evaluating of CPT data, (Larsson et al, 1995). The software is interactive and permits use of information from other investigation methods and sampling. The software evaluates different soil parameters according to the methods mentioned above.

6. USE OF CPT IN GEOTECHNICAL DESIGN

6.1 Design concept

Design of shallow and deep foundations in Sweden is carried out using partial safety factors. An analysis shall be performed for the working load (concerning mainly deformations) as well as for the ultimate load (failure conditions).

Calculations are made using the design values obtained from characteristic values via the method of partial safety factors and according to the following relations:

$f_d = f_k / (\eta\, \gamma_m\, \gamma_n)$ or $f_d = f_k\ (\eta\, \gamma_m\, \gamma_n)$

where

- f_d the design value of the material property, e.g. friction angle
- f_k the characteristic value of the material property, e.g. friction angle
- η difference between the properties of a sample and those of the actual structure, which for soils is generally not relevant, i.e. $\eta = 1.0$
- γ_m relates to the uncertainty in the determination of the material property (range of values - different for ultimate and working load)
- γ_n relates to the safety class of the structure (three classes)

6.2 Determination of geotechnical parameters from cone penetration test

Characteristic values of geotechnical parameters shall be based on average (medium) values. In some instances, however, it is permitted to correct these values with respect to the effects of the investigation method, variations in time etc. In friction soils, these characteristic values are usually determined from penetration tests or other in situ tests. Table 4 shows the correlation between different sounding methods and typical geotechnical parameters. It is also more common today to evaluate parameters from the relations described in section 5.3.

6.3 Bearing capacity and settlements of shallow foundations

The loadbearing capacity for shallow foundations is frequently determined according to the method proposed by Meyerhof (1956) from

$q_b = 3\ q_c\ (b\ (1+ d/b)) / 40 \qquad \text{(MPa)}$

where

- q_b ultimate load, MPa
- q_c average cone resistance to depth b below slab, MPa
- b width of slab, m
- d minimum foundation depth, m

The calculated ultimate load is reduced by 50 % if the ground water level is located at or above the foundation level.

The characteristic value of the friction angle can be determined directly according to the relation proposed by Lunne and Christofferssen (1983).

Settlements are usually determined following the method proposed by de Beer (1965). The settlement modulus C is determined from cone penetration tests according to the relation

$C = 1.5\ (q_c / \sigma_o')$

where σ_o' is the effective overburden pressure. The settlement modulus E_k in friction soils can be estimated from the following relation:

$E_k = 4\ q_c \qquad (q_c \leq 10\ \text{MPa})$

$E_k = 2\ q_c + 20\ \text{MPa} \qquad (10\ \text{MPa} \leq q_c \leq 50\ \text{MPa})$

Alternatively, the method proposed by Schmertmann (1970) and Schmertmann et al. (1978) is used for determination of settlements.

6.4 Bearing capacity of pile foundations

The design bearing capacity R_d of friction piles in sand is composed of the contribution from shaft resistance R_{md} and base resistance R_{sd}.

$R_d = (R_{md} + R_{sd}) / \gamma_{Rd}$

where

- γ_{Rd} Partial safety factor (generally equal to 1.6)

The shaft and base resistance can be estimated from cone penetration tests according to the following relations:

$R_{md} = \Sigma\ \alpha_m\ A_{mi}\ (q_{ci} / \gamma_n\ \gamma_{mm})$ and

$R_{sd} = \alpha_s\ A_s\ q_{qs} / (\gamma_n\ \gamma_{ms})$

where

- α_m Pile shaft capacity factor, c.f. Table 3
- q_{ci} Cone penetration resistance, average value within the layer
- A_{mi} Pile surface area within the layer
- α_s Pile base capacity factor, c.f. Table 3
- q_{qs} Cone penetration resistance at pile base, as average value in the depth interval of ± 4 pile diameters measured from the pile base
- A_s Pile area at the base

Table 3. Pile shaft and base capacity factors a_s and a_m for driven piles, based on cone penetration tests in friction soils

Pile type	α_s	α_m
Concrete pile	0.5	0.0050
Steel pile	0.5	0.0025
Timber pile (conical, root upwards)	0.5	0.0090

Cone resistance values higher than 10 MPa may not be used with this design method. Also the method proposed by de Ruiter and Beringen (1979) can be used for designing pile capacity. The base resistance is determined as an average value according to the relation below:

$f_s = (I+II)/2 + (III)/2$

where

- I average cone penetration resistance between the level of the pile base and a distance of 0.7 and 4 pile diameters below

II minimum cone penetration resistance between the level of the pile base and a distance of 0.7 and 4 pile diameters below

III average cone penetration resistance between the level of the pile base and 8 pile diameters above

This method gives similar results, but the maximum design values are limited to 120 kPa for shaft resistance and 15 MPa for base resistance, respectively. When sleeve friction measurements are carried out during the cone penetration test, these values can be used directly for assessment of pile shaft resistance. De Ruiter and Beringen have also proposed limiting values for pile base resistance, which are dependent on soil type.

Generally, the cone penetration test is not used for assessment of pile bearing capacity in cohesive soils.

6.5 Slope stability

Slope stability problems are of great importance in Sweden. Cone penetration tests are frequently used to establish the general geotechnical conditions in critical sections. CPT are considered to be the most reliable method at present for detecting layers of silt and sand in soft clays, which may have a great influence on the stability of a slope, especially in the case of high pore water pressure.

Correlations between results from CPT tests and undrained shear strength, see section 5.3 has been improved. These relations are used together with field tests (vane test) or laboratory tests (fall cone test) to obtain a profile of shear strength against depth. The trend in Sweden is towards increasing usage of CPT tests in slope stability investigations.

7. COMPARISON AND CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

Using results from a comparative investigation at 14 test sites, a correlation between different site investigation and laboratory methods has been obtained by Bergdahl and Ottosson (1988). The results from this study are summarised for common Swedish sounding methods in Table 4. Both the weight sounding test and dynamic probing type HfA are standardised in Sweden. Further elaboration of the table has been carried out for various purposes, but the contents are almost the same.

In fine silt, the above values should be verified by laboratory tests or other in situ tests (pressure meter or dilatometer). In silty and clayey soils or sedimented soils with some organic content, penetration tests shall be complemented by sampling and compression tests or other in situ tests.

A relation between dynamic probing and SPT has also been established. The blow count for 0.2 m penetration for dynamic probing type HfA is equal to the blow count for 0.3 m penetration for SPT performed in accordance with the European standard.

Table 4. Characteristic values of friction angle and E-modulus for naturally deposited friction soil based on penetration tests.

Relative Stiffness	Cone Penetration Resistance q_{ck}, MPa	Friction Angle $\phi_k{}^1$, degrees	E-modulus $E_k{}^2$, MPa	Swedish Weight Sounding Vimk ht/0.2 m	Heavy Ram Sounding HfAk bl/0.2 m
Very loose	0 - 2,5	29 - 32	< 10	0 - 10	0 - 4
Loose	2.5 - 5.0	32 - 35	10 - 20	10 - 30	2 - 8
Medium	5.0 - 10	35 - 37	20 - 30	20 - 50	6 - 14
High	10 - 20	37 - 40	30 - 60	40 - 90	10 - 30
Very high	> 20	40 - 42	60 - 90	> 80	> 25

1 *For silt, reduce the value by 3°; for gravel, increase the value by 2°.*

2 *Values correspond to settlements after 10 years. Some investigations suggest that these values may be 50 % lower in silt and 50 % higher in gravel. In overconsolidated soils, the modulus values may be significantly higher. In settlement estimates at loads exceeding 2/3 of the ultimate failure load, the modulus should be reduced by 50 % for higher loads.*

8. MAJOR AREAS OF RESEARCH

The most recent research activities have dealt with the possibility of rationalising the test by using a slot filter instead of conventional porous filters, Larsson (1995) and Elmgren (1995). This development is continuing with the use of new combinations of pressure transmitting fluids and tests in new types of soil.

The main activities today are otherwise related to utilising the method and gathering experience in full scale projects in which the soil behaviour in full scale construction is also recorded. The methods of evaluating of properties are thereby controlled and will possibly be improved. This is also achieved by correlation to other test methods and full scale field tests in soils, such as silts, fine-grained till and sulphide soils (svartmocka), where experience of the method so far is relatively limited.

A special research programme is related to the possibility of evaluating the effects of deep compaction by the CPT test incorporating other measured parameters than cone resistance.

9. FUTURE TRENDS AND DEVELOPMENT

The future trends for research on CPT tests are of two types. For conventional geotechnical purposes, current activities will continue. There is continuous improvement of the equipment and the interpretation methods, and also the applications are becoming wider, although no radical changes are expected. In the environmental field, a number of new probes and measuring techniques are being developed. This is mainly taking place abroad, but some of these probes will be tested also in Sweden.

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Cone Penetration Testing in Switzerland

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SYNOPSIS: Due to the varying geology, compared to other regions of Europe, in Switzerland it is important to distinguish between soil deposits in which cone penetration tests can be carried out successfully and those in which cone penetration is impossible. In soft post-glacial deposits as well as in silts and sands the piezocone test provides valuable data related to soil profiling, relative density of cohesionless soils, hydraulic characteristics and steady state groundwater conditions. The piezocone test is widely used in conjunction with other investigation methods. This offers the possibility for a quantitative evaluation of soil parameters, like shear strength or soil stiffness and provides a more comprehensive and fundamental knowledge of the investigated soil.

1. GEOLOGY

The use of the piezocone test in Switzerland should always take into consideration the very rapidly changing geological and geotechnical conditions in the subsoil. The actual sequence of soil formations results mainly from the most recent Ice Ages and consequently is strongly heterogeneous. Compared to other European regions it can not be assumed *a priori* that cone penetration tests can always be carried out at any given location.

From north to south Switzerland can be divided into three geological zones:

- the Jura (pre-alpine sedimentary rocks, formed of limestone, sandstone, marlstone and claystone),
- the "Mittelland" (Molasse - sedimentary rocks, originally consisting of sediments that were alluviated during the formation of the Alps) and
- the Alps.

Molasse can also be found in a certain region in southern Switzerland (Ticino).

In the Mittelland the encountered moraines mostly consist of overconsolidated and compacted sandy and clayey silts with varying amounts of gravel and boulder. In such ground conditions cone penetration tests are usually not possible or are extremely difficult to carry out. In some other parts of the Mittelland the Molasse reaches the surface and often a thin weathered layer is observed.

In river areas, as well as near existing and former alpine lakes, the soil conditions are strongly heterogeneous:

On the one hand, there are fluvio-glacial deposits of boulder and gravel where cone penetration is impossible.

On the other hand, there are normally consolidated soft deposits consisting of alternating layers of peat, clays, silts, sands and gravel.

A third type of deposit which is often encountered consists of normally consolidated soft post-glacial lacustrine clays, silts and fine sands, which may exhibit a very high sensitivity (Fig. 1).

It is in such difficult soil conditions that cone penetration tests represent an important subsoil investigation tool for the engineer.

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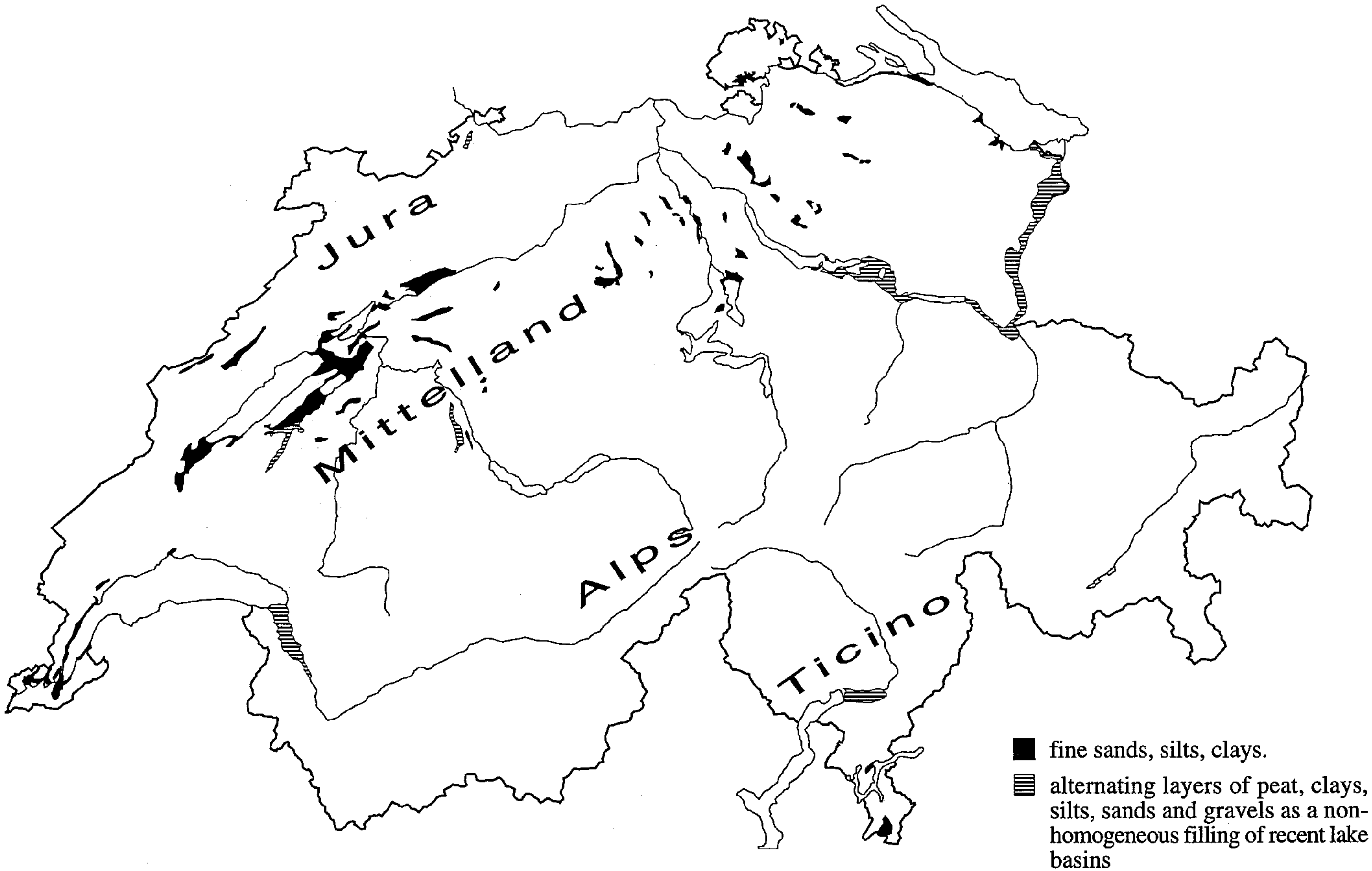

Fig. 1: Occurrence of fine grained and soft clayey and peaty deposits in Switzerland (source: hydrogeological map of Switzerland)

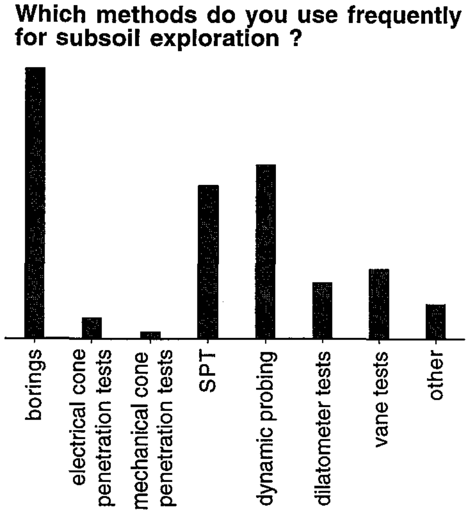

Fig. 2: Current practice in subsoil exploration

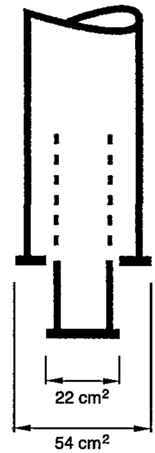

Fig. 3: Cross-section of static/dynamic penetrometer

2. SUBSOIL EXPLORATION

In order to ascertain the current practice of cone penetration testing in Switzland, a questionnaire was sent out to surveyors from public authorities, engineering offices and companies, including some general questions about the use of other investigation methods.

As a result of this survey it was revealed that the most commonly adopted procedure is to carry out borings in conjunction with standard penetration tests (SPT) or vane tests (Fig. 2). Penetration tests used, other than the SPT, are dynamic probing and electric or mechanical cone penetration tests.

Another penetrometer currently available in Switzerland can either be pushed or driven into the subsoil, the so-called Andina-Penetrometer (Fig. 3, cf. Sanglerat 1974). When pushed statically the resistance acting on the whole 54 cm^2 tip is registered by load cells situated at the top of the rods. A reading for the overall skin friction is also available. If the soil stiffness increases, the mode of operation is changed to an alternating pushing of the inner rods and the outer tubing and the pressure on the 22 cm^2 end plate is measured. When reaching the static capacity, the penetrometer is driven, reaching a maximum driving energy of 1.4 MNm for the 22 cm^2 end plate. The number of blows may then be correlated with the static resistance by means of a driving formula.

Summarizing the practice being followed in Switzerland it is seen that, despite the poor and sometimes misleading results obtained, dynamic probing is the most widely used field test. As revealed by the survey, cone penetration tests are employed by some of the engineering offices, most of them reporting satisfaction with regard to the test results. The types of soil investigated were stated to be

- soft silty and clayey deposits in the region of present and former alpine lakes,
- silts and sands,
- deposits formed by silting-up.

In these cases the cone penetration tests are carried out with a certain knowledge of the geotechnical conditions or subsequent to a preceding boring.

3. EQUIPMENT AND PERFORMANCE OF CONE PENETRATION TESTS

According to the results of the above-mentioned survey, electric cone penetration tests in Switzerland are carried out using 10 cm² cones. The pore pressure is almost always measured (CPTU). The fluid used for saturation of the piezocone is either water or silicone oil. In all cases the porous filter element is located immediately behind the cone tip (position 1 in Fig. 7). A correction for the cone resistance is carried out by most of the engineers using the formula

$$q_T = q_c + (1 - a)u$$

where a is an equipment-dependent value (Campanella et al. 1982), u and q_c are the measured pore pressure and cone resistance respectively, and q_T is the total pressure acting on the cone tip.

About half of the cone penetration tests are stopped one or more times in order to monitor the decay of the pore pressure with time. If the penetration has been stopped within a highly permeable layer, a value for the *in situ* pore pressure is obtained almost immediately. When a layer with low permeability is encountered dissipation of the pore pressure is observed. In any case plotting the readings of the pore pressure on a logarithmic time axis may provide some help to the test operators when fixing the duration of the measurement.

The representation of the results usually comprises plots of the cone resistance, the sleeve friction and the pore pressure.

Based on the results of the survey, it appears that cone penetration tests including measurements of environmental data have not yet been carried out.

In Switzerland there are no national codes or standards for cone penetration tests. However, it can be presumed that these tests are carried out according to international standards, i.e. ISSMFE (1988).

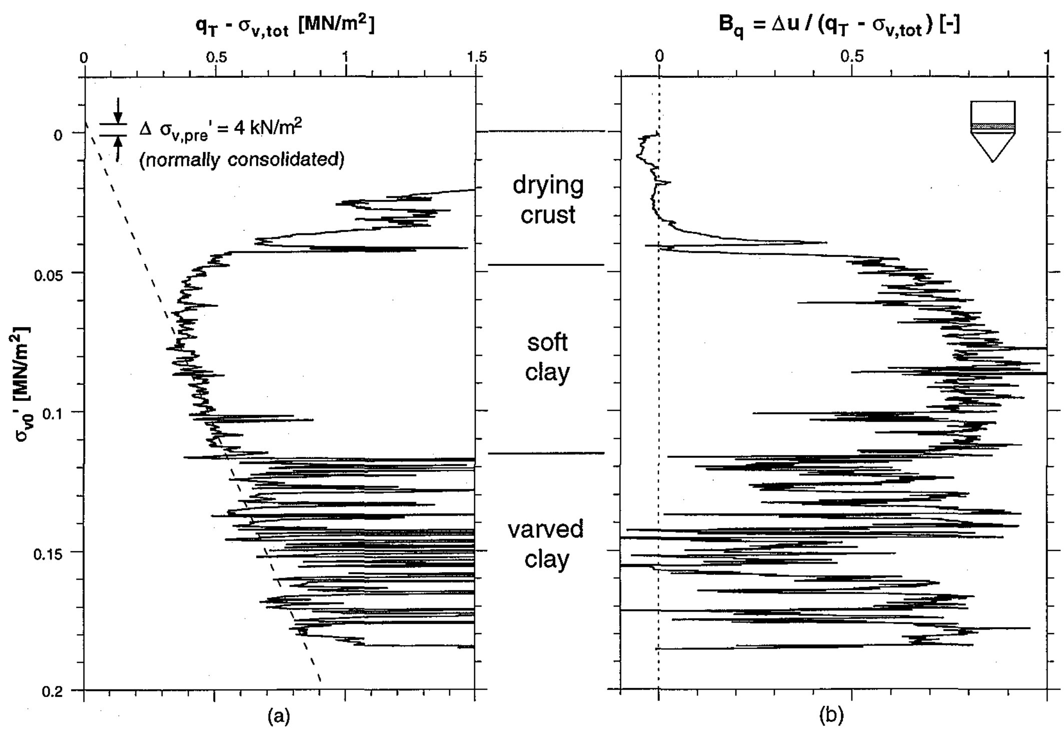

Fig. 4: Preconsolidation pressure of a lacustrine clay deposit

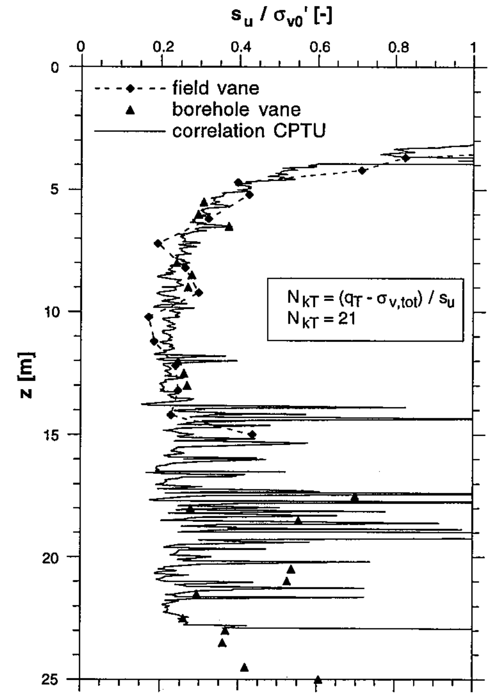

Fig. 5: Evaluation of N_{kT} for the lacustrine clay deposit (c.f. Fig. 4)

4. INTERPRETATION AND USE IN GEOTECHNICAL DESIGN

In Switzerland cone penetration tests are in almost all cases carried out in conjunction with other investigations. It is rare that, due to local experience, investigation methods like borings and *in situ* tests other than cone penetration tests are omitted and only cone penetration tests are carried out. This is mainly due to the fact that the geotechnical soil conditions vary from one soil deposit to another, as well as within one single deposit. Therefore, in Switzerland, cone penetration tests are used as an integral part of subsoil investigations leading to a good knowledge of the investigated soil.

For soil classification and stratigraphy the charts of Robertson et al. (1986) and Robertson (1990) are mainly used. Another soil classification chart which has proved useful is the one by Douglas and Olsen (1981).

Many of the soft clayey deposits, at certain depths, show distinct thin layers of silts and fine sands. In these cases some additional considerations have to be taken into account (cf. Douglas and Olsen 1981 and Robertson and Campanella 1983): On the one hand, within thin layers the true values for cone resistance and sleeve friction are not fully reached. Further, the stiffness of the underlying layer influences the measurements of the cone resistance even before the cone reaches the interface between the soil layers. On the other hand, when passing the interface between two soil layers the different and depth-dependent response of the sensors measuring the cone resistance and the sleeve friction adds some spurious scatter to calculated values like the friction ratio

$$R_f = \frac{q_c}{f_s}$$

where q_c and f_s are the measured values for the cone resistance and the sleeve friction respectively.

When determining the undrained shear strength s_u of cohesive soils from penetration tests, in general the formula

$$s_u \cdot N_k = q_c - \sigma_{vo}$$

is used where σ_{vo} is the total vertical overburden pressure and the values for N_k are taken from the corresponding chart by Jamiolkowski et al. (1988).

Most engineers determine the friction angle of non-cohesive soils according to Durgunoglu and Mitchell (1975), whereas for relative density there is no commonly adopted evaluation method.

Apart from using results of cone penetration tests for stratigraphy or for the general evaluation of soil parameters they are used as well for direct applications in geotechnical design (see e.g. Amann and Wollenhaupt 1983, Amann et al. 1988). When calculating the stability of embankments or the earth pressure on rètaining walls the increase of the undrained shear strength of a soft clay with depth is taken into account. In this case the comparison of the measured cone resistance with results obtained from laboratory tests or field vane tests yields a detailed profile of the undrained shear strength. Determining the distribution of the relative density of a non- cohesive soil deposit is another convenient application in this context. In general, such applications strongly depend on the local geotechnical conditions and on the demands and details of the related project.

5. COMPARISON WITH OTHER INVESTIGATION METHODS

Referring to comparisons of results obtained from cone penetration tests with results obtained from other investigation methods, in Switzerland experience could mainly be gained when determining the undrained shear strength of soft normally consolidated clays. Following Bjerrum (1954) and related publications, it is assumed that in normally consolidated clays the ratio of undrained shear strength s_u and the effective overburden pressure σ_{vo}' is approximately constant. With the penetration resistance $q_T - \sigma_{vo}$ in proportion to the undrained shear strength of the soil, the effective preconsolidation pressure can be derived from the relationship between the penetration resistance and the effective overburden pressure σ_{vo}' (Fig. 4a). For the example given in Fig. 4a there is a good agreement with results obtained from oedometer tests and with a general knowledge of the stress history of this deposit. The high values of B_q in Fig. 4b clearly indicate the occurrence of soft clay layers below a so-called drying crust. The subsequent correlation for the empirical factor N_{kT} is achieved by dividing the shear resistance obtained from other tests by the effective overburden pressure σ_{vo}' (Fig. 5). Another example is given in Fig. 6. Here the undrained shear strength obtained by cone penetration tests and dilatometer tests is compared for a highly sensitive deposit, showing a good agreement over wide depth ranges.

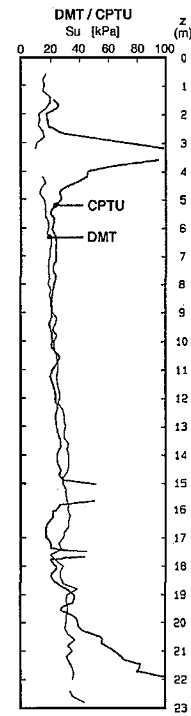

Fig. 6: Undrained shear strength obtained from CPTU and DMT tests

6. RESEARCH ACTIVITIES

Up till now the interpretation of cone penetration test results with regard to the shear strength of cohesive soils has been based mainly on empirical correlations, which, despite their importance, are frequently considered to be unsatisfactory (Wroth 1988). There is only limited knowledge about the stress and strain fields induced in the soil by the insertion of a cone penetrometer.

Approaches used for the analytical modelling of the cone penetration test such as the Cavity Expansion Theory (Vesic 1972) or the Strain Path Method (Baligh 1985) are based on simplified kinematics. When using the Strain Path Method, in addition either the constitutive law for the soil or equilibrium is not satisfied (Baligh 1985).

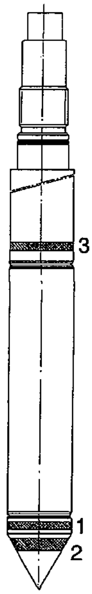

Fig. 7: Piezocone design with three pore pressure elements

The relationships between the failure mechanism induced by the cone penetrometer and the mechanical properties of the soil are considered to be a major topic of further research on cone penetration testing.

Within an ongoing research project at the Swiss Federal Institute of Technology, Zurich, a newly developed 10 cm^2 cone penetrometer, capable of measuring the cone resistance, sleeve friction and the pore pressure at three different locations simultaneously (Fig. 7) is to be used. Thus an improved knowledge of the generated pore pressures and the behaviour of different types of soil during failure induced by penetration will be gained. In this context, the possibilities of determing shear strength from cone penetration tests through the comparison with results obtained from laboratory investigations on undisturbed samples will be researched. While the determination of empirical correlations for the tested soil types remains an important secondary aim, the main emphasis of this research is the analytical modelling of the cone penetration test.

Based on the results of this research, at a later stage, the comparison of the analytical model with measurements of the pore pressure distribution and the strain field obtained from a full scale model test is aimed at.

7. FUTURE TRENDS

Due to the geological conditions, in Switzerland borings in conjunction with Standard Penetration Tests will always play an important role. In the deposits of soft clays and fine grained non-cohesive soils, however, piezocone tests provide information that is not accessible by other investigation methods, or that can only be obtained by a much greater effort. This finds its expression in the answers to the questionnaire that was sent out, when it is stated that, compared to other investigation methods, cone penetration tests were carried out either more frequently or in a constant proportion within recent years. Almost all engineers that are concerned with cone penetration tests would welcome a more frequent use of this test.

From the technical point of view it is seen that an additional measurement of the pore pressure on the cone tip would provide some additional information about the mechanical behaviour of the penetrated soil and would give a possibility of checking the saturation of the pore pressure transducers and the porous filters.

There is a substantial need for the further development of the "environmental cone" that is expected to become an important tool for the engineer concerned with the exploration of contaminated soil and former waste deposits.

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CPT in TURKEY

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SYNOPSIS : The extend of implementation of CPT during the period of last twenty years in Turkey since the realization of the First European Symposium in Penetration Testing ESOPT-I in 1974, in Stockholm, is given in this report. The response of related institutions and engineers to the questionnaire prepared for this task is carefully evaluated. After a brief information on the geological setting of the country and mechanism of tectonics, the resulting high seismicity is emphasized.

The implementation of other in-situ testing procedures in parallel to the usage of CPT is summarized. The listing of CPT equipment in use with calibration and measurement capabilities is provided. The common procedures used in the practice for the interpretation of the CPT measurements are also discussed. The use of CPT in various aspects of geotechnical design in variety of civil engineering structures are demonstrated by means of case studies reported in the literature. Correlation of CPT with the results of other methods are also discussed with specific emphasis on SPT blow counts and shear wave velocities. Further, major areas of research and future trends and new developments are evaluated.

1. INTRODUCTION

First National Report for Turkey on Penetration Testing was prepared in 1974, Durgunoğlu and Togrol (1974). At that time SPT testing was the only tool in penetration testing, and very few application of CPT were available. The first usage of CPT in Turkey was realized in construction of Alsancak Harbor in Izmir in late 1950's. During the period of 1975-1995, extensive development were realized in Turkey in the utilization of CPT testing. The CPT test is very effective because there is many large areas with alluvial soil deposits and subject to major developments due to their preferred topography. In addition various cost effective soil improvement techniques such as, preloading, use of sand or prefabricated drains, dynamic consolidation, stone columns, vibroflotation, compaction piles, mini piles, "jet-grout" columns, etc. are extensively employed for foundations of various civil engineering structures. The use of CPT is also very effective due to its application in a very short period of time and most of the instances time limitation is a very important demand imposed by the client.

For the preparation of this national report, in order to gather as complete as possible information considering that most of the information is either published in internal reports or unpublished, a CPT evaluation form is prepared by the authors and mailed to potential users. In this form, CPT equipment, calibration techniques and corrections, the purpose of the test, the utilization of the measurements, comparison with other measurement techniques and related references were asked to be identified. The purpose of the test is identified as follows :

1. Identification and classification of soils

2. Estimation of soil properties
3. Identification of soil pollution
4. Correlation with other measurement techniques

On the other hand, for the utilization of CPT measurements, eight different options for selection were offered:

1. Slope stability and embankments
2. Retaining structures
3. Shallow foundations
4. Deep foundations
5. Soil liquefaction potential
6. Soil improvement
7. Environmental geotechnics
8. Optimization of alignments

Distribution of the questionnaire between the related groups were as follows:

• Main Contractors	47
• Foundation Subcontractors	15
• Civil Eng'g Consulting Firms	6
• Governmental Institutions-Members	9
• Faculty Members-Geotechnical	30

As expected, very few reply were received from the main contractors, however other groups are contributed with a great enthusiasm. As a result, this report were prepared based on the information obtained from the questionnaire, published articles and authors general knowledge of the state of the CPT in the country.

2. GEOLOGICAL-GEOTECHNICAL CONDITIONS AND SEISMICITY

According to Mineral Research and Exploration Institute of Turkey (MTA), Turkey is bordered in the north by the Russian platform and in the south by the Arabo-Syrian Massif, which constitutes a part of the northern Indo-African platform. Between these great ancient platforms there exist probably less prominent massifs in Turkey, where the early Paleozoic beds lie unconformably on their substratum. It is possible to consider this massif as a dome consisting of series of beds. From the top downward, stratigraphic series may be represented by Marbles and Crystalline series (Early Paleozoic) which show increased metamorphism going downward and comprise sericite, chlorite and mica schists with biotite. In some areas non-metamorphosed limestone outcrops are encountered which overlies the marbles or the schists of the massif. Various geological units are located in Turkey depending on the locality and the seismicity of the region. In any how the seismicity of Turkey is predominant factor in engineering design and therefore seismicity and geology should be evaluated together. For example, in the site of 1976 Çaldıran Earthquake which caused heavy damage, geology was classified into quaternary sediments, tertiary and mesozoic sedimentary rocks and metamorphic rocks.

According to general outline of the seismicity of Turkey, several aspects of plate tectonics theory is exceptionally well applicable in certain parts of the country. In the north, seismic activity is associated with the slip of the North Anatolian Transform Fault which is quite similar to the San Andreas Fault in California. It has also a well exposed fault trace, creep and predominantly right-handed motion. The East Anatolian Fault connects the North Anatolian Fault and the Dead Sea rift to form a triple junction at Karlıova in east central Turkey. The East Anatolian Fault shows sinistral motion while the Anatolian Plate moves westwards. In the southeast, there is a major thrust zone. In western Turkey, however, seismic activity is associated primarily with normal faulting. The largest earthquake magnitudes expected over a period of seventy-five years predicted using Gumbel's third asymptotic distribution of extreme values by Burton et al (1984) indicates that highly populated and industrialized regions of Turkey namely Marmara and Aegean regions, are under a risk of high seismicity. Expected seventy-five year earthquake magnitudes are in the range of M=6.5 to 7.5. Therefore, in addition to North Anatolian fault related seismic activity, the seismicity of the western Turkey is also quite high. The complicated geometry of the faulting in this area and the interaction between adjoining microplates create a complicated strain pattern and as a result, strike-slip faulting at the two western strands of the North Anatolian fault, normal faulting at the Marmara and Menderes graben systems and thrust faulting in southwestern Turkey are observed, Dewey and Şengür (1979). As a result of high seismic activity, based on the previous records and risk assessments, country is divided into five seismic zoning for engineering design and construction as shown in Figure 1, Durgunoğlu (1994).

Invariably most of the developments occurred at flat areas, shorelines, alluvial basins due to their attraction in ease of transportation. In such areas, problematic subsoil conditions, loose granular soils and soft clays do prevail with high ground water table. As a result, in combination with high seismicity of the country the use of CPT became indispensable during the last twenty years period in Turkey.

3. IN SITU TESTINGS

Most common in situ measurement technique in Turkey is SPT performed within the borehole. Especially during last ten years, the importance of realization of standard testing technique and possibility of major differences in blow counts otherwise, is much more recognized by the engineers and technicians. Whenever the utilization of SPT is not possible, boulders, large gravels, weathered rock, fractured rock, etc., pressuremeter testing is performed, Durgunoğlu (1983). Usually Menard type pressuremeter is utilized for this purpose for both strength and modulus measurements. According to final information obtained, total of five different pressuremeter testing units are available in Turkey in governmental and private sector. Pressuremeter measurements were also utilized for determining the extent of soil improvement below a historical mosque as a result of extensive cement grouting, Karadayılar and Durgunoğlu (1990).

The dynamic penetration test is not extensively used in Turkey. There are few light-weight sounding equipment available and they are only utilized in very routine soil investigations for small, relatively unimportant structures, Kayalar (1990).

Vane shear testing is utilized for the purpose of determining in-situ undrained shear strength of the fine grained soils.

The utilization of CPT testing in Turkey is mostly for soil investigation and foundation design purposes for various kinds of civil engineering structures. Due to high seismicity, in general the prediction of soil amplification and soil liquefaction potentials utilizing CPT is also a main concern.

The effectiveness of soil improvement using various techniques is also evaluated by means of CPT in various projects. A case study for a major steel complex development is presented to this symposium, Durgunoğlu et al (1995a). In addition, in this case study, the conclusions of soil improvement utilizing CPT testing as a result of stone column application is further verified by means of large-scaled (4.0mx4.0m) surface loading tests.

4. CPT EQUIPMENT

There are considerable number of CPT equipment being used in Turkey presently. Listing of such equipment is given in Table 1. It is seen that most of the equipment is originated from Netherlands. Governmental institutions, universities, consulting firms and foundation subcontractors retain the equipment. There are few locally made CPT equipment available only under low capacity and utilize mechanical cone. For calibration, usually original units of the manufacturer is utilized. Attention is given to zero shift in the records of electrical cones, especially when soundings are performed through soft clays.

The results usually presented in the form of unit tip resistance, q_c versus depth and unit local skin friction, f_s versus depth. In case of electrical cones, continuous graphs will be presented in opposite to mechanical cones where the values of tip and skin resistance are only measured at certain intervals. Majority of the time the

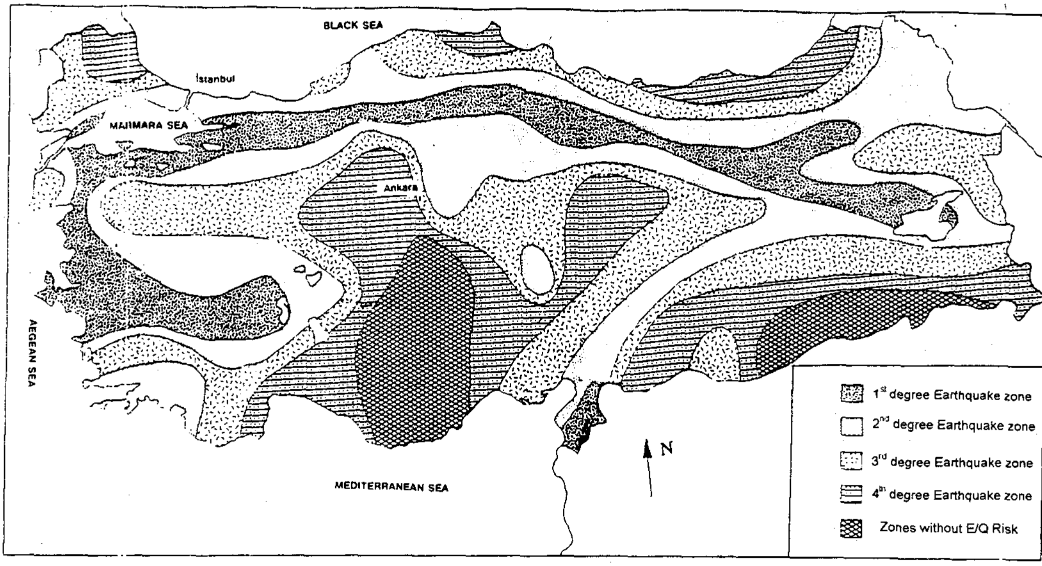

Figure.1. Seismic Zone Map of Turkey

Table. 1. The listing of CPT equipment in Turkey

Institute or Company	Brand	Type	Calibration	Measurement
GOVERNMENTAL INSTITUTES				
Turkish Highway Directorate (KGM)	Gouda, 200kN	Electrical	Gouda Unit STU-100	Tip+Skin
Turkish Highway Directorate (KGM)	Maihak, 25kN	Electrical-Vibrating wire	Local	Tip+Skin+pwp
Turkish Railway Department (DLH)	Dutch, 100kN	Mechanical	Not operating	Tip
Turkish Water Authority (DSI)	Gouda, 200kN	Electrical-Mechanical	Gouda Unit	Tip+Skin
UNIVERSITIES				
Istanbul Technical University (ITU)	OYO, 40kN	Electrical	Zero shift	Tip+Skin
Karadeniz Technical University(KTU)	Gouda, 75kN	Mechanical	Manometer	Tip+Skin
CONSULTING-SOIL INVESTIGATION (PRIVATE)				
Geoteknik	Locally made, 100kN	Mechanical	Manometer	Tip+Skin
STFA-Temel Investigation Co.	Locally made, 25kN	Mechanical	Manometer	Tip+Skin
ZETAS Earth Technology Corp.	A.P. v.d. BERG, 100kN	Electrical	Zero shift	Tip+Skin+pwp+ conduct.+ water& gas sampling
FOUNDATION SUBCONTRACTOR (PRIVATE)				
MENSOY Construction Co.	Gouda, 200kN Gouda, 100kN	Electrical Mechanical	Gouda Unit Manometer	Tip+Skin+pwp Tip+Skin

continuous variation of friction ratio versus depth is also plotted.

In few instances the piezocone were utilized for the measurement of excess pore water pressure. Most of the time the response of the piezo element were poor due to smear effect when such cones were utilized especially in the soundings of organic clays. In one project the decay of penetration pore pressures were observed. In addition, in one instance pore pressure measurements were used to predict the development of excess pore water pressures as a result of pile driving, Durgunoğlu et al (1995a).

The test is conducted according to relevant ASTM standard D3441.

5. INTERPRETATION OF TEST RESULTS

Utilization of CPT in standard soil investigations became common practice, if the subsoil conditions are soft, or there may be a demand of soil improvement as a result of such investigations, Ergun (1982).

Efforts made by writing articles for local engineers and technicians in Turkish language on the effectiveness of utilizing CPT testing on various foundation engineering problems such as soil investigations, correlations with other methods, determination of pile capacities and others are proved to be an effective way in achieving to this point in the utilization of CPT in 1995 in our country, Aksoy (1989), Yetimoğlu and Sağlamer (1990), Egeli (1990).

The CPT testing is most commonly utilized within a soil investigation program which permits ease in interpretation of the results. Although common charts given by Robertson and Campenella (1988) for soil identification is used, the results are further checked by means of correlation boreholes and laboratory testings.

In certain projects where extensive CPT will be implemented for determination of soil parameters (i.e. shear strength of clays, friction angle of sands, modulus of clays and sands) the common practice is to develop site consistent coefficients using earlier developed general equations utilizing results of conventional testing. In some cases the range of values for parameters is specified and sensitivity analysis of the certain evaluation is recommended.

In such sites with extensive soft clay subsoils, the undrained shear strength, in-situ or/and after the improvement, is also determined by means of vane testing and proper correction coefficient is developed for such a site utilizing CPT cone tip resistance measurements.

In sandy subsoils with known stress history, the interpretation of CPT results for determination of friction angle is performed either using methods proposed by Durgunoğlu and Mitchell (1975) or Janbu and Senneset (1974). Whenever the estimation of relative density is required, the curves proposed by Schmertmann (1978) are utilized.

6. USE OF CPT IN GEOTECHNICAL DESIGN

Utilization of CPT in the period of 1975 to 1995 in Turkey has become very extensive. The CPT is used in the geotechnical design of various civil engineering structures. At early stages after 1974 ESOPT-I Symposium, realization of CPT measurements in Turkey were made by units obtained from outside the country. For instance, in a fertilizer factory in Marmara Region, a steel ammonium tank was ruptured and consequently the operation of the unit was intercepted. The client was interested to repair the tank for existing operations and to realize an immediate construction of a new tank near by the existing unit. In order to find out the causes of rupture and not to meet the similar problems with the new tank near by, the utilization of CPT measurements along the periphery of both tanks were strictly required by the consultant. In such circumstance the contractor is decided to subcontract these measurements to Fugro+Netherlands, due to unavailability of the CPT equipment at that time in Turkey. The evaluation of the measurement clearly indicated the presence of soft local subsoils only under the certain portion of the damaged tank along the periphery. Consequently, the differential settlements realized at certain locations along the periphery is shown to exceed the limits tolerable by the steel tank structure. As far as the author's knowledge, this case was a very straightforward early satisfactory implementation of CPT in a foundation problem in Turkey. In the design of the new unit, obviously settlements are estimated based on the CPT measurements and a rigid special mat foundation were designed to accommodate the expected differential settlements, Tezcan et al (1977).

The extensive applications have occurred as a result of motorway construction program initiated in 1982 as part of the Trans European Motorway (TEM). Turkey is planning to construct total of 3,000 kilometers of motorway until the year of 2001. Already more than half of the project is realized. Obviously Turkish Highway Directorate (KGM) pioneered the utilization of CPT during this period. Table 2 summarizes total of seventeen referenced project performed by KGM utilizing CPT. In the same table, similar seven CPT applications in highways are also summarized. Each use is also categorized based on the purpose and applications in this table. KGM has also offered the usage of CPT equipment to other institutions responsible from the construction of infrastructures in Turkey. In this table, two examples for the application of CPT in railroad and sewerage projects are also listed.

Department of State Hydraulic Works (DSI) has recently purchased a CPT device and initiated its use in water works, DSI (1995).

Turkey is a major grain producing country. As a result, there is a great need of storage silos for grains that is bought from the farmer until its use or exportation. These sites for silos are usually selected so that ease of transportation will be achieved. For this purpose mostly sea transportation is preferred due to cost effectiveness and as a result, silos are forced to be constructed near shorelines or harbors and therefore their foundations should be utilized under mostly soft and loose subsoil conditions. Consequently utilization of CPT at these sites is very effective, and as a result soil investigations could be completed at a relatively shorter period of time with a tight budget and having high quality quantitative geotechnical data Erol et al (1991). For this specific project CPT measurements were performed with a unit from Fugro-Netherlands.

Within the last ten year-period, due to high population growth and a big migration towards large metropolitan areas in Turkey, various governmental organizations such as Turkish Real Estate Bank and Turkish Directorate of Housing have planned and developed major housing projects. Most of these projects covers numerous multi-storey buildings in some cases located at sites with very poor subsoil conditions and high seismicity. Therefore, cost effective soil investigation, building foundations, design against earthquakes and implication of soil improvement makes the utilization of CPT very attractive in comparison to other conventional methods. One of the largest ever housing development project is being realized at Bostanlı-Izmir which is located at the west of Turkey at the reclaimed shoreline with very poor subsoil conditions and with very high seismicity. The extremely high cost of land in big metropolitan areas such as Izmir, makes such major housing development projects feasible cost-wise even under such undesirable subsoil conditions and high seismicity. However, obviously even under such circumstances, developer demands cost effective foundation design and soil improvement. As a case study for such a project, determination of vertical pile capacities of cast-in-situ driven piles and utilization of improved modulus in lateral deflection of such piles using CPT is evaluated in a paper given to this symposium, Durgunoğlu et al (1995b).

Recently vibrex piles i.e. cast-in-situ driven piles using Fundex technique has been realized in various projects with soft subsoils as cost and time effective alternative, especially compared to bored piles. However, in order to justify their use, the availability of CPT data is highly demanded. As a result, local foundation subcontractors purchased and utilized CPT equipment as means of justifying the usability and effectiveness of such piling technique, Soyaçıkgöz (1995). This equipment has been

Table. 2. UTILIZATION OF CPT BY TURKISH HIGHWAY DIRECTORATE

Project	Section-Structure	Purpose				Applications								Reference		
		1	2	3	4	1	2	3	4	5	6	7	8	Authors	Int. Rep. No.	Year
I. MOTORWAYS																
Anadolu Motorway	Adapazarı-Kazancı	√	√		√									Bezirci, M.H.	I-79	1983
Anadolu Motorway	İzmit-Başiskele-Sakarya-Kazancı	√	√		√	√		√	√					Ulukan, B.	I-82	1984
Anadolu Motorway	İzmit-Sakarya		√			√		√	√	√	√			Aksoy, S.	I-83	1984
Istanbul 2nd Bosphorus Crossing	Peripheral Expressway		√		√			√	√					Aksoy & Ulukan	XVII-66	1985
Istanbul 2nd Bosphorus Crossing	Çobançeşme Intersection		√						√	√				Ulukan, B.	XVII-75	1985
Istanbul 2nd Bosphorus Crossing	Peripheral Expressway		√		√	√		√	√		√			Ulukan, B.	XVII-76	1985
Kınalı-Sakarya Motorway	Sakarya Viaduct		√		√				√					Aksoy, S.	I-96	1986
Kınalı-Sakarya Motorway	Sapanca-Adapazarı		√											Uyguner, C.	I-98	1986
Kınalı-Sakarya Motorway	Karasu-Beyikçayırı-Ispartakule-Yarımburgaz Viaducts		√		√				√					Tamgaç & Sert	XVII-83	1986
Kınalı-Sakarya Motorway	Karasu Viaduct		√		√				√					Fırat, Ç. et. al.	XVII-85	1986
Kınalı-Sakarya Motorway	Various Sections		√			√		√	√					Ulukan, B	XVII-86	1986
Kınalı-Sakarya Motorway	Beylikçayırı Viaduct		√						√					Ulukan, B	XVII-87	1986
Kınalı-Sakarya Motorway	Various Intersections	√	√					√	√					Aksoy & Özcan	XVII-96	1989
TAG Motorway	Evri Swamp Crossing	√	√		√								√	Aksoy & Karahan	H-521	1989
TAG Motorway	Ceylan Plain Crossing		√		√	√							√	Müftüoğlu, E.	H-617	1990
Kınalı-Sakarya Motorway	Yarımburgaz Bridge Approach Fills	√	√		√	√					√			Aksoy & et. al.	XVII-66	1993
Ist.-Bursa-Balıkesir-İzmir Motorway	Bursa-Susurluk	√	√										√	Aksoy, S.	14-58	1994
II-HIGHWAYS																
İzmir Intercity Crossing	Turan Intersection Bridge		√						√					Bezirci, H.	II-31	1982
İzmir Intercity Crossing	Zafer Payzım Intersection Bridge		√		√				√					Ulukan & Aksoy	II-35	1984
Bursa-İnegöl-Bozüyük	Intersection Bridge	√	√		√	√								Aksoy & Özcan	XIV-51	1991
Bursa-İnegöl-Bozüyük	İnegöl-Garaj Intersection Bridge Approach Fills	√	√		√	√			√		√			Aksoy, S.	XIV-52	1991
Manavgat-Alanya	Akseki Intersection Approach Fills	√	√		√	√					√			Etkesen & Kızıroğlu	XIII-16	1993
Afyon Intercity Crossing	Intercity		√		√						√			Aksoy, S.	H-762	1994
Bursa-Gemlik-Armutlu	Gemlik City Crossing	√	√		√	√					√			Özcan, M.	XIV-60	1995
III-RAILROADS																
Hanlı-Bostankaya Railway	Section I	√	√			√		√	√		√			Aksoy, S.	H-457	1987
İzmir Crossing	Intercity	√	√			√		√	√		√			Aksoy & Müftüoğlu	H-481	1988
IV-WATER SUPPLY-SEWERAGE																
Samsun Water Serewage	Landslides		√		√	√					√	√		Ulukan, B	H-301	1983
Ankara-Dikmen Sewerage	Soil Investigation	√	√		√									Ulukan & Aksoy	H-377	1985

LEGEND

PURPOSE
(1) Soil identification and classification
(2) Soil properties
(3) Soil pollution
(4) Correalation with other in-situ testing

APPLICATIONS
(1) Slope stability-embankments
(2) Retaining structures
(3) Shallow foundations
(4) Deep foundations
(5) Soil liquefaction
(6) Soil improvement
(7) Enviromental geotechnics
(8) Alignment optimization

utilized in various projects, including housing, commercial centers, and viaduct foundations.

Large scale industrial complexes i.e. Toyota-SA auto factory in Adapazarı, Borçelik Steel Factory in Gemlik, are constructed at locations of flat areas or near shorelines, both of them under very high seismicity. The utilization of CPT is proved to be very effective in the assessment of liquefaction potential and determination of pile capacities, Durgunoğlu, A.T. (1993), Durgunoğlu, H.T. (1995a).

In one project CPT is also effectively utilized in optimization of motorway alignment in plan and profile, through a swamp area, Durgunoğlu et al (1992).

CPT is also utilized for determination of the extend of the improvement as a result of dynamic consolidation in a harbor storage area, Önalp (1985) and optimization of soil improvement using stone columns in a steel complex, Durgunoğlu et al (1992).

During March 13, 1992 Erzincan Earthquake with magnitude of M=6.8, saturated sandy silt soil deposits are liquified in Ekşisu Region. The soil conditions and liquefaction potential of deposits are studied utilizing extensive SPT and CPT testings, Erken et al (1993).

Most recently the utilization of CPT equipment in environmental geotechnics has been realized in Turkey. ZETAŞ Earth Technology Corp. utilized conductivity soundings in addition to usual measurement of tip and skin resistances using electrical cone. The measurements are performed at the abandoned solid waste dump areas of Municipality of Izmir, in order to determine the extend of solid waste and pollution at the subject sites. In addition, with the aid of the CPT equipment using a special filter and a sampler, it was possible to obtain water and gas samples at various locations and depths at the subject sites using a special electrical pump unit. The results so far obtained are very encouraging and a final report and remedial design of the sites will be developed based on the findings.

7. CORRELATION OF CPT WITH OTHER METHODS

One of the interesting research project is conducted as a joint effort between Turkish Highway Directorate Research Department and Middle East Technical University. In this project, comparison of SPT and CPT measurements i.e. q_c versus N correlations together with laboratory measured modulus M_c versus q_c correlations are developed for a specific site in Ankara having stiff clays, Özkan et al (1990). The results are compared with previously developed empirical relations in the literature.

Following the March 13, 1992 Erzincan Earthquake, a detailed site investigation were carried out utilizing SPT, CPT, and seismic wave velocity measurements using both down-hole and cross-hole techniques. Site specific correlations applicable for alluvial soils between shear wave velocity, SPT-N and CPT-q_c values are developed and compared with some other empirical correlations already developed in the literature, İyisan and Ansal (1993)

8. MAJOR AREAS FOR RESEARCH ACTIVITIES

A research proposed by Durgunoğlu at Boğaziçi University is undergoing to develop correlations between CPT-q_c and v_s using seismic cone. The utilization of seismic cone will enable to measure the v_s values almost at the same location of the previous sounding. Another research project proposal is under development between Boğaziçi University and State Highway Directorate Research Department for developing correlations between the results of various in-situ measurement techniques. This project is considering to construct a stress-controlled test chamber to achieve well defined and controlled stress-strain conditions.

9. FUTURE TRENDS AND NEW DEVELOPMENTS

It is clear that the use of CPT will be enhanced in the coming years considering the availability of the equipment locally and its inherent advantages compared to other testing methods. In special instances where the stress history of the subsoil is not known simultaneous use of dilatometers or pressuremeters is planned to be realized

Further, extensive use and preference of CPT for measurement of dynamic soil properties and for the assessment of soil liquefaction potential compare to other procedures in the near future is expected. Due to high interest on the rehabilitation of existing solid waste disposal sites and selection and evaluation of potential landfill sites in large cities, the extensive utilization of conductivity cone and environmental cone will be realized. Further, in the near future, present trends of limited use of piezocone will be changed by the realization of more effective and trouble-free piezo elements.

It is also expected that in the near future, the utilization of CPT technique in the investigation and solution of offshore problems will be realized. This is due to the fact that country must realize the construction of pipelines for various offshore discharge of water, treated in near shore sewerage treatment plants. In the past for a project of offshore sewerage pipeline in Gemlik Bay in order to measure undrained shear strength of very soft sea bottom sediments (C_u~2-3kN/m^2), diver operated and locally developed vane shear device was utilized in Turkey, Durgunoğlu et al (1980). There is also a need for the realization of great water supply project for the city of Istanbul which will require the crossing of the pipeline through the Bosphorus. Further, the planned major projects such as the railway and subway tunneling under the Bosphorus and Izmit Bay crossing with a long major bridge within the Istanbul-Izmir motorway project will further amplify the need of CPT for the determination of geotechnical properties of sea bottom sediments.

10. ACKNOWLEDGMENT

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CPT95 - National Report on UK Practice

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SYNOPSIS: This is a report on the state of practice of CPT/CPTU testing in commercial site investigations in the UK. It briefly gives a geological and geotechnical description of the UK highlighting the complex nature and extent of naturally occurring deposits which in geotechnical terms are classed as soil. Different site investigation techniques in common use and in development are referred to emphasising the role CPT/CPTU testing takes. CPT/CPTU testing represents about 3% of the turnover of UK site investigation for the construction industry. CPT testing is most often carried out with a 15 cm^2 cone following procedures in accordance with BS1377:1990 which is similar to the International Reference Test Procedure. There is increasing use of the CPT/CPTU and other sensors including pressuremeter, seismic and environmental sensors.

1. INTRODUCTION

This is the National Report on the current state of practice for static electric cone penetrometer testing in commercial site investigations in the United Kingdom. It was prepared at the request of the secretariat of the International Symposium on Cone Penetration Testing (CPT'95) held in October 1995. It primarily covers electric friction cone (CPT) and piezocones (CPTU) with reference to other sensors where relevant. It excludes dynamic and mechanical cones. It concentrates primarily on land and near shore testing although some UK contractors and consultants also operate CPT/CPTU in deep water.

The Report is based on discussions with specialist contractors operating CPT/CPTU equipment, results of questionnaires and published information. Questionnaires covering the topics discussed below were sent to main contractors, consultants and client organisations who were known to be actively involved in the geotechnical engineering input to projects.

2. GEOLOGICAL AND GEOTECHNICAL DESCRIPTION OF THE UK

2.1 Geology

Naturally occurring deposits formed prior to the Pleistocene glaciation are generally referred to by geologists as rocks and comprise the 'solid' formations shown on geological maps. The strength of these 'solid' formations varies such that some of them are classified for engineering purposes as soils. These include, for example, many of the deposits in the south east of the UK such as London Clay and Gault Clay. Naturally occurring deposits formed after the Pleistocene glaciation are generally referred to as 'drift' deposits and these are predominantly soils.

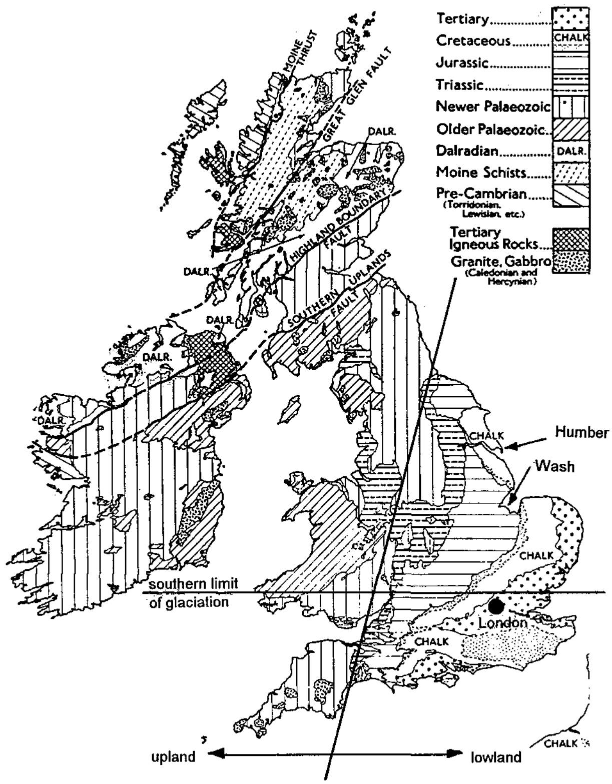

Figure 1 Main geological features of the UK (after Blyth and de Freitas, 1984)

The geology of the UK and hence the distribution of soil and rock types is complex. The UK can be divided approximately into two areas by the diagonal line shown on Figure 1. To the north and west of that line the dominant character is upland dissected by plains and valleys of limited extent. This area is underlain by rocks which are mainly older than the Carboniferous Coal Measures formed during the Palaeozoic period. The land to the south and east is undulating lowland underlain by rocks younger than the Carboniferous Coal Measures.

Most of the UK has been affected by glaciation which has resulted in extensive drift deposits of till (boulder clay), laminated clays and other glacial materials as far south as London. In the upland area these deposits are confined to the plains and valleys which are where most development has taken place. The rocks forming the upland are either exposed or covered by a thin layer of soil. Periglacial and lacustrine deposits are also found in the plains and valleys together with estuarine and coastal muds and silts. Peat and other organic soils are found throughout the upland region. The majority of site investigation in the upland area is confined to testing drift deposits and establishing rock head.

Most of the rocks in the lowland area are sedimentary deposits laid down in marine or estuarine environments. Many of these strata, formed during the Mesozoic and Tertiary periods, have a relatively low strength and so are classed for engineering purposes as 'soils' (e.g. London Clay) although, at depth, they may become sufficiently strong to be classed as 'rock'. It is also possible to have a profile consisting of alternating layers of 'soil' and 'rock' due to varying degrees of lithification of the sedimentary deposits within the profile.

The rocks are generally named after the principal locality in which the formation is well displayed (e.g. London Clay, Oxford Clay). The actual lithology of a deposit varies and maybe different from that inferred by the name (e.g. Lower Greensand includes clay layers).

The exposures tend to run in the north-east south-west direction and have an influence on the topography since the harder, more resistant layers give rise to scarps. Unlike the upland areas, much of the rock is overlain by drift comprising glacial, periglacial and lacustrine deposits. Major deposits of estuarine and coastal muds and silts are found in areas around the Humber and the Wash.

The Midlands is essentially a drift-covered Triassic plain which includes Mercia mudstones (Keuper marls), limestones, Lias Clay and Bunter sandstone. To the south east of this plain lie the scarplands which are formed of outcrops of Jurassic and Cretaceous rocks which comprise alternate layers of weak and strong rocks. The sequence of rocks is very varied and includes, in the Jurassic sequence, massive limestones and sandstones, Oxford Clay and Kimmeridge Clay. The Cretaceous sequence includes the Lower Greensand, Gault Clay and Chalk. The south east area of the UK consists of the London and Hampshire basins which lie to the north and west of the Weald. The basins, which are bounded by chalk uplands, comprise Oligocene and Eocene deposits including London Clay, Thanet Sand and Bagshot Beds. Terrace gravels and extensive alluvial deposits are found in the London Basin. The Weald, in the south east corner of the UK, is an area bounded by a chalk rim which varies in colour and strength enclosing exposures of Upper and Lower Greensand, Gault Clay, Weald Clay and Purbeck clays, shales and limestones.

2.2 Geotechnical Properties

The variety of soil types ranging from organic deposits to weak rocks implies that the soils in the UK encompass the complete range of strengths and stiffnesses. Drift deposits in the upland region are predominantly firm to stiff to hard gravelly sandy clay containing boulders, lenses of sand and gravel and lenses of laminated clay. The more recent deposits comprise softer organic and alluvial clays and loose sands.

Pre glacial soils in the lowland area are generally either stiff to hard clays or dense to very dense sands. These are particularly prevalent in the south east and include, for example, London Clay and Thanet Sand. Glacial soils of similar properties to those in the upland region are found as far south as London. More recent drift deposits include medium dense terrace gravels, soft organic clays and peats and head deposits.

3. SITE INVESTIGATION PRACTICE IN THE UK

3.1 General

Site investigation in the UK, carried out in accordance with BS5930:1981, is used to determine stratigraphy, obtain profiles of ground properties and produce design parameters. The techniques used have evolved from the location of areas of development and their geology. For example, the presence of stiff clays, e.g. London Clay in the South East of England, has led to the development of U100 sampling and triaxial testing.

It is common practice to use shell and auger rigs to obtain driven U100 tubes of cohesive soils and carry out SPT tests in granular deposits. Rotary coring is used in rock and piston sampling in soft clay. Rotary cored and pushed tube samples of stiff clay and dense sand are increasingly being used especially on larger projects.

UK practice tends to be based on indirect design methods rather than direct design methods. Design parameters are most commonly obtained from classification, quick undrained triaxial and oedometer tests on samples of clay, and SPT tests in sand. On larger projects it is common to include effective stress testing of soils and weak rocks, prebored pressuremeter testing in weak to strong rocks and self-bored pressuremeter testing of clays, sands and weak rocks. Dynamic probing and sampling are increasingly being used to obtain soil profiles when designing foundations for low rise structures and to assess areas of instability.

CPT/CPTU testing is being increasingly used to compliment all of the above SI techniques (see Section 3.2).

Geophysical testing including radar, seismic and resistivity techniques, are becoming more common.

Improvements in drilling, sampling, field and laboratory testing, data management and quality control are continually taking place such that UK SI practice is developing to meet the needs of the geotechnical engineering profession.

Most major consultants and some client organisations include geotechnical sections. There are a few geotechnical consultants. Site investigation tends to be carried out by nominated contractors or by contractors who compete on a tender basis. Site investigation contractors range from the small one rig operators to the major companies offering a range of services in all aspects of geotechnical engineering. There are a number of specialist sub contractors who operate a particular piece of equipment for example CPT operators.

3.2 CPT/CPTU Testing

CPT testing has seen a steady growth from the single cone truck in the 1970s to the present range of trucks, crawler mounted rigs and demountables (see section 4.1). The development of the CPTU, the availability of additional sensors and the realisation of the potential of CPT/CPTU testing has increased interest in the method.

The decision to use and specify CPT/CPTU is most often taken by consultants (over 70% of the cases) who are predominantly geotechnical and environmental consultants, with clients directly specifying its use in about 10% of CPT/CPTU jobs. The remaining 20% originate directly from either the main or sub contractor.

Nearly all the CPT/CPTU testing in the UK is undertaken by specialist contractors most often working as sub-contractors to the main site investigation contractor, but also directly for the client.

CPT/CPTU testing in the UK is now being carried out in all types of soil and occasionally weak rock. Table 1 gives an approximate breakdown of testing by soil type. CPT/CPTU testing is also used in fill but usually only to identify the extent of the fill rather than to determine its characteristics.

Table 1 Types of soils in which CPT/CPTU are used

Soil Type	% of CPT/CPTU Testing
alluvium	25
stiff overconsolidated clay	10
till (boulder clay)	10
sands and gravels	20
chalk	15
fill	20

CPT/CPTU testing is used to varying degrees throughout the country depending upon current construction activity, on local soil types and, to some extent, the previous experience of local consultants. Around 90% of the UK testing is in England with over 60% being concentrated in the southern part of the country.

Currently the division of work between CPT and CPTU in the UK is about 75% and

25% respectively of a total of around 80,000 metres per year. This represents about 20% of the annual exploratory meterage for site investigation for construction which, in terms of turnover, is about 3% of the total annual site investigation budget.

In many cases the main reasons for specifying CPT/CPTU are for soil profiling and classification and in the extrapolation of borehole and laboratory test information around a site. Derived parameters may range from simple estimates of shear strength or relative density to the full range of properties that can be assessed from the CPTU.

The main use of the CPT/CPTU in design is in indirect design methods based on the derived soil properties. There is, however, an increasing use in the UK of CPT/CPTU in empirical design formulae such as for pile design.

In addition to the above uses CPT equipment is now being used for sampling, (e.g. with the Mostap sampler) and increasingly in applications where other sensors such as environmental, seismic and pressuremeters are the primary tool being used. This is seen as an expanding area of development in both commercial practice and research.

4. CPT/CPTU EQUIPMENT AND TEST PROCEDURE

4.1 Equipment

Both 10 cm^2 and 15 cm^2 CPT and CPTU equipment is currently in use in the UK. It is operated from self propelled 20 tonne trucks, heavy and lightweight crawler mounted rigs and demountables with a range of pushing capacity and bearing pressures thus allowing equipment to be used in and on a variety of soil and surface conditions. There are currently four contractors undertaking land and shallow water work operating a total of six trucks, three crawler rigs and numerous demountable rigs which are typically used for shallow water work. Two of the contractors also frequently operate in the deep water environment.

Most cones have inclinometers fitted. Increasing use is made of seismic and pressuremeter cones to determine stiffness profiles. Environmental cones including resistivity, redox potential, temperature and conductivity sensors are being developed and used.

Over recent years there appears to have been a contractor-led move away from the use of 10 cm^2 cones to 15 cm^2 cones, to a position currently where 70% of the UK land testing is being carried out with 15 cm^2 equipment. This is particularly true of CPT testing where the robustness and friction-reducing effects of the 15 cm^2 cone (on 10 cm^2 rods) is preferred in the variable UK soils. For CPTU the work is equally divided between the two cone sizes.

The basic equipment in use for CPT/CPTU satisfies either BS1377:1990 or the International Reference Test Procedure (IRTP); the differences between these two documents are discussed briefly in Section 4.4. Neither document deals with the 15 cm^2 cone and so tolerances etc. specified for the 10 cm^2 cone are generally applied to the 15 cm^2 cone. Friction sleeves on the 15 cm^2 cone are usually 200 cm^2. The specification for the shoulder length behind the cone tip is often not adhered to strictly.

BS1377:1990 refers to filters but does not explicitly refer to CPTU testing. Contractors tend to follow IRTP guidelines when using CPTU. Both face and shoulder filters are used and the choice is either left to the contractor or specified. Face filters are more commonly used for profiling.

4.2 Test Procedures (and Calibrations)

The specialist contractors undertaking the work carry out testing in accordance with either an in house specification or BS1377:1990 both of which are based around the IRTP. The in house specification can be either one of their own or that of one of the organisations commissioning the work. There are other standards used including the UK Department of Transport Specification.

Calibrations are generally carried out in accordance with BS1377:1990 which specifies checks to be made every 2000 m of testing or whenever any repair or overhaul is undertaken. Additional calibrations are sometimes carried out before and after each contract and every three to six months. The frequency of calibrating equipment is often left to the discretion of the specialist sub contractor operating the cone equipment. Sometimes more frequent calibrations are specified especially when a high level of accuracy is required. Calibrations are usually carried out in central laboratories though simple on site checks are made if specified. Contractors maintain records of calibrations and repairs to equipment to demonstrate repeatability. Consultants often insist that a contractor satisfies the quality assurance arrangements and specification and therefore do not require additional site calibrations. An increasing number of site investigation contractors are being accredited through NAMAS (the National Measurement Accreditation Service, a service of the UK National Physical Laboratory).

Measurements of all sensors are typically taken every second, recorded automatically and stored on magnetic media for further processing. Other rates of data sampling are used when specified especially when carrying out dissipation tests.

4.3 Correction and presentation of results

Data are automatically logged and stored on magnetic media for post processing and data transfer. Some processing of data is undertaken prior to producing the factual report. This includes removal of spurious data points, depth correction for sensors remote from the tip, and depth correction to allow for deviation from the vertical which is assessed from inclinometer readings.

Site and profile specific information conforming to BS1377:1990 is supplied (see Table 2). Profiles of q_c, f_s and q_c/f_s (and u for CPTU) are routinely produced together with a stratigraphical profile from all CPT tests. Additional profiles of q_t and q_{net} are produced from CPTU tests, but often only at the request of the engineer. Values of q_t should be derived from pore water pressure measurements on the shoulder, however, it is still common practice in the UK to use face filter pore pressure measurements multiplied by standard factors to derive an equivalent 'shoulder' value. The graphs for each profile are plotted on a single A3 sheet and presented in a factual report. BS1377:1990 gives no guidance on suitable scales for plotting the data; typically a scale of 1:100 is used for depth with the derived parameters being plotted at scales similar to those recommended in the IRTP. Alternative scales are used when specified. Data are often requested and

Table 2 Information to be submitted with each profile

1. method of test used
2. graphs with respect to depth of q_c, f_s, R_f and u when appropriate
3. inclinometer readings, when taken, beyond a limit to be specified
4. capacity and type of penetrometer
5. type of resistance measuring system
6. the type of tip and the conditions it is in with respect to wear
7. the position and type of filters
8. the depth over which a friction reducer or pushed rods with reduced diameter has been used.
9. details of any abnormality of unusual event
10. observations of sounds or unusual vibrations from the push rods
11. depth to water level in the hole after the withdrawal of the tip
12. whether or not the hole was backfilled
13. profile location and number
14. date of test
15. weather at time of test
16. name of organisation and operators who carried out the test.

supplied on disk in a format given by AGS (1992) as a standard and saved as ASCII files.

Data from other sensors are plotted according to the type of test being carried out, for example pore pressure dissipation curves from CPTU tests and applied pressure against cavity strain for cone pressuremeter tests

4.4 National Codes and/or Standards

Most operators apply BS1377:1990 or use in house specifications based on IRTP. The specification in BS1377:1990 differs slightly from the IRTP. Some of the main differences are as follows:

a) BS1377:1990 sets tighter tolerances for the cone diameter and wear.

b) It does not specify in detail requirements for CPTU equipment and procedures.

c) The cylindrical extension or shoulder on the cone is specified as 2 to 5 mm in length compared with 7 to 10 mm in the IRTP. This is probably as a result of (b) above.

d) The specified calibration accuracy is a little more ambiguous than in the IRTP.

5. INTERPRETATION OF TEST RESULTS

As mentioned in Section 4.3 CPT/CPTU data are always interpreted by the contractor to give stratigraphical profiles. Further interpretation by the contractor is only undertaken if specified. Many consultants use the data given in the factual report to carry out further interpretation to derive soil parameters. Interpretation is usually based on experience, in house databases and readily available summaries of correlations (most often those reported by Meigh, 1987). For more detailed interpretation and especially for CPTU testing more recently published correlations are sought and used.

5.1 Soil Classification and Stratigraphy

These are produced as part of the factual report using in house and published correlations such as those given by Meigh (1987).

5.2 Soil Parameters

A number of references are used to determine soil parameters from CPT/CPTU tests; the most widely used set of references being those cited by Meigh (1987). The parameter most often derived from tests in clay is the undrained shear strength. Relative density and angle of shearing resistance are often determined from tests in sand. Drained and undrained moduli of elasticity are also derived together with the coefficients of consolidation and permeability, the latter being determined from CPTU tests only. Some consultants use CPTU tests to determine the overconsolidation ratio.

Derivation of parameters from other sensors is based on recent publications and current research especially as some of the sensors are still in their development stage.

6. USE OF CPT IN GEOTECHNICAL DESIGN

The major use of CPT testing in design is for shallow and piled foundations and retaining walls. Design of foundations is primarily based on indirect methods but some direct methods are used. CPT tests are often used in conjunction with other tests to develop profiles of soil types and to determine representative parameters for design. Other examples of geotechnical problems in which CPT/CPTU have been used are investigating failures due to liquefaction, lateral loading of piles, compaction control, and determining the extent of contaminated land.

More specialist uses of CPT testing include estimating volumes of spoil heaps and assessment of rock quality at the base of bored pile foundations in weak rock.

7. COMPARISON AND CORRELATION OF CPT WITH OTHER INVESTIGATION METHODS

CPT results are often used in conjunction with results of other tests to obtain site specific correlations. These and in house correlations supported by published correlations are the prime means of obtaining soil parameters.

8. MAJOR AREAS FOR RESEARCH ACTIVITIES

Although CPT/CPTU testing is slowly increasing in popularity in the UK the test is still not used to its full potential. Test specifications appropriate to the different UK soil types tested and the levels of information required are being developed. Research is continuing into improving methods of interpretation of test data for the various soil parameters that can be deduced so that guidance may be given on the most appropriate procedures for different soil types. Effort is ongoing in broadening the database of experience in different soil types to promote confidence in the capabilities of the test.

A major part of current research is the continuing development of equipment containing further sensors together with the interpretation of their output.

Research is being carried out on the performance and interpretation of the triple element piezocone which contains filters in the face, shoulder and friction sleeve. This equipment has been developed to give more information about the pore pressure regime around the tip during penetration, which should enable soil parameters to be determined more accurately than from standard CPTU tests. In particular the assessment of coefficients of consolidation and permeability from dissipation tests should be enhanced.

Research is continuing into the interpretation of seismic cones and the enhancement of the equipment. Much attention is being given to linking small strain shear modulus values from different measuring techniques and to the prediction of small strain stiffness profiles in different soil types. The purpose of these studies is to enable results from seismic cone tests to be used to establish soil stiffness at a range of strain levels encountered in the performance of structures.

Cone pressuremeters are being used increasingly to determine the strength and stiffness of soils. The speed of installation and testing, the repeatable disturbance and the CPT results obtained during installation make this equipment of practical interest. Research is being conducted into the interpretation of these tests to take into account the large amount of disturbance to the soil as the probe is installed.

A major area of research is the continuing development of CPT equipment containing additional sensors to establish the environmental conditions. These sensors include conductivity, fluorescence, pH, redox potential, soil moisture and temperature.

Research is also being undertaken in collaboration with other European countries in the development of semi empirical foundation design rules.

9. FUTURE TRENDS

With the dissemination of the results of the ongoing research on CPT/CPTU testing, there is likely to be increasing use of the testing in the UK, with a wider range of applications than at present. This will lead to improvement and enhancement of the UK database and thus to more detailed correlations and further confidence in the test.

The various types of equipment, particularly environmental, described in section 8 which are currently mainly research tools should become increasingly used as the equipment is improved and the results and their interpretation validated.

Accreditation of laboratory and field testing is becoming increasingly common and desirable. CPT/CPTU testing is already becoming an accredited test within the UK and it is likely that more operators will seek accreditation.

10. ACKNOWLEDGEMENTS

This report represents the views of consultants, contractors, clients, and research organisations involved in CPT/CPTU testing and interpretation in the UK. The authors thank all those who have contributed information.

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U.S. National Report on CPT

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SYNOPSIS: Over the two decades since its introduction into the United States, the CPT has now established its position as a routine, reliable, and expedient means for site characterization, stratigraphic profiling, evaluation of soil engineering parameters, and geotechnical design. Piezocones provide measurements of penetration pore water pressures either midface (type 1) or at the shoulder (type 2). The latter is necessary for the proper correction of tip resistance in soft soils, while the former provides better resolution and detailing in stiff fissured materials.

Most areas of the U.S. are now serviced by specialty firms with cone trucks for optimizing production, penetration depths, and areal coverage. Portable cone systems are available for small projects and remote locations. Research on CPT testing and analysis continues to have high priority within many organizations.

The incorporation of additional sensors has increased cone versatility, thus favoring use of seismic-, resistivity-, natural gamma-, and chemical-cones. This is attractive on geoenvironmental projects for expedient contaminant mapping while minimizing site damage and wastes generated from testing.

1. GEOLOGICAL REGIONS

The vast expanse of the United States of America includes many complex, mixed, and varied geomorphological formations throughout the 50 states. An overview of the surficial geologies of the U.S. is given by Hunt (1986).

While CPT is not possible in the very rocky and mountainous regions of the country, the test is very applicable to the coastal regions, inland sedimentary deposits, residual soil types, as well as reclaimed lands formed from hydraulic fills, dredgings, and mine tailings. The cone has become particularly popular for use in the marine sediments of the Atlantic and Pacific Coastal Plain Provinces, the deltaic and marine sediments of the Gulf states, glacial lacustrine deposits around the Great Lakes to New England, and floodplain alluvium from the Great Plains and following the Mississippi River. Throughout these regions, a number of specialized testing firms with cone trucks or trailers to provide CPT services.

2. TYPES OF PENETRATION TESTING

Penetration tests common to U.S. practice include the standard penetration test (SPT) and cone penetration test (CPT). In some regions, the flat blade dilatometer test (DMT) is used. Testing procedures for these three tests are essentially standardized, for the most part, excepting local practices that accommodate a the specific geology or needs of the region.

Other kinds of penetration tests are employed throughout the U.S., but are unique and localized to certain parts of the country. For example, a variety of dynamic cones and driven penetrometers (Texas Highway-type, Sowers drive cone, Michigan-state, Dinastar), and large split-barrel type samplers (D&M, Converse, Acker) is used that are similar in concept to the SPT. However, the cone/spoon diameters, hammer sizes, and driving forces of these devices are not standardized, except perhaps within a given locality.

For gravelly soils, the Becker penetration test (BPT) has been developed and primarily utilized in western North America. Initially developed for use in assessing pile driveability and design of pile lengths in very coarse deposits, the BPT has also proved useful in evaluating liquefaction potential (e.g., Sy & Campanella 1994).

3. CPT EQUIPMENT

Most serious CPT work in the U.S. is now performed using standard electronic cones having a 60° apex pushed continuously at 20 mm/s, essentially making the mechanical versions obsolete. Initial designs that required the amplification of electric cone outputs, have now been replaced with superior electronics within the cone using signal conditioning to provide better resolution, increased reliability, and minimal noise.

Data acquisition systems typically include a portable computer, analog-digital converter, storage media (hard drive, floppy drives), and strip chart recorder or printer.

3.1 Penetrometers and rigs

Penetrometers having diameters of either 35.7 mm (10 cm^2 projected area) and 43.7 mm (15 cm^2) are used routinely. Because of the superior stratigraphic detailing, much CPT is accomplished with piezocones (designated PCPT or CPTU). As shown in Fig. 1, two basic types of piezocones are used for routine site investigations: (1) one with midface element for pore water pressure measurements (designated u_1 or u_t), and (2) with shoulder or behind the tip position (u_2 or u_{bt}). Earlier versions of the type 1 design placed the element at the cone apex. However, later designs put the element midface because it is less vulnerable to damage and excessive wear.

There are advantages and disadvantages

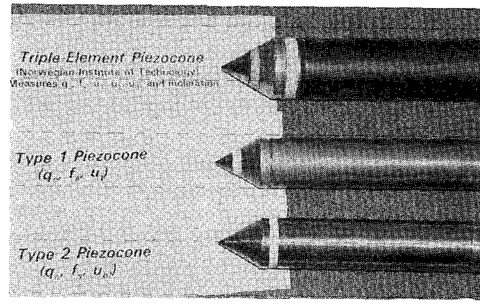

Fig. 1. Type 1 (midface element) and type 2 (shoulder element) piezocones with 10 cm^2 tips in comparison with 15 cm^2 multi-element cone.

associated with either type 1 or 2 cones. Pore water pressure readings from type 2 cones are necessary for the proper correction of measured cone tip resistances ($q_c \rightarrow q_T$), as per Campanella & Robertson (1988) and DeBeer et al. (1988). This correction is very important in soft to medium stiff intact clays, but not significant in medium to dense clean sands or overconsolidated fissured clays where small positive, zero, or even negative Δu_2 readings are obtained (Mayne et al. 1990).

In general, type 2 cones may be more appropriate to the northern regions of the U.S. because of the preponderance of near surface recent geologic deposits comprised of soft to firm lightly overconsolidated soils, including marine deposits of the Atlantic coastal plain (e.g, Boston Blue clay; Calvert clay), glacial lacustrine sediments of the Great Lakes, and alluvial deposits. Maximum detailing is accomplished using type 1 cones, however, and if stratigraphic profiling is paramount, u_1 measurements may be preferred.

In contrast, the hot temperate climate of the southern U.S. has formed overconsolidated and stiff materials by desiccation (e.g., Florida, Lousiana Southern California). If the materials are fissured, little detail is observed with type 2 readings and u_2 measurements can be small or even negative. Thus, a type 1 cone may be of better value in profiling. On the other hand, some difficulties have been noted in smearing and clogging of type 1 face porous elements during penetration of fat plastic clays, e.g. the

Beaumont clays of Texas. In this case, less smearing occurs with a type 2 cone. Of course, exceptions to the above geographic generalities occur, and hard OC Cretaceous clays can also be found in the northern U.S. (e.g., Washington, DC), as well as the presence of very soft deltaic sediments in the south (e.g., offshore Gulf of Mexico). Thus, an ideal scenario for general use would be a dual-element cone for site characterization (Juran & Tumay 1989).

In order to maximize production and efficiency, most regions of the U.S. are now serviced by commercial testing firms and research institutions with specialized cone trucks (e.g., see Fig. 2). Compared with a conventional 10-tonne drill rig, a standard cone truck weighs about 20-tonnes, although special 30- and 40-tonne models have been built that use stronger rods in order to successfully penetrate dense sands or facilitate the completion of soundings with penetration depths of up to 60 m or more.

Fig. 2. Cone operated by Fugro Geosciences; manufactured by Hogentogler & Company.

During CPT, depth increments are measured above ground using either potentiometers, depth wheels, or ultrasonic beams. Successive increments are summed to give the total depth. Although cableless systems are available (Larsson & Mulabdić 1991), almost all U.S. systems employ a cable through the rods to connect the cone to the data acquisition unit at the ground surface.

3.2 Testing & calibration procedures.

Electronic penetrometers require a minimum of two calibration procedures: (1) load cell calibration in a compression apparatus to obtain output voltage for q_c and f_s; and (2) hydrostatic calibration in a triaxial cell for determination of output voltage for u_1 or u_2, as well as the net area ratios for correction of tip (q_T) and sleeve (f_T) resistances. Details on these calibrations have been given elsewhere (e.g., Jamiolkowski et al. 1985; DeBeer et al. 1988). If the cone will be used in very cold or hot climates, a calibration check for temperature variations is also recommended (Lunne et al. 1986).

Calibrations of ancillary devices such as potentiometers for depth measurements and oscilloscopes for seismic velocity arrival times are handled separately.

Porous elements are often made of flexible polypropylene and are disposed of after each sounding. In stiff or dense soils, stainless steel or rigid ceramic elements are better for type 1 cones because of high abrasion and the compressibility of the filter affects the pore pressure readings (Campanella & Robertson 1988).

Proper saturation of the porous element is important for quality results (Lunne et al. 1986). A 50/50 mixture of glycerine and water provides excellent results, although some testing firms use either silicon oil or distilled water. A prophylactic is placed over the saturated cone with a rubber band to maintain saturation until penetration.

3.3 Corrections & data presentation

Type 1 cones have a non-standard position of the pore pressure element, as shown in Fig. 3, thus giving different recorded u_1 for each particular cone (Brown 1993). For stratigraphic profiling, this is unimportant since only relative variations with depth are compared. For the assessment of soil properties, however, these differences in u_1 affect the numerical outcome and interpretation and therefore should be taken into account.

Measured data from cone soundings are usually presented graphically (and/or digitally)

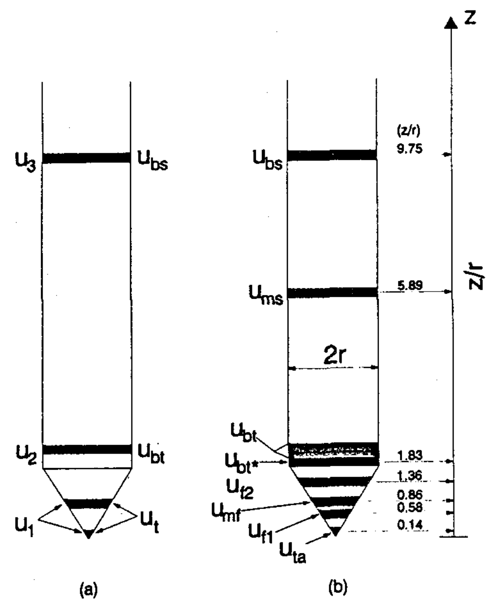

Fig. 3. Porous element positions: (a) general; and (b) specific or "rainbow" cone (Brown 1993).

in terms of the individual readings versus depth, including: cone tip resistance (q_c or q_T), sleeve friction (f_s or f_T), and penetration pore pressure (u_1 or u_2 or excess Δu). Sometimes, an additional profile of dynamic pore pressure, u/q_c, or the normalized piezocone parameter, $B_q = \Delta u/(q_T-\sigma_{vo})$, is presented (Wroth 1984).

Figure 4 illustrates an example profile summary of q_T, f_s, and u from single-, dual-, and triple-element soundings in overconsolidated desiccated clay in Baton Rouge, Louisiana. Note the u_3 position is located behind the friction sleeve. Excellent repeatability of results is evident for the first three channels, while differences are observed for the two trailing pore pressure locations, thus inferring difficulties in maintaining saturation.

If piezocone dissipation tests at specific test depths are made, the results are presented with either penetration pore pressure (u) or excess Δu versus time or logarithm of time (e.g., Levadoux & Baligh 1986).

For seismic cone soundings, a downhole assessment of the time arrivals of the compression (P) and shear (S) waves can be obtained, thus producing profiles of the interpreted P- and S-wave velocities vs. depth (Campanella 1994). Usually, these data are presented at discrete increments corresponding to rod changes (approx. each meter). Alternatively, the complete wave trace record with time (forward and/or reverse) can be shown for each event.

If a conductivity cone is used (Campanella & Weemes 1990), a continuous profile of electrical resistivity is presented and used to infer the presence of subsurface contaminants.

3.4 National standards

Standard procedures for CPT have been established by ASTM D-3441 since 1975 that addressed mechanical and electrical friction cones. For the piezocone test, a revised ASTM D-3441 procedure has been proposed in which a type 2 porous element position is recommended (Farrar 1995).

4. INTERPRETATION

The results of CPT and PCPT are used for delineating soil strata and for evaluating the geotechnical engineering parameters of the subsurface layers. In recent environmental applications, cone data are used to infer or detect the presence of anomalies such as contaminants in the pore fluid.

4.1 Soil classification and stratigraphy

Piezocone results are unsurpassed in the detailing of soil stratigraphy. Exceptional resolution makes the detection of thin seams and lenses possible, particularly via the pore pressure channel. This facet is very important from a geoenvironmental viewpoint and for slope stability evaluations.

Soil classification using CPT and PCPT data is indirect and relies entirely on empirical charts for interpretation of strata (see Table 1).

4.2. Engineering parameters

Considerable effort has been made to derive soil engineering properties from the results of cone and piezocone data. Methodologies have

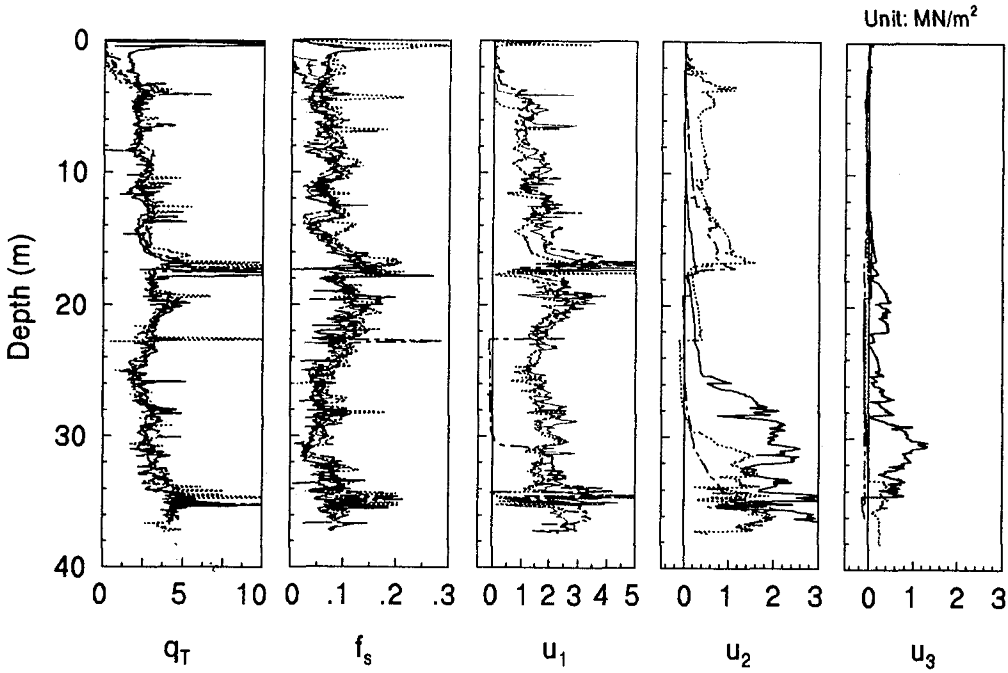

Fig. 4. Summary profiles of piezocone soundings in desiccated clay at Baton Rouge, LA (Chen 1994).

been developed using empirical and statistical methods, backcalculation, analytical studies, and numerical simulation.

In the organization of Table 1, interpretation methods have been grouped into one of three basic categories: those specifically addressing (a) clays and cohesive materials, (b) sands and cohesionless materials, and (c) applicable to both soil types. Abbreviated references are given in the table to conserve space.

4.3 Environmental data

Many recent developments in CPT (or direct-push technology) have centered around its use for geoenvironmental concerns. The incorporation of additional sensors within the penetrometer to instantaneously and continuously monitor phenomena offers significant potential for evaluating subsurface chemical and biological conditions.

Some success has been obtained with sensors that quantify bulk resistivity of the pore fluid (Campanella & Weemees 1990) or electrical conductivity (Woeller et al. 1991a, 1991b), temperature (MacFarlane et al. 1983), pH and redox (Olie et al. 1992), light hydrocarbons (Malone et al. 1992), neutron moisture (Shibata et al. 1992), and petroleum vapors (Horsnell 1988). Most of these methods involve contaminant mapping by inference, however, and no direct chemical assessment is made. A review of some additional sensors and technologies under development is given by Bowders & Daniel (1994).

In addition to modifications and add-ons to the standard cone, direct-push technology has development has led to other specialized probes for sampling and testing groundwater and soil during environmental site characterization, including: push-in soil samplers, push-in water samplers, push-in piezometers, and soil-gas samplers. One example of these specialized devices is the "hydro-trap", a commercial groundwater sampler used to obtain volatile

Table 1. List of soil parameters interpreted from cone and piezocone data.

ALL SOIL TYPES *(Clays, Silts, and Sands)*

- **Soil Classification**
  - Begemann (1965), 6th ICSMFE (1), Montreal, 17-20.
  - Schmertmann (1978), Report TS-78-209, FHWA, Washington, DC, 145 p.
  - Douglas & Olsen (1981), Cone Testing & Experience, ASCE, New York, 209-227.
  - Jones & Rust (1982), ESOPT (2), Amsterdam, 607-613.
  - Vlasblom (1985), Report No. 92, Laboratorium voor Grondmechanica, Delft, 51 p.
  - Senneset & Janbu (1985), STP 883, ASTM, Philadelphia, 41-54.
  - Olsen & Farr (1986), ASCE GSP 6, Blacksburg, 854-868.
  - Robertson (1990), CGJ 27 (1), 151-158 and CGJ 28 (1), 173-178.
  - Cheng-hou et al. (1990), Engineering Geology 29 (1), 31-47.
  - Jefferies & Davies (1993), ASTM GTJ 16 (4), 458-468.
- **Effective Friction Angle (ϕ')**
  - Senneset & Janbu (1985), STP 883, ASTM, Philadelphia, 41-54.
  - Sandven (1990), PhD Thesis, Norwegian Inst. of Tech (NTH 1990.3), Trondheim.
  - Masood & Mitchell (1993), ASCE JGE 119 (10), 1624-1639.
- **Effective Cohesion Intercept (c')**
  - Senneset et al. (1989), Transporation Research Record 1235, 24-37.
- **Hydraulic Conductivity (k)**
  - Schmertmann (1978), Report TS-78-209, FHWA, Washington, DC, 145 p.
  - Parez & Fauriel (1988, Revue de Française de Géotechique 44, 13-27.
- **Constrained Modulus ($M = 1/m_v$)**
  - Mitchell & Gardner (1975), ASCE In-Situ Measurement (II), Raleigh, 279-345.
  - Senneset et al. (1982), ESOPT (2), Amsterdam, 863-870.
- **Shear Wave Velocity (V_s)**
  - Hegazy & Mayne (1995), Paper A87, CPT'95, Linköping.

CLAYS

- **Undrained Strength (s_u)**
  - Jamiolkowski et al. (1982), ESOPT (2), Amsterdam, 599-606.
  - Robertson & Campanella (1983), CGJ 20 (4), 734-745.
  - Lunne et al. (1985), 11th ICSMFE (2), San Francisco, 907-912.
  - Keaveny & Mitchell (1986), ASCE GSP 6, Blacksburg, 668-685.
  - Aas et al. (1986), ASCE GSP 6, Blacksburg, 1-30.
  - Konrad & Law (1987), CGJ 24 (3), 392-405.
  - Marsland & Powell (1988), PTUK, Birmingham, 209-214.
  - Jamiolkowski et al. (1988), ISOPT-1 (1), Orlando, 263-296.
  - Rad & Lunne (1988), ISOPT-1 (2), Orlando, 911-917.
  - Stark & Juhrend (1989), 12th ICSFME (2), Rio, 327-330.
  - Houlsby & Wroth (1989), 12th ICSFME (1), Rio, 227-232.
  - Stark & Delashaw (1990), Transportation Research Record 1278, 96-102.
  - Chen & Mayne (1993), ICCHGE (II), St. Louis, 1305-1312.
- **Stress History (σ_p' or OCR)**
  - Tavenas & Leroueil (1979), 7th ECSMFE (1), Brighton, 281-291.
  - Tumay et al. (1982, ESOPT (2), Amsterdam, 915-921.
  - Wroth (1984), Geotechnique 34 (4), 449-489.
  - Jamiolkowski et al. (1985), 11th ISCMFE (1), San Francisco, 57-154.
  - Battaglio et al. (1986), 4th SE Asian Geotechnical Seminar, Singapore, 129-143.
  - Konrad & Law (1987), Geotechnique 37 (2), 177-190.
  - Mayne & Bachus (1988), ISOPT (2), Orlando, 857-864.
  - Sills et al. (1988), PTUK, Birmingham, 247-250.
  - Sully et al. (1988), ASCE JGE 114 (2), 209-215.
  - Sandven (1990), PhD Thesis, Norwegian Inst. of Tech (NTH 1990.3), Trondheim.

Table 1. (cont.)

Mayne (1991), Soils & Foundations 31 (2), 65-76; 32 (4), 190-192.
Mayne & Chen (1994), 13th ICSMFE (1), New Delhi, 283-286.

- **Effective Stress Friction (ϕ')**

Senneset & Janbu (1985), ASTM STP 883, Philadelphia, 41-54.
Lunne et al. (1985), 11th ICSMFE (2), San Francisco, 907-912.
Keaveny & Mitchell (1986), ASCE GSP 6, Blacksburg, 668-685.
Sandven et al. (1988), ISOPT-1 (2), Orlando, 939-953.
Senneset et al. (1989), Transportation Research Record 1235, 24-37.

- **In-Situ Stress State (K_o)**

Mayne & Kulhawy (1990), Transportation Research Record 1278, 141-149.
Sully & Campanella (1991), ASCE JGE 117 (7), 1082-1088.
Masood & Mitchell (1993), ASCE JGE 119 (10), 1624-1639.

- **Coefficient of Consolidation (c_h)**; from dissipation tests:

Hansbo et al. (1981), Geotechnique 31 (1), 45-66.
Battaglio et al. (1981), Cone Testing & Experience, ASCE, New York, 264-302.
Tumay & Acar (1985), ASTM STP 883, Philadelphia, 72-82.
Jamiolkowski et al. (1985), 11th ICSMFE, San Francisco (1), 54-157.
Levadoux & Baligh (1986), ASCE JGE 112 (7), 707-745.
Gupta & Davidson (1986), Soils & Foundations 26 (3), 12-22.
Robertson et al. (1988), CGJ 25 (1), 56-61.
Houlsby & Teh (1988), ISOPT-1 (2), Orlando, 777-784.
Kabir & Lutenegger (1990), CGJ 27 (1), 58-67.
Teh & Houlsby (1991), Geotechnique 41 (1), 17-31.
Robertson et al. (1992), CGJ 29 (3), 539-550.
Sully & Campanella (1994), 13th ICSMFE (1), New Delhi, 201-204.
Burns & Mayne (1995), Paper A33, CPT'95, Linköping.

- **Hydraulic Conductivity (k)**

Tavenas et al. (1982), ESOPT (2), Amsterdam, 889-894.
Robertson et al. (1992), CGJ 29 (4), 539-550.
Elsworth (1993), ASCE JGE 119 (10), 1601-1623.
Manassero (1994), ASCE JGE 120 (10), 1724-1746.

- **Constrained Modulus ($D = 1/m_v$)**

Robertson & Campanella (1983), CGJ 20 (4), 734-745.
Sandven et al. (1988), ISOPT-1 (2), Orlando, 939-953.
Kulhawy & Mayne (1990), Report EL-6800, EPRI, Palo Alto, 306 p.

- **Shear Wave Velocity (V_s)**

Mayne & Rix (1995), Soils & Foundations 35 (2).

- **Low-Strain Shear Modulus (G_{max})**

Bouckovalas et al. (1989), 12th ICSMFE (1), Rio, 191-194.
Mayne & Rix (1993), ASTM GTJ 16 (1), 54-60.

- **Sensitivity (S_t)**

Robertson & Campanella (1983), CGJ 20 (4), 734-745.

- **Unit Weight (γ_T)**

Larsson & Mulabdic (1993), Rept 42, Swedish Geotechnical Inst., Linköping, 240 p.

- **Rigidity Index (I_r)**

Chen and Mayne (1994), Report CEEGEO-94-1, Georgia Tech, Atlanta, 280 p.

SANDS

- **Effective Friction Angle (ϕ')**

Trofimenkov (1974), ESOPT-1 (1), Stockholm, 147-154.
Mitchell & Lunne (1978), ASCE JGE 104 (7), 995-1012.
Robertson & Campanella (1983), CGJ 20 (4), 734-745.
Lunne & Christophersen (1983), 15th Offshore Tech. Conf. (1), Houston, 181-192.
Mitchell & Keaveny (1986), ASCE GSP 6, Blacksburg, 823-839.

Table 1. (cont.)

Jamiolkowski et al. (1988), ISOPT-1 (1), Orlando, 263-296.
Kulhawy & Mayne (1990, Report EL-6800, EPRI, Palo Alto, 306 p.

- **Relative Density (D_r)**
  Schmertmann (1978), Report TS-78-209, FHWA, Washington, DC, 145 p.
  Lunne & Christophersen (1983), 15th Offshore Technol. Conf. (1), Houston, 181-192.
  Jamiolkowski et al. (1985), 11th ISCMFE (4), San Francisco, 1891-1896.
  Kulhawy & Mayne (1991), ISOCCT-1, Clarkson University, 197-211.
- **State Parameter (ψ)**
  Been et al. (1986), Geotechnique 36 (2), 239-249.
  Been et al. (1987), Geotechnique 37 (3), 285-299.
  Jamiolkowski & Robertson (1988), PTUK, Birmingham, 321-342.
- **Stress State (K_0)**
  Manassero (1991), ISOCCT-1, Clarkson University, 239-248.
  Mayne (1991), ISOCCT-1, Clarkson University, 249-256.
  Masood & Mitchell (1993), ASCE JGE 119 (10), 1624-1639.
- **Constrained Modulus ($M = 1/m_v$)**
  Robertson & Campanella (1983), CGJ 20 (4), 734-745.
  Jamiolkowski et al. (1988), ISOPT-1 (1), Orlando, 263-296.
  Kulhawy and Mayne (1990), Report EL-6800, EPRI, Palo Alto, 306 p.
- **Low-Strain Shear Modulus (G_{max})**
  Baldi et al. (1989), ISOPT-1 (2), Orlando, 643-650.
  Rix & Stokoe (1991), ISOCCT-1, Clarkson University, 351-362.
  Lunne et al. (1994), 12th ICSMFE (4), Rio, 2339-2403.
  DeAlba et al. (1994), 13th ICSMFE (1), New Delhi, 173-176.
  Olsen (1994), Rept. TR-GL-9429, Waterways Experiment Station, Vicksburg, 322 p.
- **Shear Wave Velocity (V_s)**
  Baldi et al. (1989), 12th ICSMFE (1), Rio, 165-170.
- **Overconsolidation Ratio (OCR)**
  Mayne (1991), ISOCCT-1, Clarkson University, 249-256.
- **Liquefaction Potential**
  Jamiolkowski et al. (1985), 11th ICSMFE (4), San Francisco, 1891-1896.
  Robertson & Campanella (1985), ASCE JGE 111 (3), 394-403.
  Seed & DeAlba (1986), ASCE GSP 6, Blacksburg, 281-302.
  Shibata & Teparaksa (1988), Soils & Foundations 28 (2), 49-60.
  Sugawara (1989), 12th ICSMFE (1), Rio, 233-238.
  Stark & Olson (1995), ASCE JGE 121 (12).

NOTES:

ASCE	= American Society of Civil Engineers, New York.
ASTM	= American Society for Testing and Materials, Philadelphia.
CGJ	= Canadian Geotechnical Journal, National Research Council/Canada, Ottawa.
ECSMFE	= European Conference on Soil Mechanics & Foundation Engineering.
EPRI	= *Electric Power Research Institute, Palo Alto, CA.*
ESOPT	= European Symposium on Penetration Testing.
FHWA	= Federal Highway Administration (US DOT), Washington, DC.
GSP	= Geotechnical Special Publication (ASCE).
GTJ	= Geotechnical Testing Journal, ASTM, Philadelphia.
ICCHGE	= International Conference on Case Histories in Geotech. Engineering.
ICSMFE	= International Conference on Soil Mechanics & Foundation Engineering.
ISOPT	= International Symposium on Penetration Testing, Orlando, FL.
ISOCCT	= International Symposium on Calibration Chamber Testing, Potsdam, NY.
JGE	= Journal of Geotechnical Engineering, ASCE, New York.
PTUK	= Penetration Testing in the UK, Thomas Telford, London.
STP	= Special Technical Publication (ASTM).

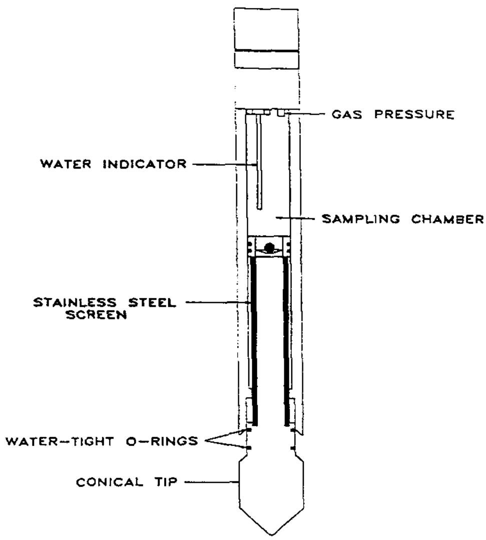

Fig. 5. Push-in hydro trap for obtaining groundwater samples (Yilmaz 1995).

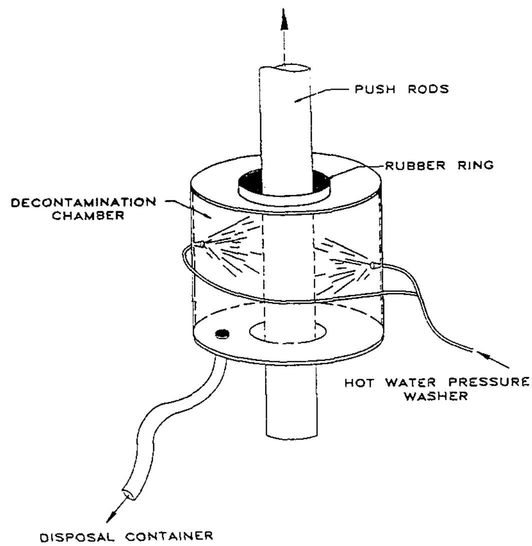

Fig. 6. Environmental CPT rod washing system (Yilmaz 1995).

organic compounds under controlled confining pressures, as shown in Figure 5.

For geoenvironmental CPT explorations, additional measures must be undertaken to decontaminate the push rods during extraction from the ground. This is necessary for the safety and health of the crew and to reduce potential contamination of the subsequent test location. Fig. 6 illustrates one commercial concept used for rod washing.

Of equal concern is the potential for cross-contamination of aquifers and groundwater reserves by penetration. Many states (e.g., LA, NJ, CA) require that exploratory holes be grouted upon completion. Thus, self-grouting systems or companion grouting units for hole closure have been developed (Yilmaz 1995).

5. CPT IN FOUNDATION DESIGN

Routinely, CPT data are used for the analysis and design of foundations, including bearing capacity and settlement of spread footings, driven piles, and drilled shafts (bored piles). Both direct and indirect methods of CPT assessment are used, as discussed in the following sections.

In a recent ASCE/FHWA-sponsored prediction symposium involving large footings on sand (Briaud & Gibbens 1994), the CPT proved to be the most preferred test (used by 30% of predictors) for assessing the foundation performance, as compared with 25% for SPT, 16% for PMT, 14% for DMT, and 10% for triaxial tests.

The CPT is also useful in assessing compaction control during placement of structural fills and in the evaluation of effectiveness of ground modification techniques (e.g., vibroflotation, dynamic compaction) and site improvement works (Mitchell 1986).

5.1 Direct methods

In these approaches, the measured CPT data are directly input into empirical formulas to provide estimates of foundation capacity and settlement (e.g., Schmertmann 1978). For example, regarding the prediction of axial capacity of deep foundations, there are at least 6 methods for driven piles (Robertson et al. 1988) and 5 for drilled shafts or bored pile systems (Alsammam 1995). Poulos (1989) provides a review for both types.

5.2 Indirect methods

In these methods, the CPT data are used to estimate soil properties that are input into a theoretical model for predicting capacity or deformation response. General procedures for interpreting engineering parameters for foundation analysis from in-situ data are given by Kulhawy & Mayne (1990). Berardi et al. (1991) outline settlement analysis procedures for spread footings on sand from CPT data. CPT-based methods for calculating axial capacity and settlement of driven piles are discussed by Robertson et al. (1988) and Poulos (1994), respectively. Van Impe (1994) extensively covers drilled and bored pile analysis from CPT.

6. COMPARISONS & CORRELATIONS OF CPT WITH OTHER METHODS

Comparisons of cone measurements can be made within the test (intra-correlative) or between other in-situ tests that are conducted in the field adjacent to the CPT location (inter-correlative).

6.1 Intra-correlative studies

Internal relationships among paired sets of q_c and f_s readings with physically-retrieved soil samples form the original basis for empirical soil classification charts. In sands, intra-correlative trends for f_s vs. q_c are discussed by Parkin (1988). For clays, Mayne et al. (1990) present u vs. q_T trends for face- and shoulder-type PCPTs. More specifically, one set of intra-correlations for uncemented intact clays is shown in Fig. 7. However, the relationships also depend on OCR and degree of fissuring (Powell et al. 1988).

Initially, q_c vs. f_s plots were used to infer soil type (e.g., Schmertmann 1978). Later, Senneset et al. (1989) developed a soil classification scheme based on q_T vs. B_q, while Robertson (1990) suggested a system that utilizes all three readings of the PCPT. However, the normalization scheme for q_T and f_T (and u) should actually depend upon soil type (Olsen 1994). That is, $Q = (q_T-\sigma_{vo})/\sigma_{vo}'$ is appropriate for clays, while for clean sands,

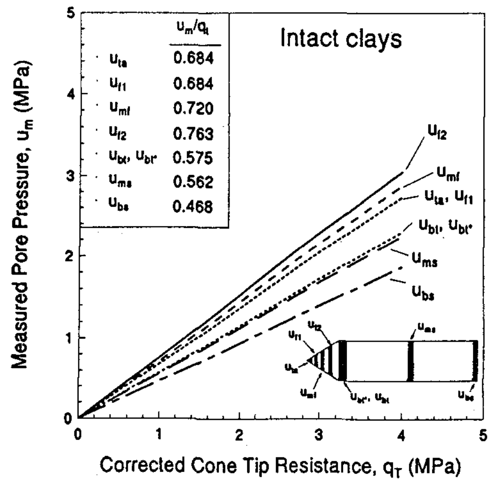

Fig. 7. Summary interrelationships between penetration pore pressures and tip resistance for unstructured intact clays (after Brown, 1993).

the parameter $Q^* = (q_T-\sigma_{vo})/(\sigma_{vo}')^{0.5}$ may be more appropriate.

6.2 Extra-correlative studies

Relationships between the SPT-N value and CPT-q_c have been studied for a variety of soil types (Schmertmann 1978; Robertson et al. 1983; Mullen, 1991). The ratio of q_c/N generally increases with mean grain size and averages (Kulhawy & Mayne 1990):

$$q_c/N_{60} \approx 544\ (D_{50})^{0.26} \qquad (1)$$

where q_c is in kPa, N_{60} = energy corrected SPT resistance, and D_{50} is in mm. However, there is considerable variation around this average. The ratio q_c/N_{60} also depends on the percent fines (particles < 75μ), as well as other factors.

Correlations between CPT and DMT have also been investigated. For clays (intact and fissured), the DMT contact pressures (p_o) are comparable to type 1 PCPT penetration pore pressures (Mayne & Bachus 1989), such that:

$$\text{Clays: } u_1 \approx p_o \qquad (2)$$

In clean sands, measurement of the DMT blade thrust provides a wedge resistance (q_D) that is comparable in magnitude to q_c. For McDonald Farm sand, Campanella & Robertson (1991) found that:

$$\textbf{Sands:}\quad q_D/q_c \approx 1.1 \qquad (3)$$

which was later confirmed appropriate for Toyoura sand in calibration chamber studies (Bellotti et al. 1994).

6.3 National Test Sites (NGES)

Since 1988, a combined effort by the Federal Highway Administration (FHWA) and National Science Foundation (NSF) resulted in the cataloging and establishment of several National Geotechnical Experimentation Sites (NGES) throughout the U.S. (Woods, 1994).

Currently, 40 designated sites have been cited throughout the 48 contiguous states and Alaska, with the majority classified as Level 3 (unfunded) sites, as shown by Fig. 8. Two high-priority (Level 1) and three medium-priority (Level 2) sites receive annual funding for databasing, management, and site improvements. The NGES permit the opportunity to compare cone and piezocone data with results derived from other in-situ tests and/or laboratory devices.

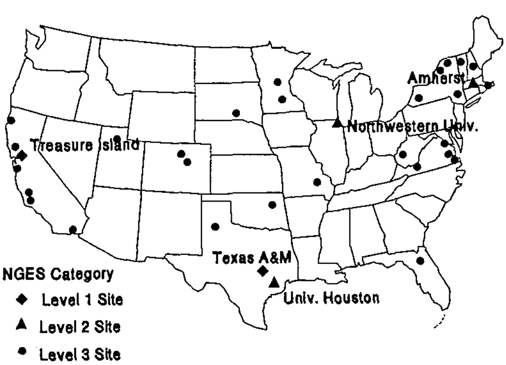

Figure 8. NGES Sites in U.S.A.

7. MAJOR RESEARCH ACTIVITIES

Academic research on CPT has been mainly directed at improved means of interpretation, including analytical studies based on limit plasticity (Masood 1990) or cavity expansion (Chen 1994 for clays; Salgado 1993 for sands). In addition, numerical simulation of the penetration problem has been attempted using dislocation-based models (Elsworth 1991), strain paths (Whittle & Aubeny, 1993), and discrete elements (Huang & Ma, 1994).

Empirical and theoretical methods for interpreting CPT data have been derived from analyses of available field data (Birgisson 1991; Brown 1993; Olsen 1994) or from controlled laboratory calibration chamber testing (Kulhawy & Mayne, 1990). Recent programs providing additional reference data include field studies by Mullen (1991) and Murray (1995) and chamber tests for sands (Puppala et al. 1991; Rix & Stokoe 1991; Salgado 1993), silty sands (Brandon & Clough, 1991), and clays (Kurup et al. 1994).

Research by cone manufacturers and commercial testing firms has focussed on improvements in equipment performance and reliability (also see future trends). The effects of temperature variations, electromagnetic noise, and zero drift have essentially been eliminated with the use of internal signal conditioning and microelectronics.

Many innovative ideas have resulted in the increased applicability of the cone to difficult soil conditions and to geoenvironmental problems (Mitchell 1988). Bratton et al. (1995) and Auxt & Gilkerson (1995) provide recent reviews of the various types of geoenvironmental penetrometers, their capabilities, and shortcomings.

8. FUTURE TRENDS

Innovative new developments and the incorporation of additional capabilities to the CPT are currently underway (Bowders & Daniel 1994). A few interesting ongoing projects are discussed briefly in the following paragraphs.

The USAE Waterways Experiment Station has developed an elaborate design (SCAPS = site characterization and analysis penetrometer system) that includes a fiber-optics guide from a sapphire window located near the cone tip to measure the laser-induced fluorescence (LIF) of petroleum chemicals which may be present (Schroeder et al. 1991; Stark, 1991). A similar system using a fiber optic chemical sensor (FOCS) and photo-ionization detector was developed (Leonard & Tillman 1993), but has been discontinued because the sensor is not reversible. An LIF probe utilizing a tuneable laser fluorescence system (ROST system) is now in use for the identification of aromatic hydrocarbons (Yilmaz 1995).

A miniature cone unit for expedient CPTs has been designed with a caterpillar thrust system and self-coiling rods that mount at the front of a 4-wheel drive diesel pickup truck (Tumay 1994).

A new subsurface vision probe has been proposed for direct optical soil classification by CPT. The development will use a sapphire viewing window, high-resolution video camera, and image processing center to determine grain size distributions in real time (Hryciw & Raschke 1996).

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CONE PENETRATION TESTING IN VIETNAM.

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Abstract.

In Vietnam, cone penetration testing has been used extensively during the last 15 years for the soil investigation of cohesive soils and loose granular soils as well as for the solution of practical problems such as design of earth structures, deep and shallow foundations. The report will summarize the state of practice and Vietnamese experiences on the Equipment and Testing procedure, interpretation of test results and solution of foundation problems. Correlation between the test results of CPT and other methods, geotechnical parameters, pile driving analysis, vibration due to pile driving, pile capacity and other Geotechnical problems are presented.

INTRODUCTION.

The cone penetration testing (CPT) is a popular soil investigation method in Vietnam today due to our soil condition which is consisted of extensive deposit of soft clay and together with different layers of sand, silt and clay. CPT is considered as the most effective and valuable method for interpretation of soil types and soil properties. The method can be used for determination of the geological profiles, evaluation of the bearing capacity of the pile, underained shear strength and Young modules. The state of practice for cone penetration testing in Vietnam is presented in this report.

1. SOIL CONDITION OF VIETNAM

Vietnam has a biggest cities and provinces that are situated in the Red river and Mekong river deltas. In these regions the soil deposits are quartenary formation, formed by the simultaneous activity of sea and rivers. Based on the origin, age and material composition, can be divided in to following formations :

- Modern alluvial marine formation, which is formed in the quatenary era.
- Modern alluvial deposits, which is formed by the sedimentary activity of rivers.
- Middle Holocene deposits, which is formed during the sea progress. Its distribution is complicated, the thickness is varied from 3 to 25m. This deposit consists of different soil layers such as clay, organic, peat, silt and sand. The soil is often very soft clay, its consistency is liquid and is of loose structure.
- Pleistocene marine formation is resulted from the sea progress into the river and formed stiff layers in the area.

2. TYPES OF PENETRATION TESTING AND OTHER FIELD METHODS ARE USED IN VIETNAM.

2.1 Standard Penetration test (SPT).

The SPT was introduced in Vietnam long time ago and today is popular method for soil investigation, SPT can be done in stiff soil such as sand, gravel and stiff clay. In many cases, SPT can be used to increase the depth for soil investigation and can be used for soil classification and evaluation of pile capacity.

2.2 Weight Sounding

The Swedish weight sounding is used in Vietnam about 15 years ago, the equipment is simple and it is possible to obtain the value of relative density of cohesionless soil and pile capacity. The equipment is cheap, easily manufactured, it is suitable to develop in Vietnam.

3. CPT EQUIPMENT USED IN VIETNAM.

Different static penetrometer equipment are used in Vietnam such as GEOTECH, GOUDA, ADINA and PVS... The test procedure and calibration are follow the Swedish experiences and recommendation due to the fact that the CPT was introduced to Vietnam by the Swedish Geotechnical Institute and this Institute have been making a lot of contribution to development of Geotechnical Engineering in Vietnam. The presentation of results is according to international practice, the variations of point resistance q_c and friction f_s with depth are recorded during the test or from reading values.

The standard for CPT was publicated in 1985 in Vietnam, equipment , test procedure, correction, presentation of test results, interpretation of test results and use of CPT in geotechnical design are included.

4. INTERPRETATION OF TEST RESULTS.

The typical results from CPT test in different places of Vietnam are presented in Fig.1 and Fig.2 for Hanoi City, and Hai Hung province. The CPT test were carried out by PVS and GOUDA.

Cone penetration test is widely used in Vietnam for soil classification, stratigraphy, layers of sand, clay, silty clay can be determined.

The soil parameters can be obtained from the interpretation of CPT.

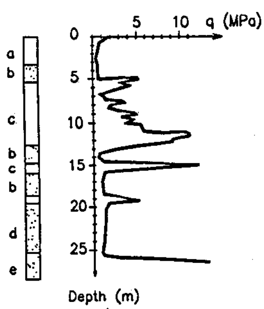

Fig.1 Typical results of CPT test (Hanoi City)
a.Clay b.Mud
c.Fine sand d.Soft clay
e.Gravel

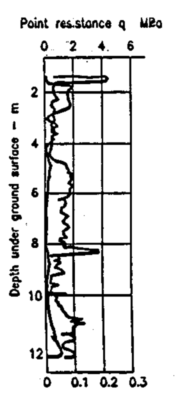

Fig.2 Typical result of static sounding at Hoang Thach (Hai Hung province)

4.1 Soil classification and stratigraphy.

The soil classification can be defined using q_c. Typical soils of Bac bo delta and q_c are shown in Table 1, according to D.T.Tuong et al (1992)

Table 1 : q_c for Bac bo delta soils

Soils	q_c (MPa)
Holocene alluvial clay soil	1.0 - 1.5
Pleistocene marine clay soil	2.5 - 3.5
Holocene soft organic soil	0.2 - 0.5
Holocene alluvial sand	4.0 - 6.0
Pleitocene alluvial marine sand	5.0 - 10.0

4.2 Soil parameters.

4.2.1 Undrained shear strength.

The undrained shear strength of cohesive soils can be expressed as

$$c_u = \frac{q_c}{N_c} \quad (1)$$

According to of B. D. Nhuan et al (1985) the correlation between c_u and q_c for soft silty clays is plotted in Fig.3 . The mean value of N_c is 20.

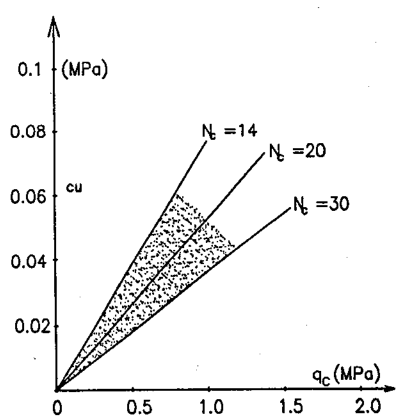

Fig.3 correclation between vane shear streng and cone resistance on shoft clays

4.2.2 Evaluation of deformation modules

The compressibility of soil is usually determined by odometers tests, e.g. E_o the deformation modules. By comparison between E_o and the cone resistance q_c, the following empirical equation was proposed by B.D.Nhuan et al (1985):

$$E_o = 3.44\, q_c^{\,0.78} \text{ (MPa)} \quad (2)$$

The values of q_c and E_o are shown in Table 2.

Table 2 : q_c and E_o

q_c (MPa)	E_o (MPa)
< 0.5	1.0 - 2.0
0.5 - 1.0	2.0 - 3.0
1.0 - 1.5	3.0 - 4.5
1.5 - 2.0	4.5 - 6.0
> 2.0	> 6.0

4.3 Evaluation of the maximum value of particle velocity.

The ground particle velocity V due to pile driving has been studied by many researchers. Based on the results of vibration measurements on ground surface in conection with pile driving and point resistance q_c of CPT, the following empirical correlation was proposed by T.D. Ngoc (1992).

$$V = 0.16\, q_c^{\,0.4}\, E / R_s \qquad (3)$$

Where :

V = Maximum value of ground particle velocity (mm / s)

q_c = Cone penetration resistance, kg / cm^2

E : Young ' s modules (N / cm^2)

R_s = is the distance from tip of pile to the measurement point, $R_s^2 = H^2 + R^2$

H : depth of embedment of pile, m

R : Radial distance from the pile to the point of measurement, m.

5. USE OF CPT IN GEOTECHNICAL DESIGNS.

5.1 Allowable bearing capacity for shallow footings

The allowable bearing capacity is considered as contact pressure, which does not develop a plastic zone below a depth corresponding to 1/4 of the width of footing.For typical soil condition and typical footing : width = 1.0 - 1.5m, depth of foundation = 1.0 - 2.5m. The allowable bearing capacity q_{all} of shallow footing can be evaluated from static cone penetration results :

for sand
$$\frac{q_c}{40} < q_{all} < \frac{q_c}{20} \qquad (4)$$

for cohesive soil
$$\frac{q_c}{20} < q_{all} < \frac{q_c}{10} \qquad (5)$$

In practice B.D.Nhuan et al (1985) propose to use the values of allowable bearing capacity as show in Table 3.

Table 3 : q_c and allowable bearing capacity

q_c (MPa)	q_{all} (MPa)
< 0.5	< 0.5
0.5 - 1.0	0.05 - 0.08
1.0 - 1.5	0.08 - 0.12
1.5 - 2.0	0.12 - 0.16
2.0 - 2.5	0.16 - 0.20
2.5 - 3.0	0.20 - 0.24
> 3.0	> 0.24

5.2 Pile Capacity.

The comparison between the results from static load test on pile and predicted values of pile capacity using the results from CPT and weigh sounding have been made by several researchers in Vietnam according N.T.Tien (1991,1992,1995). The following correlations have established and used in practice:

$$c_u = \frac{q_c}{15 - 20} \qquad (6)$$

$$c_u = \frac{M_w}{3 - 4} \qquad (7)$$

Where :

q_c : point resistance from CPT (MPa)

M_w : Number of half turn for 20cm of penetration of weight sounding in MPa.

c_u : Undrained shear strength of a cohesive soil.

For sand the following correlation between the skin friction f_s and q_c is often used :

$f_s = 0.01\ q_c$ for $q_c \le 2.5$ MPa

$f_s = 0.005\ q_c$ for $q_c \approx 10.0$ MPa

$f_s = 0.004\ q_c$ for $q_c \approx 20.0$ MPa

The value of unit point resistance q_p can be evaluated by

$$q_p = 0.5\ q_c.$$

6. CASE RECORDS

6.1 Swedish Children Hospital, Hanoi

A typical result of static sounding test is shown in Fig.4.1. The R.C piles with a pentagonal cross section are used for underpinning of the buildings. The cross section area of the pile is 235 cm^2 and the perimeter is of 58.5 cm

Two piles were jacked to the depth of 18m and loaded in order to determine their ultimate bearing capacity. The predicted pile capacity from CPT results is about 400 kN.

Fig. 4.2 show the results of static load test.

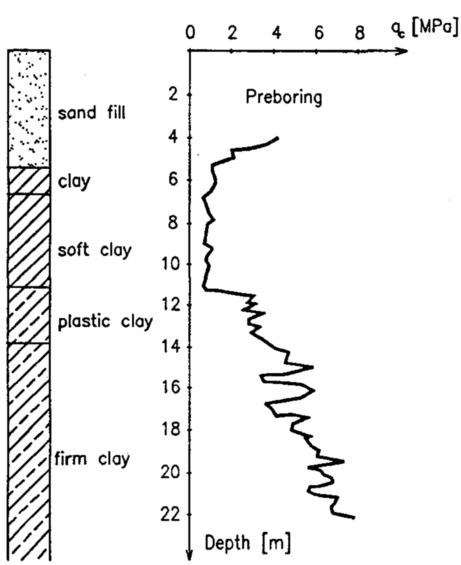

Fig.4.1 Rerult of cone penetration test at the Swedish Children Hospital (Hanoi)

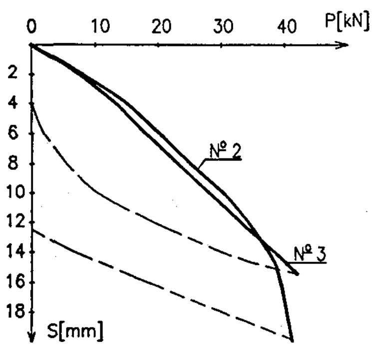

Fig.4.2 Load vs settlement curves of test piles

6.3 Building at 38 Nguyen Hue, Ho Chi Minh City.
The construction site is situated in a densely populated area of the city. Typical cone penetration is shown in Fig.5.1. The driven R.C pile, which have 25x25cm of cross section and 25m of length is used for this project.
The prediction of pile capacity from CPT is :
Q= 1.17 MN, Q_s = 0.8 MN and Q_p = 0.37 MN.

Results from static load test of 2 piles are shown in Fig.5.2

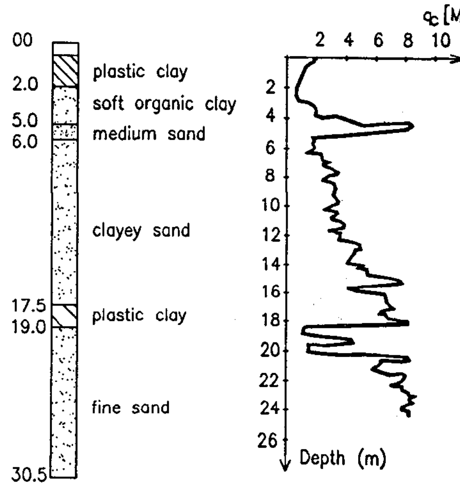

Fig.5.1 Result of static penetration test, Ho Chi Minh City

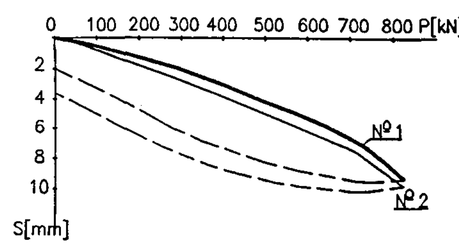

Fig.5.2 Load vs settlement curves of test piles

6.4 La Thanh Hotel, Hanoi.
The results from static cone penetration and jacked forces are shown in Fig.6 according to P.D.Long et al (1987). The concrete pile 20 x 20 cm is jacked down by hydraulic jack. Interesting correlation between jacked forces and q_c can be observed.

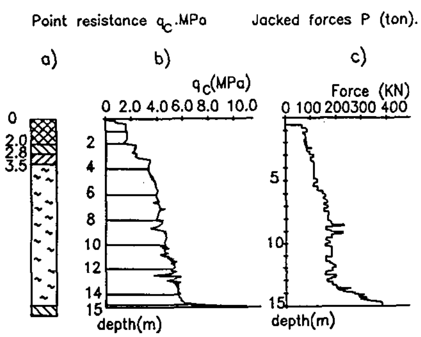

Fig.6 Soil condition at the position of jacked pile and diagram of jacked force
a. Soil profile
b. Static penetration results
c. Jacked forces

7. Conclusions and suggestions

- The cone penetration test is valuable method for soil investigation in Vietnam
- Soil classification and soil parameters can be defined and evaluated from the results of CPT.
- Results from CPT can be used for design of shallow and depth footings.
- Correlations of results from CPT with other investigation methods should be established for practical purposes.
- It is recommended to use CPT for other data and environmental data in Vietnam.

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The objective of the SGF is to promote development in geotechnics and foundation engineering through lectures, discussions and committee work, and to cooperate with Swedish, Nordic and other international bodies having a similar orientation.

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