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CPTU and DMT in soft clays and organic soils

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Edited by

Zbigniew Młynarek and Jędrzej Wierzbicki

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Zbigniew Młynarek and Jędrzej Wierzbicki

Technical editors
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Foreword

In the last decade in-situ methods of subsoil testing have received particular attention. In these groups of tests a unique position is occupied by static penetration tests and dilatometer tests. This is because these methods make it possible to determine strength and deformation parameters of soils at geostatic state of stress in the subsoil. Geological processes in many countries, particularly in Poland, lead to a situation when interpretation methods for characteristics obtained from CPTU tests and SDMT dilatometer tests are constantly improved thanks to the establishment of the so-called local correlation dependencies. Experience gained in this respect by numerous foreign research centers is particularly valuable, thus invitation to the Seminar extended to outstanding foreign specialists was the keynote for Prof. dr hab. Z. Młynarek when deciding together with his team to organize international seminars in Poznań on in-situ subsoil testing. The first seminar was held in 2001, followed by the next ones in 2004, 2007 and 2010. Since the first seminar the Organizing and Scientific Committee was joined by dr T. Lunne (the Norwegian Geotechnical Institute, Oslo) and dr J. Powell (Building Research Establishment, United Kingdom), who have unfailingly extended their advice and help in the determination of the seminar program.

The present seminar is devoted to a particularly important problem, i.e. determination of geotechnical parameters of soft cohesive and organic soils using CPTU, SDMT and vane test - VT. The organizers of this seminar have divided the proceedings into 4 sessions. The first will be devoted to static penetration tests and dilatometer tests in soft soils, the second to the in situ investigations in organic soils, the third to some aspects of site characterization by CPTU and SDMT, while the fourth will contain presentations connected with foundation of structures on weak subsoil. This session will be supplemented with a speech given by a representative of a. p. van den Berg from The Netherlands. This talk will present the latest achievements a. p. van den Berg, concerning measurement of parameters in CPTU testing.

The official part of the Seminar opening ceremony will include a lecture by dr R. Massarsch on the history and prospects for the development of static penetration method. The first session is devoted to the application of static penetration and flat dilatometer test to determine geotechnical parameters of soft soils. In this session dr R. Ghanekar (Institute of Engineering and Ocean Technology IEOT, India) will comment on the concept to determine unit weight of Indian offshore soft calcareous clays based on CPT characteristics. Prof. M. Long (University College Dublin, Ireland) will present the application of CPTU and SDMT to the characterization of Irish silts. Prof. D. de Groot (University of Massachusetts, Unites States of America) will discuss results of CPT testing for soils, which are classified as intermediate soils, i.e. dusts. Identification of subsoil structure under the Vistula River dikes using CPTU will be the subject of a lecture by dr L. Bałachowski and prof. Z. Sikora (Gdańsk University of Technology, Poland).

Session 2 of the Seminar is a continuation of the first session and it will also be supplemented with papers delivered on DMT. This session will include lectures by prof. H. Tanaka (Hokkaido University, Japan) about the determination of geotechnical properties of peaty ground in Hokkaido, while the presentation by prof. Z. Lechowicz (Warsaw University of Life Sciences, SGGW, Poland) will be related to the application of SDMT in the determination of geotechnical parameters of organic soils. Lecture by dr J. Powell is connected with the evaluation of changes in parameters of organic soils under an earthen structure. Prof. F. Danziger (Federal University of Rio de Janeiro, Brazil) will discuss the results of seismic DMT in very soft organic clay. The first two sessions will be concluded with a panel discussion, moderated by eng. K. Schjetne (Norwegian Geotechnical Institute, Norway). During this discussion the Seminar participants will be able to present their problems connected with the interpretation of CPTU and SDMT.

The third session is devoted to SDMT and assessment of relationships between mechanical parameters of soils obtained from these tests and from CPTU. Prof. S. Marchetti (University of L'Aquila, Italy) will discuss aspects of DMT and SDMT generally unknown until recently. Prof. P. Monaco (University of L'Aquila, Italy) in her lecture will present the examples of application of SDMT in silty-clayey sites while eng. D. Marchetti (Studio Prof. Marchetti s.r.l., Italy) will give the overview of the use of seismic dilatometer for in situ soil investigations. Eng. A. Barwise (Gardline Geosciences, United Kingdom) will present a new design of the cone with a friction sleeve of greater sensitivity developed for tests in soft loams. Correlations between shear strength parameters and the constrained modulus of organic soils recorded in CPTU, SDMT and vane tests will be analyzed in a presentation by prof. Z. Młynarek, dr S. Gogolik (Poznan University of Life Sciences, Poland), prof. J. Wierzbicki (Adam Mickiewicz University, Poland) and eng. M. Bogucki (ILF Consulting Engineers, Poland).

The fourth session of the Seminar will start with the dr T. Lunne's lecture on the determination of undrained shear strength in a deep water site in the Norwegian Sea. Next part of this session will comprise interesting presentations concerning foundation of structures on weak subsoil by dr J. Kawalec from Tensar International. This session will also include a paper by a representative of a. p. van den Berg – eng. M. Woollard. Finally eng. T. Pradela (Menard Polska) will present case histories related to modern techniques of soil improvement. The discussion panel for sessions 3 and 4 will be moderated by dr J. Powell.

The present Seminar, similarly as the previous ones, is a special event for Polish geotechnical engineers, as it will broaden their knowledge on the interpretation of CPTU and SDMT methods, currently implemented in Poland on an extensive scale. Apart from the scientific aspect, a particular success of this Seminar is connected with the participation of outstanding specialists in this field and with the special atmosphere of this event. This Seminar could be organized thanks to the financial support of our sponsors, thus the introductory remarks to this monograph offer a great occasion to mention these sponsors on behalf of the Organizers. These are the four companies:

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A Historic Overview of Cone Penetration Testing

K. Rainer Massarsch
Geo Risk & Vibration AB, Bromma, Sweden

Abstract. Penetration testing is a relatively recent geotechnical field investigation method, but which has become very popular during the past four decades. A historic overview of penetration testing methods is given, with emphasis on the development of cone penetration testing. The European Symposium on Penetration Testing, ESOPT I, held in 1974, was the platform, which initiated many positive developments, such as harmonization and standardization of penetration testing equipment and methods. The role of ISSMGE Technical Committees in this development is described, in particular the efforts by ISSMGE TC16/TC102 in organizing international symposia and conferences. The results of efforts to standardize penetration testing are presented. An effort was made to review the contents of papers published at symposia held during the past 40 years to detect major trends in testing and application of test results.

Introduction

A review of recent developments in geotechnical and foundation engineering clearly shows that cone penetration testing (CPT) has indeed become one of the most widely used geotechnical investigation methods. Examples are different types of pressuremeter, the dilatometer, and geophysical methods. Some of the reasons are: - testing method and equipment are relatively easily standardized, - the test provides a continuous soil profile, compared to other, intermittent, tests, - different types of sensors can be incorporated in the same probe, - the method is relatively fast and therefore less expensive than most other in-situ methods, - the rapid development in electronic engineering has facilitated quick and efficient data acquisition and evaluation of large data volumes, and – the concerted effort by researchers and geotechnical practitioners from different countries has helped to develop design concepts which can be used for the solution of many types of geotechnical, geo-environmental and earthquake-related problems. However, it is important to recognize that also other in-situ methods have become available and are widely used today. Examples are different types of pressuremeters, the dilatometer and geophysical methods.

Looking back at the developments over a time period of 40 years in a relatively young engineering discipline, such as geotechnical engineering, is fascinating. At the end of the 1960s, many of the engineering challenges of today did not exist or were just emerging. Heavy structures were either founded on rock or hard soil or supported by piles. The number and size of land reclamation projects was still small and their planning, design, and execution was often based on relatively unsophisticated concepts. Geotechnical earthquake engineering was emerging and offshore engineering was still in its infancy. The use of electronic equipment on a construction site was an exception and restricted to scientific projects. Data recording was often manual, by reading dial gages. Also the accuracy and reliability of field measurements was low and their quality depended to a large degree on the skills of the site crew. A factor, which limited the development of equipment and the application of penetration testing results in practice, was the variability of methods and equipment used in different parts of the world. Various types of in-situ tests had been developed, often in specific regions, and design tools based on local experience, which limited their wider application.

Without the vision of a group of outstanding geotechnical engineers – most of them no longer active – who recognized the need of information exchange, the rapid progress in CPT accomplished during the past 40 years would not have been possible. This report is an attempt to pay tribute to their important contributions.

Early developments of penetration testing

Penetration testing is a relatively new soil investigation method, compared to boring, excavation and visual site inspection. Comprehensive reviews of the history of site investigations have been published by for instance, Sanglerat (1972) and Broms and Flodin (1988). The first known, documented use of penetrometers for engineering applications was in the 14th century when the German military engineer, Konrad Kyeser (1366 – 1405?) began to write books on military and mechanical arts: A medieval concept of ordered practices or skills. He showed examples of screw-type tools to test materials and probably also the ground, (Cambefort, 1955). Leonardo da Vinci (1452–1519) sketched a screw type implement which may be advanced into the ground by means of turning the lever. However, this screw was most likely never used during the lifetime of Leonardo da Vinci. Little evidence of the practical application of sounding tools can be found until the end of the 17th century.

A ram sounding device was invented in Germany by Nicolaus Goldmann (1611 – 1665) as described by Broms & Flodin (1988): Hereto on the site at each place, a pointed rod can be driven, and one can notice the penetration depth for each blow, and in this manner one can find differences in the subsoil, according to a translation from German by H. Zweck (1969). This is most likely the forerunner of the light German penetrometer, re-invented in the mid 1930s. Also in the 18th century, Germany dominated the development of new soil exploration tools. In Central Europe, with relatively favorable (hard) ground conditions, engineers worked mainly with auger tools. In France, different sounding devices were used to determine ground conditions. H. Gauthier (1660 – 1737) described the practical application of penetrometers for construction of bridges and fortifications. In northern European countries with many deep deposits of loose or soft soil, however, the simpler rod penetration concept was applied.

During the 19th century, several handbooks were published in Europe, mainly in Germany, which included chapters on soil, rock and foundation engineering (Grundbau). The continental philosophy of building on firm ground prevailed in many countries. Sounding rods (Sondiereisen or Visitiereisen) were described for the first time in detail in Handbuch der Tiefbohrkunde by the German engineer Tecklenburg (1885/86). The sounding rod had a maximum length of approximately 4 m and was provided at the bottom with a point and an eye at the top for a wooden cross bar, intended for the withdrawal of the rod.

In England, different types of soil investigation tools were used. For the construction of the Westminster bridge, Charles Labelye, a naturalized Swiss engineer and architect, was employed. He used drill rods to obtain information on the nature and consistency of the soil, determined by the resistance to the penetration of the boring rod.

In North America, wash borings was for a long time the most commonly used drilling method. A remarkable penetration testing campaign was carried out in Canada in 1872 in connection with the construction of the Intercolonial Railway. The chief engineer was Sir Stanford Fleming (inventor of the Standard Time Zones) who pioneered engineering techniques, including soil sampling and the prestressing of piers. He proposed a new soil investigation method where a steel rod was pushed down into the soil and the required force was measured (Legget & Peck, 1973). The friction along the rod was eliminated by a 125 mm diameter steel casing. The rod used for the testing had a diameter of 75 mm and the end was blunt. The rod was loaded axially using weights. The time of loading was varied. This was probably the first application of a static penetrometer.

In Scandinavia, several handbooks, based mainly on the work by Tecklenburg, were widely used for military engineering projects. In the 1890s, almost everyone in charge of site investigations chose his own penetration testing method. In very soft soils, sounding rods were employed to investigate for instance areas where landslides had occurred. Rods were pushed into the ground, to depths of 10 to 12 m, often without reaching firm bottom. Svensson (1899), a Swedish railway engineer, was dissatisfied that the strength and the bearing capacity of soils could not be predicted quantitatively. He suggested that the penetration resistance should be measured more systematically and be expressed in terms of the number of men required to push down the rod. Grad 0 was used to indicate when the rod sunk by its own weight; Grade 1 when one man was required; Grad 2 when two man were required etc. Half-grades were also used. Ernst Wendel (1900), at the time head of the Harbour and River Works in Gothenburg, proposed to replace square sounding rods by a new type of needle probe, composed of several short pipe sections which were connected using in-side couplings in order to achieve a smooth outside surface, shown on a drawing from 1914. Instead of expressing the soil resistance in terms of number of men, Wendel proposed to drive down the probe using a weight, which was dropped from a predetermined height, and to measure the penetration for each blow.

Emergence of modern penetration testing

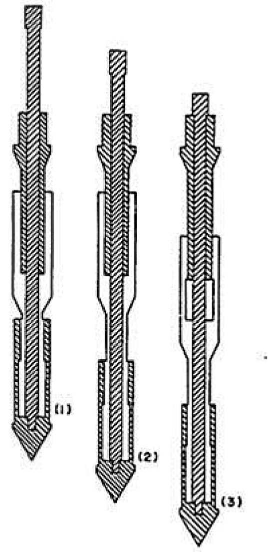
Although different types of penetration tests and sounding tools had been used during the past centuries, excavation and soil boring was the dominant method of soil investigation in Europe. However, at approximately the beginning of the 20th century, modern penetration testing methods started to develop in Central and Northern Europe where soft and loose soil deposits were encountered in connection with the development of the transportation infrastructures, such as railways, harbors, and roads.

The concept to determine the strength of soils by pushing or dropping a cone into the soil was developed early. This method was used by John Olsson in 1915 to determine the undrained shear strength of very soft clay, (Massarsch & Fellenius, 2012). A pocket penetrometer was later developed by the Danish State Railroads in 1931, which is based on the principle of the fall cone test. A predecessor of the mechanical cone penetrometer was the wash point penetrometer where a conical point with 70 mm diameter, attached to the lower end of a 50 mm diameter heavy wash pipe, is pushed into the soil, Terzaghi & Peck (1948). A 75 mm diameter casing eliminated the skin friction resistance along the wash pipe. The force required to push the penetrometer 250 mm into the soil, using a hydraulic jack, was measured. Water jets were then turned on so that the casing could be driven down to the level of the cone. After the water had been turned off, the point was forced down an additional 250 mm and the corresponding force was measured. This penetrometer has not been used widely, probably because of the difficulty to operate the equipment.

The Dutch cone penetrometer was initially developed around 1930 by Pieter Barentsen, a civil engineer at the Department of Civil Works (Rijkswaterstaat) in the Netherlands (Barentsen, 1936). He invented a way to measure accurately the resistance of the soil reacting on the conical tip. He inserted an inner rod into the sounding tube and pushed with this inner rod manually on the interior part of the conical tip. The soil resistance was read out by means of a hydraulic measuring head provided with a pressure gauge, Fig. 1.

The purpose of penetration testing at the time was to determine the thickness and bearing capacity of approximately 4 m thick hydraulic fill near the town of Vlaardingen. The 10 cm2 cone with a 60 degree apex angle was pushed down by two men and the penetration resistance was read on a manometer. A hand-operated cone penetrometer was first built by Goudsche Machinefabriek, Holland. Because the maximum force to push down the inner rod had to be delivered by the weight of the operator (approx. 80 kg), the maximum measurable cone resistance was often not sufficient to advance the penetrometer, which limited the application of the apparatus. In 1935, under the supervision of T.K. Huizinga, then director of Delft Laboratory of Soil Mechanics (LGM), the first deep cone penetration test with a pushing force of 10 tons was performed, as described by Platema (1948). The original cone penetrometer involved simple mechanical measurement of the total penetration resistance required to push a tool with a conical tip into the soil. Different methods were employed to separate the total measured resistance into components generated by the conical tip (the tip friction) and friction generated by the rod string. In 1950, a jacket cone was developed by J. Vermeiden to avoid measuring errors that could occur when sand entered the cavity between the rods. However, this sleeve affected the measured penetration resistance, particularly in clay.

a) Mechanical cone testing around 1950



b) Begemann mechanical cone with friction sleeve

Fig. 1. Early Dutch mechanical penetrometer, from History of Cone Penetration Testing – Gouda GeoEquipment BV.

Static penetration testing equipment was also developed and used in other European countries. A cone penetrometer was developed at the Belgian Geotechnical Institute by DeBeer (1945). It was provided with a fixed cone and the total skin friction resistance was measured separately. The first mechanical cone penetrometer in the USSR was developed in 1953, with a maximum capacity of 100 kN (Broms & Flodin 1988).

In the early 1950s, another significant development was the SGI cone penetrometer, which was mounted on a vehicle. It was invented by Kjellmann and Kallstenius at the Swedish Geotechnical Institute. The penetrometer had a conical point with either 25 mm or 40 mm diameter. A special feature of this penetrometer was that the rod could be rotated as the point was pushed into the soil. From the torque measurement it was possible to separate the point resistance from the skin friction. This separation was done automatically by the machine.

A significant advancement of the static cone penetrometer by Begemann was to measure the skin friction resistance every 0.2 m with a separate friction sleeve located just above the cone (Begemann, 1965). He published graphs enabling the frictional load capacity for different sorts of piles to be deduced from the measured local friction resistance. Figure 1b shows the mechanical friction cone introduced by Begemann (1953).

The first experimental electric cone penetrometer dates back to the end of World War II in Germany, (Broms & Flodin, 1988); followed by prototypes developed by the Rotterdam civil servant Bakker and LGM, Delft, in 1949. Fugro was first to introduce the electric cone in 1965 for routine soil investigation. In the mid 1970s, following research with pore pressure measurements in Norway, USA, and Sweden, standard electric penetrometers were equipped with sensors to measure the pore pressure during the penetration of the cone (Torstensson, 1975, Wissa et al., 1975). The Piezocone could measure in addition to the cone resistance and the sleeve friction also the pore pressure. The equipment was further developed by Fugro and other manufacturers. In Sweden, Torstensson (1975) was first to carry out a so-called dissipation test. From measured changes in pore pressure during a pause in the CPT test, it became possible to calculate the permeability of fine-grained soils.

Inclinometers were incorporated in modern penetrometers to detect deviation of the penetration path from the vertical direction. Another interesting development has been the incorporation of acoustic sensors in order to determine soil type, based on the emission of sound during cone penetration, (Massarsch, 1986).

Campanella and Robertson (1984) integrated a small velocity seismometer into an electronic cone penetrometer. The cone penetration test is briefly stopped to conduct seismic down--hole tests at specific depths. Today, the seismic cone penetration test can be provided with one, or an array of seismic sensors. Performing seismic down-hole measurements during a penetration test is much quicker and less expensive that standard cross-hole tests or down-hole tests.

Extensive research during the past decade, especially in the area of geo-environmental engineering, has led to the development of new, sophisticated penetrometers. Different types of sensors can be incorporated in the probe, for example laser-type and fiber-optical sensors, high-resolution ground-penetrating radar antennas and integrated opto-electronic chemical sensors. It is interesting to note that the cone penetrometer has evolved from a elatively simple, standardized geotechnical investigation tool into a multi-purpose testing instrument. The cone penetrometer has the potential of becoming a multi-purpose instrument, which offers new areas of application for geo-environmental investigations.

European symposium on penetration testing, ESOPT

The (first) European Symposium on Penetration Testing, ESOPT I, was held at the Royal Institute of Technology (KTH) in Stockholm, Sweden from June 5-7, 1974 (Fig. 2). The symposium was organized by the Swedish Geotechnical Society (SGF). Conference chairman was Prof. Bengt B. Broms, then director of the Swedish Geotechnical Institute and professor at the Royal Institute of Technology (KTH) in Stockholm. All European national geotechnical societies were invited to take part in the symposium, as well as a few societies from outside of Europe, with special interests in penetration testing. The response to the invitation was overwhelming and it was necessary to restrict the number of participants (originally estimated to be 80), primarily because of the limited facilities. The number of participants exceeded 200. A total number of 136 papers from 28 countries were submitted to the symposium. The Organizing Committee believed that it would be valuable to have - as background material primarily for the group discussions - a description of the different penetrometers and the test procedures utilized in various countries, of the methods used in the interpretation of the test results and the need of standardization of different testing methods. National reports were prepared by 19 European countries and 8 countries from outside Europe. All national reports followed the same outline: - geological background, - description of penetrometers used in each country, - test procedures, - interpretation and evaluation of test results, and - needs of future developments including standardization. In this way, it was possible to obtain a comprehensive picture of the status of penetration testing at that time. State-of-theart reports were presented at the symposium by four general reporters, representing Scandinavia, Central and Western Europe, Eastern Europe, and countries outside of Europe. The division into four groups was made primarily because equipment and testing procedures used in the interpretation of the test results differed between the various regions.

An important objective of ESOPT I was to promote the needs in standardization of different penetration testing methods. Therefore, participating countries were invited to present their national standards or guidelines on penetration testing and/or to comment on existing standards used elsewhere. Contributions from 21 countries were received. General Reports, summaries of the group discussions and a listing of national standards were documented in Vol. 2:1. Accepted papers were published after the conference in Vol. 2:2. The ESOPT I proceedings are available from the CPT'14 conference website, thanks to the generosity of the Swedish Geotechnical Society (SGF).

In hindsight, ESOPT I was a milestone in the development of penetration testing, and in particular that of cone penetration testing. For the first time, a comprehensive document became available which illustrated the practical application of different types of penetration tests. The information provided in the ESOPT proceedings formed the basis for harmonization of interpretation methods and future standardization.

Site of field demonstration
Penetration testing in action
Bengt B. Broms (right)
E. DeBeer (left)
K. Senneseth and N. Janbu
E. deBeer, M. Fukuoka and A. Vesic

Fig. 2. Field demonstration and participants at ESPOT I in Stockholm

Evolution of modern cone penetration testing

The Role of ISSMFE/ISSMGE

The International Society for Soil Mechanics and Foundation Engineering (ISSMFE), through its technical committees, has played an important role in promoting and developing the application of cone penetration testing. At the 4th International Conference on Soil Mechanics and Foundation Engineering in London in 1957 an ISSMGE Subcommittee on penetration testing was created for the purpose of studying static and dynamic penetration tests with a view to their standardization.

In order to initiate and to stimulate the work on standardization and because of the rapid development of penetration testing during the past years, Prof. B. B. Broms, then chairman of the national Swedish Committee on Penetration Testing, offered to arrange a European symposium on penetration testing in Stockholm. The proposal was discussed with Professor de Beer, Belgium, at that time ISSMFE Vice President for Europe, during a visit to Stockholm in 1972. The proposed plan was approved with minor modifications and the (first) European Symposium on Penetration Testing (ESOPT-I) was held in Stockholm in 1974. The stated aims of ESOPT I were: to document the use of penetration tests in soil investigations in different countries, to outline areas where further research is desirable, to stimulate the standardization of commonly used penetration testing methods, and to provide guidelines for future developments.

Emergence of Cone Penetration Testing from 1974

Since the first European Symposium on Penetration Testing, ESOPT I in 1974, major changes have taken place in civil and geotechnical engineering. Advanced numerical tools, such as Finite Element Analyses and other numerical methods have found increasing use in research but also for the solution of engineering problems. The reliability of numerical analyses depends to a large degree on the quality of chosen input parameters. Cone penetration testing (CPT) is one of several geotechnical investigation methods, which can provide information about strength and deformation parameters of soils, especially in soils which cannot be sampled or are difficult to sample. In the early 1970s cone penetrometers were rarely used in engineering practice in North America. During the early 1980s and thereafter, their use grew rapidly and spread to many new areas of application. New areas of CPT application emerged for the assessment of earthquake engineering problems (liquefaction susceptibility) and especially in offshore engineering.

It is interesting to note that the first application of penetrometers in space engineering was reported to ESOPT I by Mitchell & Houston (1974). On an international scale, penetration testing and in particular the CPT started to play a central role in the planning, execution, and supervision of very large land reclamation projects, required for the construction of new airports, harbors, or industrial and residential facilities.

During the past decade, cone penetrometers, provided with new types of sensors, have become increasingly important in environmental engineering. This development has only started and further progress can be expected in the years to come.

Standardization of penetration testing methods

International, regional and national standards and reference procedures have been developed with respect to penetration testing in general, and cone penetration testing in particular. This section summarizes the most widely used standards, prepared by International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE), American standards institute (ASTM International) and the International Organization for Standardization (ISO) in cooperation with the European Committee for Standardization (CEN).

ISSMFE/ISSMGE Reference Procedures on Penetration Testing

Following the discussions and reports at the European Symposium on Penetration Testing in Stockholm in 1974, the European Subcommittee on Penetration Testing submitted its final report at the 9th ICSMFE in Tokyo in 1977. The report also included reference procedures for the following penetration testing methods: Cone Penetration Testing (CPT), Standard Penetration Test (SPT), Dynamic Probing (DP) and Weight Sounding (WST). The proposal was approved by the ISSMFE Council with the recommendation that these recommendations should also be used outside Europe.

At the 10th ICSMFE in Stockholm in 1981, the Committee repeated its recommendation that papers to international conferences or journals should include results from at least one recommended standard penetration testing method. The Subcommittee also recommended Comparison between the different recommended standard penetration methods should be made in different soils to facilitate the evaluation of soil characteristics from different penetration testes.

Work on developing reference procedures for penetration testing continued within Technical Committee, TC 16 and the results were presented in Report of the ISSMFE Technical Committee on Penetration Testing of Soils - TC 16, with International Reference Test Procedures (IRTP) for: CPT - SPT - DP - WST. The following reference test procedur was published in the Proceedings of the International Symposium on Penetration Testing – ISOPT 1, held in 1988: Cone penetration test (CPT): International reference test procedure. De Beer, E.E,. Goelen, E., Heynen, W.J. & Joustra, K. International symposium on penetration testing, 1, ISOPT-1, Orlando, March 1988. Proceedings, Vol. 1, 1988, pp. 27-51.

The IRTP for the CPT was updated to include CPTU by TC16 in 1999 and published in the Proceedings of 12th European Conf. on Soil Mechanics and Geotechnical Engineering, Amsterdam. TC 16 also produced reports on the Pressuremeter in 1998 (ISC'98) and the DMT in 2001 (exist in, 2001). As a supplement to the Reference test procedure for cone penetration testing, ISSMGE Technical Committee 10, Geophysical Testing in Geotechnical Engineering prepared Guidelines for execution of the seismic cone downhole test to measure shear wave velocity (SCPT). The document was presented at the 16th ICSMGE in Osaka in 2005, Butcher et al. (2005) but never officially published.

ISO and CEN Standards (Eurocodes)

The International Organization for Standardization (ISO) has prepared standards in cooperation with the European Committee for Standardization (CEN). CEN members are bound to comply with the CEN/CENELEC Internal Regulations, which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. European Standard are published in three official versions (English, French and German).

The first Eurocode 7 Working Group, in charge of drafting a European standard on geotechnical design, was created in 1981. It was composed of representatives of the National Societies for Geotechnical Engineering of the 10 countries forming the European Community at that time. A model code for Eurocode 7 was submitted to the European Commission in 1987 – it was not actually published. This had just one part. The next stage was the ENV, or trial stage published by CEN, and it had 3 parts. The ENV version of Part 1 was published by CEN in 1994 and the ENV versions of Parts 2 and 3 were published by CEN in 1999. Details are given by Orr (2008), "The Story of Eurocode 7 in Spirit of Krebs Ovesen Session – Challenges in geotechnical engineering". During the conversion phase, the two documents were merged into the single document called Eurocode 7 Geotechnical design - Part 2: Ground investigation and testing. The formal positive vote was obtained in May 2006 and the document was published in March 2007.

Eurocode, Part 2 includes the following field testing procedures: cone penetration tests, CPT(U); pressuremeter tests, PMT; rock dilatometer tests, RDT; standard penetration tests, SPT; dynamic penetration tests, DPT; weight sounding tests, WST; field vane tests, FVT; flat dilatometer tests, DMT and plate loading tests, PLT.

The following CEN/ISO standards are relevant for cone penetration testing:

EN ISO 22476-12:2009. Geotechnical investigation and testing - Field testing - Part 12: Mechanical cone penetration test and EN ISO 22476-1:2012. Geotechnical investigation and testing - Field testing - Part 1: Electrical cone and piezocone penetration test. The standard on CPT deals with equipment requirements, the execution of and reporting on electrical cone and piezocone penetration tests as part of geotechnical investigation and testing according to EN 1997-1 and EN 1997-2. Within the electrical cone and piezocone penetration test, two subcategories of the cone penetration test are considered: electrical cone penetration test (CPT), which includes measurement of cone resistance and sleeve friction and piezocone test (CPTU), which is a cone penetration test with the additional measurement of pore pressure. The CPTU is performed like a CPT with the measurement of the pore pressure at one or several locations on the penetrometer surface.

EN ISO 22476-1:2012 specifies the following features: - type of cone penetration test; - application class; - penetration length or penetration depth; - elevation of the ground surface or the underwater ground surface at the location of the cone penetration tests with reference to a datum; - location of the cone penetration test relative to a reproducible fixed location reference point; - pore pressure dissipation tests.

Conferences on penetration testing

A large number of conferences, symposia, meetings and workshops have been held since ESOPT I, in addition to larger ISSMGE international and regional conferences. These meetings addressed different aspects of cone penetration testing, but had often a broader scope than cone penetration testing. In the Reference section of this paper, a comprehensive listing of conferences and symposia, covering different aspects of penetration testing and site characterization, is given.

In the following section, papers submitted to five international symposia/conferences on penetration testing since 1974 have been compiled in a database and evaluated with the objective of discerning trends and developments during the past 40 years:

- European symposium on penetration testing, ESOPT I - 1974
- European symposium on penetration testing, ESOPT II - 1982
- International symposium on penetration testing, ISOPT 1 - 1988
- Symposium on Cone Penetration Testing, CPT'95 - 1995
- 2nd International Symposium on Cone Penetration Testing, CPT'10 – 2010.

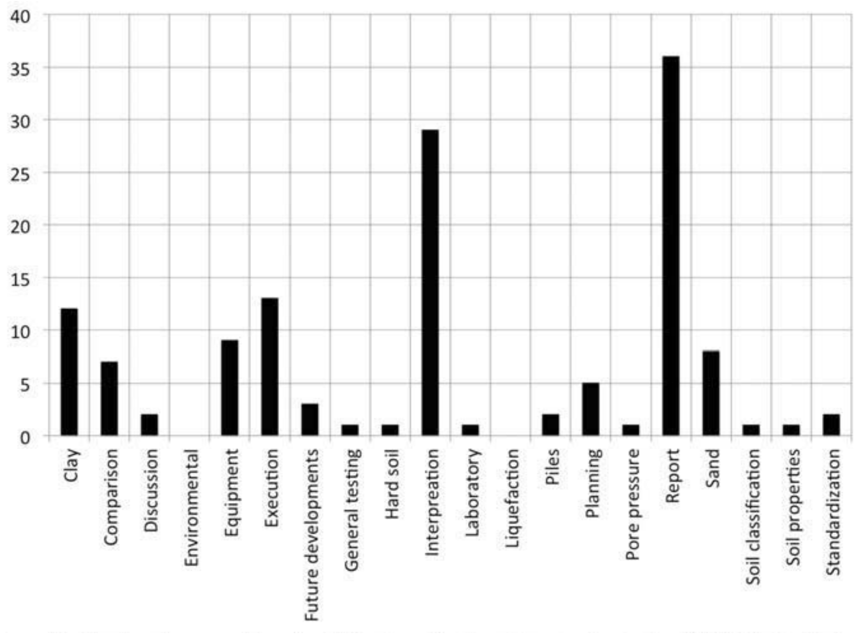


Fig. 3. Number of papers addressing different applications of penetration testing, ESOPT I, Stockholm

First European Symposium on Penetration Testing, ESOPT

In the proceedings of ESOPT I, 136 papers from 28 countries were published. It is interesting to note the relatively even distribution of submissions, with the largest number of papers from France, Sweden and the USA. The dominating topics were cone penetration testing and the comparison of different types of penetration tests.

Figure 3 shows the topics covered by the papers. As the objective of ESOPT I was to compile national and state-of-practice reports, this category dominates. The second largest number of papers addressed the interpretation of results from penetration testing. The third largest number of papers referred to the execution of penetration testing.

European Symposium on Penetration Testing, ESOPT II

The total number of papers submitted to ESOPT II was 128, coming from 29 countries. As the symposium was held in the Netherlands, contributions from this country dominate. The second largest number of contributions came from the UK (a significant increase from ESOPT), followed by the USA and France, respectively.

Again, it is not surprising that the number of papers on cone penetration testing dominate. Similarly to ESOPT I, this symposium covered all types of penetration tests.

The subject Determination of soil properties dominated the application of penetration testing, followed by a comparison of between different types of penetration tests, and the use for pile foundations. Again, determination of strength properties of clay by penetration tests was a popular subject.

International Symposium on Penetration Testing, ISOPT-1

The International Symposium on Penetration Testing, ISOPT-1 was held in held in the United States of America in 1988. A total of 111 papers from 31 countries were included in the proceedings. The largest number of contributions came from the host country, followed by Canada, Japan and the Netherlands.

As the main subject of the symposium was explicitly cone penetration testing, it is not surprising that the largest number of papers belonged to this category. The second largest group of papers addressed the comparison of different types of penetration tests. It is interesting to note the increase of papers on the dilatometer (DMT), which is not a genuine penetration test.

The largest number of papers covered the determination of soil properties, followed by questions related to data interpretation. Again, the third largest number of papers addressed the determination of properties in clay. Note that not a single paper was concerned with geo-environmental problems.

Symposium on Cone Penetration Testing, CPT'95

CPT'95 was again held in Sweden, more than 20 years after ESOPT I. The scope of the conference was specifically devoted to cone penetration testing. A record number of 151 papers from 45 countries were received. The proceedings of CPT'95 are available from the CPT'14 website. The USA had by then become the leading contributor of papers to the symposium with 30, followed by a group of countries: Canada, Denmark, Japan, Russia, Sweden and the United Kingdom.

Figure 4 shows the distribution of papers addressing different applications of penetration testing. As one important aspect of CPT'95 was to document the progress of cone penetration testing since ESOPT I, emphasis was on national and regional reports. The second most important topic of papers was the determination of soil properties, followed by the developments of new equipment. Another popular topic was the interpretation of CPT data with respect to soil properties.

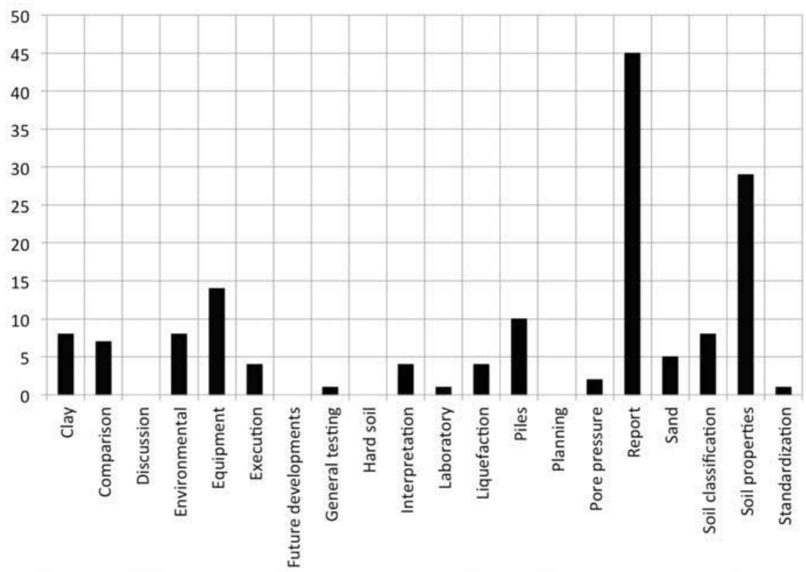


Fig. 4. Number of papers addressing different applications of penetration testing, CPT'95

2nd International Symposium on Cone Penetration Testing, CPT'10

The most recent symposium on cone penetration testing, CPT'10 was held in Huntington Beach, California in 2010, fifteen years after CPT'95. The total number of papers was 140, submitted from 31 countries.

Again, the largest number of papers was from the USA (41). Then follow papers from a large number of countries from different parts of the world, including Australia, Brazil, Canada, Italy, Korea (!) andTurkey.

Naturally, the largest number of papers was related to cone penetration testing, followed by a comparison between different types of penetration testing methods and the use of penetrometers in site investigation. Figure 5 shows the distribution of papers addressing different applications of penetration testing. Due to the fact that the symposium attempted to compile national and regional developments, the largest number of papers falls into the category Reports. Again, the determination of soil properties from CPT is the second largest category, followed by description and new developments of equipment.

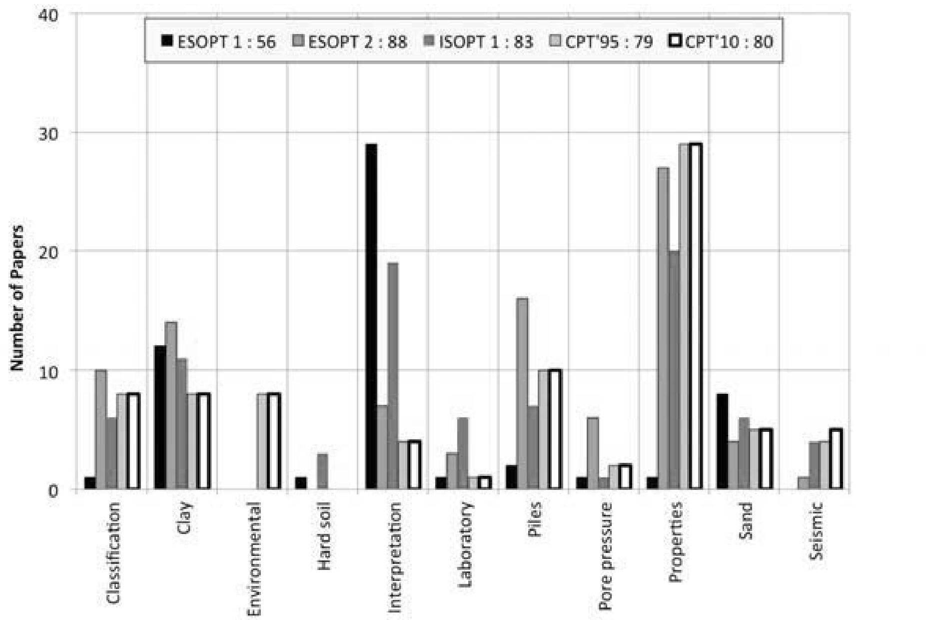


Fig. 5. Geotechnical Application of Penetration Testing Methods since ESOPT Stockholm, 1974

Trends in CPT applications during the past 40 years

The compilation of subjects of papers submitted to the symposia over the past 40 years can only give an approximate picture of the actual developments. The interest in the use of penetration tests for site investigation has steadily increased, which is not reflected in this summary. Also, other in-situ tests for site investigation, such as the pressuremeter, the dilatometer and geophysical tests, play an important role in geotechnical engineering, a fact which is not reflected in this summary.

However, some interesting trends in the geotechnical industry can be detected. The largest number of papers addressed general trends (Reports) with an aim to document regional and national developments. Clearly, there appears to be a need for updated reviews of the state-of-

-practice of penetration testing, and cone penetration testing in particular, while the need for comparison between different types of penetration testing methods appears to have declined. There is an increasing interest in the development of new types of CPT equipment while the number of papers regarding the execution of penetration tests has decreased, most likely due to the introduction of standardized methods.

Figure 5 shows the distribution of papers on different topics , submitted to conferences on penetration testing. While initially, focus has been on the interpretation of CPT and other penetration testing methods, the largest interest appears to be on the determination of soil properties. Also, there has been a steady interest in soil classification and the determination of the properties of clay soils. Another topic of great interest over the past 40 years has been the application of penetration tests with respect to the design of piles. The number of papers addressing seismic problems (liquefaction) has also increased. It should be pointed out that an increasing number of papers has been related to geotechnical offshore engineering, but it has been difficult to isolate this specific trend.

Future trends

After having presented the historic developments of penetration testing and describing the major trends over the past 40 years, it is tempting also to look ahead and try to identify future trends. However, considering the many unforeseen developments in the past, it becomes increasingly difficult to make any predictions. Still, an attempt will be made to identify future trends in penetration testing, being aware that these will not stand the test of time and probably will miss several new important aspects. Assessing future developments is of course made from a personal perspective. Future trends will be divided into four main categories: equipment and execution of penetration testing; interpretation of test data; standardization; communication and information exchange.

Equipment and Execution of Penetration Testing

If the past trend of equipment development and test execution continues, it is likely that cone penetrometers will become multi-purpose instruments applied to testing in different technical disciplines, for which new types of sensors will be incorporated. One example is the measurement of sound during penetration testing (Villet et al., 1981, Tringale & Mitchell, 1982 and Massarsch, 1986). When initially introduced in the mid 1970s, the volume of recorded data was too large to take advantage of the high resolution of sound recordings (thousands of values per second/cm). Today, it is possible to record and store large quantities of various acoustic signals (amplitude, frequency etc.) and to correlate these with other parameters, soil type and/or soil properties.

Full-flow penetrometers and push-pull type methods, such as the Iskymeter, have a potential for combined penetration and extraction testing. This trend has started within the offshore industry but may well spread to other areas of application.

Another positive trend, which is expected to continue, is the use of multiple testing methods, incorporated into one device. The seismic CPT (SCPT) is an example of such an application, where - in the same test location – deformation properties of soils can be determined by different methods.

It is also likely that the rapid development in the electronic industry will have a profound impact on penetration testing equipment. The accuracy of sensors is expected to increase and electronic equipment will become more rugged and suitable for site applications. Also the cost of electronic components, such as pressure or acoustic sensors, will decrease, making equipment more affordable.

The sensitivity of sensors and thus the accuracy of measured data will continue to improve, making measurements possible also in difficult site conditions, independent of background noise or disturbance.

It is also envisaged that the transmission of measured data from the sensor (e.g. incorporated in the cone tip) to the crew on the ground, and from there to any office world-wide, will make measured data instantly available to experts located anywhere. Sensors may become increasingly intelligent, becoming able to identify between multiple measured signals.

Interpretation of Test Data

Past trends suggest that the interest in the interpretation of penetration test date will continue. The popularity of penetration testing will depend on its ability to relate measured data either to soil parameters (stiffness, strength etc.) or directly to the performance of the ground (soil compaction) or geotechnical structures (pile capacity etc.).

Another likely trend will be the development of knowledge bases for different penetration testing methods, incorporating also probabilistic data interpretation models. Expert systems, which are widely used in other scientific areas, have not yet made a major impact on geotechnical engineering.

The comparison of results from different penetration testing methods will offer new possibilities for data interpretation and assessment of geotechnical parameters.

The computing power of even small hand-held devices will continue to increase, making it possible to treat and evaluate large data quantities rapidly and to correlate these with other soil data.

Standardization

The most commonly used penetration testing methods have been standardized, for instance in the CEN standards. Similar, though slightly different, standards or guidelines are used elsewhere. There is a continued need for integrating the different types of penetration tests (equipment specifications and test execution).

Communication and Information Exchange

It is interesting to note that at the time of ESOPT I (1974), the facsimile machine (fax) did not yet exist! With the introduction of the fax, it became possible to transmit drawings and sketches. Even more important developments were the introduction of cellular phones and the PC. The internet – and access to the World Wide Web - have fundamentally changed the ways of social and professional communication.

An important issue, recognized by the ISSMGE, is copyright to published information. ISSMGE has adopted a policy whereby the author gives exclusive publishing rights but retains copyright. This will allow the author to post publications on his website. Recognizing the importance of free information dissemination, several personal websites (google for example: either Campanella; Fellenius; Mayne; In situ GeoLinks; or Robertson, etc.) and public websites provide free access to published information (papers, reports etc.).

ISSMGE and other professional organizations publish technical articles which are accessible free of charge, such as SGI Line or the GeoEngineer: International Journal of Geoengineering Case Histories.

ISSMGE Technical Committees and others are offering short courses (Webinars) on the internet, which can be a powerful tool of information dissemination, especially in countries where access to higher education still is limited.

In spite of the increasing number of communication channels, it is obvious that the need for personal meetings and direct exchange of information and opinion will remain an important aspect of professional interaction. Participation of individuals in workshops, symposia or conferences will remain one of the most important aspects of scientific and social communication.

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Interpretation of cone penetration test (CPTU) in soft soils

Unit weight estimation from CPT for Indian offshore soft calcareous clays

Ravindra K. Ghanekar

Institute of Engineering and Ocean Technology (IEOT), Oil and Natural Gas Corporation Limited (ONGC), ONGC Phase – II, Panvel – 410 221, India

Abstract. Assessment of overburden pressure is required to derive most of the important parameters for soil classification or for estimating engineering properties from measured CPT data. This in turn requires an assessment of unit weight of the soils which generally takes time to obtain in offshore or onshore laboratories. A number of correlations have been proposed by researchers for obtaining unit weights from CPT data. An immediate assessment of unit weight, simultaneously with the performance of CPT, can allow immediate interpretation of CPT data on site using computer software.

This paper presents the result of application of four reported correlations based on basic CPT parameters on the data for soft to very soft calcareous fine grained soils from western Indian offshore. Results of attempts to refine the correlations and recommendation for use are also presented.

Introduction

Estimation of total and effective overburden pressure is required for deriving most of the engineering parameters using correlations with CPT/CPTU measurements. This requires an assessment of unit weight of the soils encountered which generally is measured on retrieved samples and takes some time to obtain in offshore or onshore laboratories. An assessment of unit weight directly from measured CPT parameters can allow estimation of engineering parameters, in the field itself, immediately after conduct of CPT/CPTU test. Attempts in this direction, reported in literature, can be put in the following two categories:

1. Direct correlations with measured CPT/CPTU parameters
2. Correlations using measurements from additional sensors with CPT

Some of the correlations, which involve total or effective overburden pressure, require an iterative approach. Measurements from CPT with additional sensors (e.g. Seismic Cone, CPT with nuclear density probe) are generally not available in routine commercial soil investigation jobs. The most desirable option for on-site assessment of engineering parameters is to make reasonable estimates of soil unit weight from direct correlations with measured CPT parameters using simple worksheets without the need of iterative procedure. To investigate this option for application on very soft to soft western Indian Offshore calcareous fine grained soils, four reported correlations were selected which employ direct correlations with measured CPT/CPTU parameters. The following sections present details of selected correlations, data used, results of direct application of the selected correlations and attempts to refine the correlation/to develop alternative relationships for application on calcareous fine grained soils of western Indian offshore.

Existing correlations

Literature review showed that there are four proposed correlations which use directly measured CPT parameters for prediction of total unit weight of soils. Only one of them (Mayne and Peuchen, 2013) deals specifically with clays i.e. normally to lightly overconsilidated clays. Other correlations were developed using data from all type of soils. These proposed correlations are detailed below.

Mayne (2007) proposed a relationship between sleeve friction (f_s) and the total unit weight (γ_t) indirectly derived from relationships between γ_t and shear wave velocity (V_s), and, V_s and f_s. Using a database which contained data for soft to stiff clays and silts; loose to dense sands and gravels the relationship proposed is:

$$\gamma_t = 2.6 \log(f_s) + 15\, G_s - 26.5 \tag{1}$$

Where G_s is the specific gravity of soil solids. The units of γ_t and f_s are kN/m^3 and kPa respectively.

Mayne et al. (2010) using data from 44 sites covering a wide range of soil type (sands, silts, clays, calcareous soils, tills etc.) proposed the following relationship involving f_s, depth (z) and cone resistance (q_t):

$$\gamma_t = 11.46 + 0.33 \log(z) + 3.1 \log(f_s) + 0.7 \log(q_t) \tag{2}$$

with the number of data points, n = 215, coefficient of determination, R^2 = 0.72 and standard error, S.E. = 1.31.The unit of γ_t is kN/m^3, f_s and q_t are in kPa and z is in m.

Robertson & Cabal (2010) proposed a general relationship for unit weight based on measured CPT parameters for soils from clays to sand based on DMT and shear wave velocity correlations and published data worldwide:

$$\gamma_t/\gamma_w = [0.27 \log(R_f) + 0.36 \log(q_t/P_a) + 1.236]\, G_s/2.65 \tag{3}$$

where γ_w is the unit weight of pore water, P_a is the atmospheric pressure and R_f is the friction ratio (f_s/q_t) in percent. The maximum depth of the data is 37.0 m with most of the data from up to 20 m depth. The relationship accounts for variation in G_s of the soils.$^{\gamma}$

Mayne & Peuchen (2013) proposed the following relationship for soft to firm normally to lightly overconsolidated clays (OCR generally less than 2):

$$\gamma_t = \gamma_w + 0.056\ (m_q)^{1.21} \qquad (4)$$

with n= 34, R^2= 0.623 and S.E. = 1.21.The resistance-depth ratio, mq is defined as the ratio of qt and z. The unit of γ_t and γ_w is kN/m^3, q_t is in kPa and z is in m. The relationship is applicable for soils falling in SBT zones 3 (clays to silty clays) and 4 (silt mixtures – silty clays to clayey silts) of Robertson (1990). The database contained data from 19 offshore and 15 onshore sites with depth in the range 1.5 m to 173 m and overall mean depth 24.2 m (± 29.8 m).tests and anisotropically consolidated triaxial tests sheared undrained in compression (CAUC) were carried out.

The data

The data used for this study was taken from recent ONGC files containing regular soil investigation data for 16 fixed offshore platform locations from Western Indian Offshore. The offshore fields are located in WGS 84 – UTM Zones 42 and 43 off the coast of Mumbai region. Most of the ONGC oil fields from western Indian offshore are represented. Most of the data is from measurements made on-board the Geotechnical Vessel. The clays in western Indian offshore generally contain significant amount of carbonate and on an average the specific gravity of soil solids is 2.8.

In most of the fields in western Indian offshore, surface layers are very soft to soft clays ranging in depth from a few meters to around 20.0 m below seafloor. In some of the locations surface sand layers of small thickness are encountered. General practice in Indian offshore is to perform a near continuous CPTU (standard 10 cm^2 A.P. van den Berg cone) generally up to a depth between 30.0 to 50.0 m and perform alternate sampling and CPTU in a borehole 5.0 m away up to a depth of 120.0 to 130.0 m. Hence, for different investigated platform locations, for very soft to soft clayey/silty soils, CPTU data and unit weight measurements at corresponding depths are available assuming that uniform conditions exist at 5.0 m distance. The soil profiles at the considered locations in this study have continuous very soft to soft clay profile generally for depth below seafloor ranging from about 10.0 to 20.0 m. In two locations small top sand layer exists followed by very soft to soft clay. Figure 1 shows Soil Behaviour Type (SBT) Index, Ic (Robertson & Wride, 1998) plotted with depth for this data and shows the soils to be either clays or silt mixtures. Terms Q_t and I_c are defined as follows:

$$Q_t = \frac{(q_t-\sigma_{v0})}{\sigma'_{v0}}; \qquad I_c = \sqrt{(3.47-\log Q_t)^2+(1.22+\log F_r)^2} \quad ; \text{ and}$$

$$F_r = \frac{f_s}{(q_t-\sigma_{v0})}$$

where σ_{v0} and σ'_{v0} are total and effective overburden pressures respectively and F_r is normalized sleeve friction.

The distribution of depth in the data used for present study is shown in Figure 2. Most of the data are from depth up to 15.0 m below seafloor. Carbonate contents generally are above 10% and reach up to about 75% or more in some layers. In all cases, the cone resistance has been corrected for unequal end area and also brought to a common seafloor reference. The pore pressure u_2 is also referenced to the seafloor. No correction has been applied to sleeve friction values.

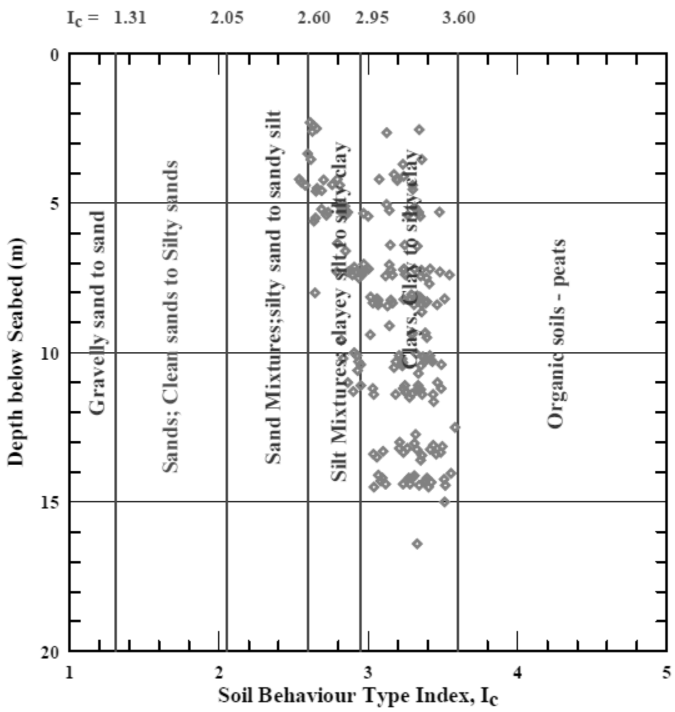

Fig. 1. SBT Index vs. depth

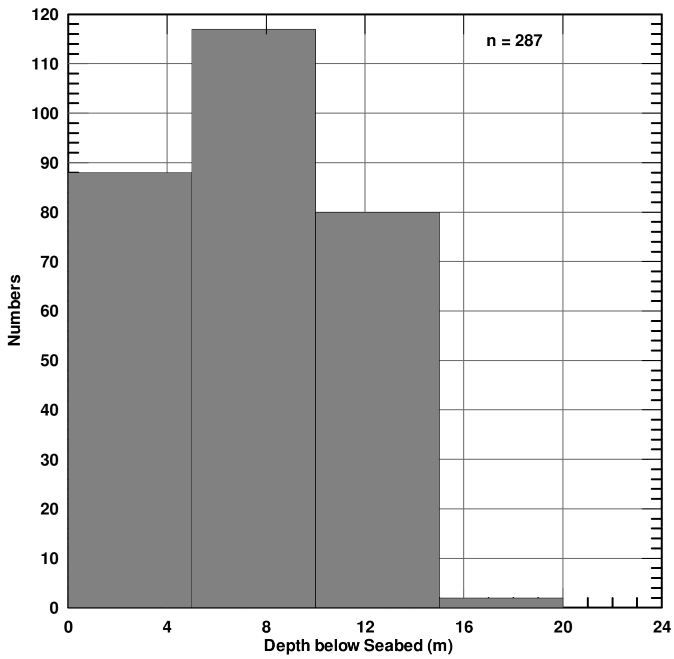

Fig. 2. Histogram of depth

Data analysis

Direct application of the selected relationships

Figures 3 and 4 present measured γ_t against the values predicted by Mayne (2007) and Mayne & Peuchen (2013) relations respectively. The range of ± 10% is also indicated on the figures. The figures show that the predictions are not very good. Figure 5 and 6 present the measured unit weights against the unit weights predicted by Robertson & Cabal (2010) and Mayne et al. (2010) relationships. It can be seen that the predictions generally are within 10% of the measured values but the scatter is high.

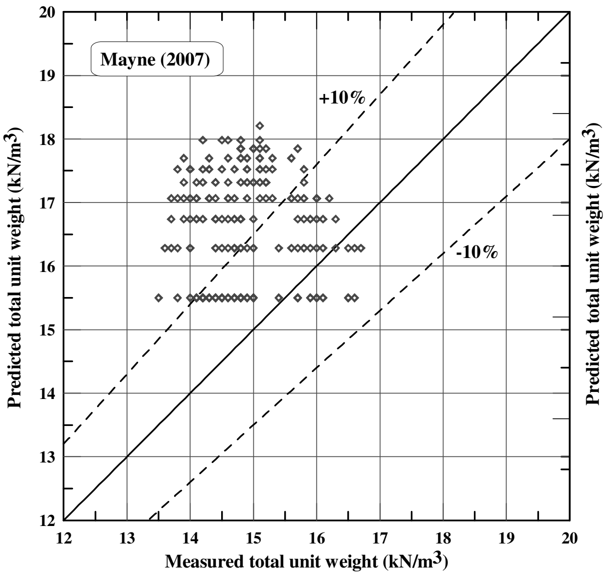

Fig. 3. Mayne (2007) predictions

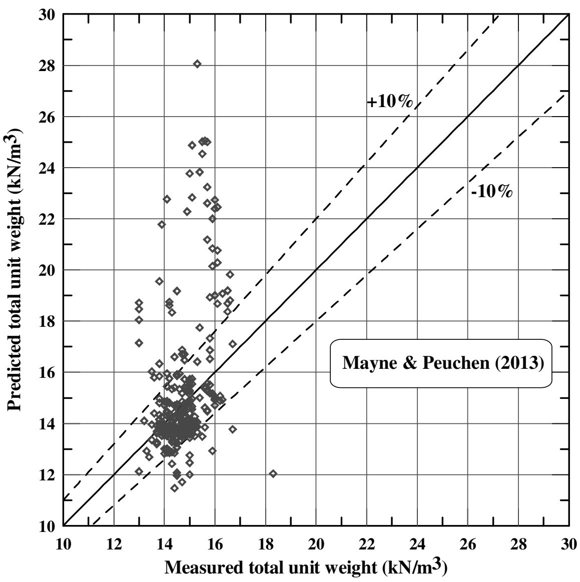

Fig. 4. Mayne & Peuchen (2013) predictions

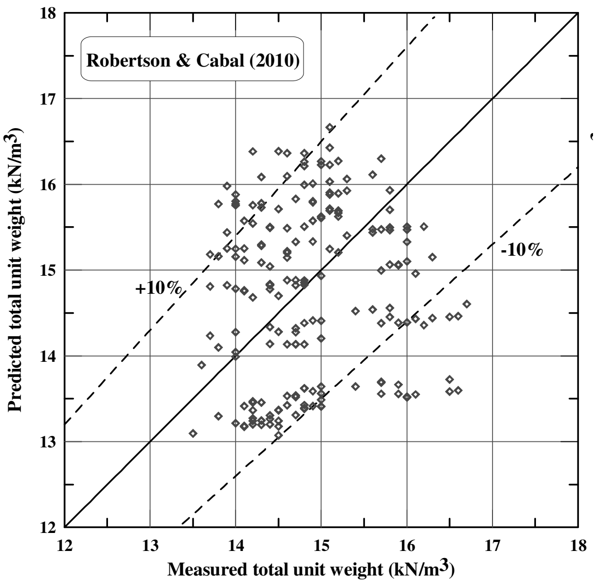

Fig. 5. Robertson & Cabal (2010) predictions

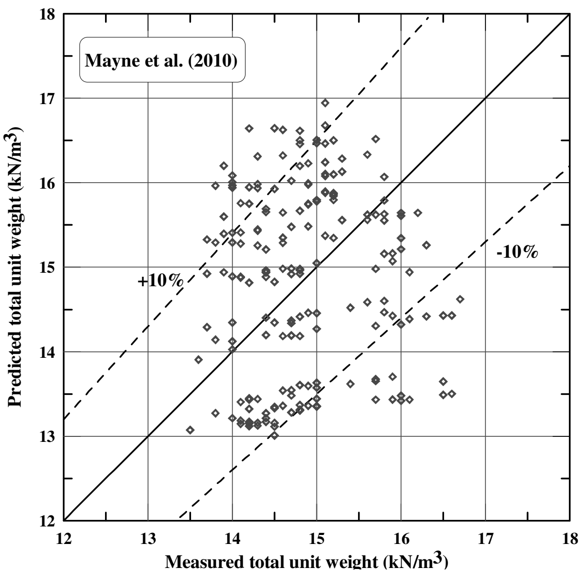

Fig 6. Mayne et al. (2010) predictions

Regression analysis using parameters from existing relationships

In an attempt to further refine the correlations, single and multi-variable regression analyses were performed on the data using the same parameters as those used in the selected relationships from literature. The results of such analyses are presented in Table 1. The relationships between γ_t and f_s, and, γ_t and qt were found to be very poor and are not reported here. Relationships involving qt in combination with f_s or z show only moderate strength (Equations C and D). However, regression result involving f_s, qt and z shows a reasonable strength (Equation A). Only moderate strength is shown by the regression involving parameters used in Robertson & Cabal (2010) relationship (Equation B). Surprisingly, relationship involving resistance-depth ratio, m (= q_t/z) as proposed by Mayne & Peuchen (2013) was found to be very poor and is not reported here. Varying the exponent of "m" did not improve the strength of the correlation. Figures 7 and 8 present the predictions from equations A & B respectively against the measured values. The figures also show the range of ± 5%. The predictions generally are within ± 5% but equation A predicts better as expected.

An interesting observation is that the terms involving sleeve friction and z always are negative unlike the published correlations. The reason for this is not very clear. Lunne (2012) showed that sleeve friction readings show significantly larger uncertainty compared to measured cone resistance and pore pressure, and, vary significantly depending on the cone type used. A possible reason for negative terms involving f_s can hence be the difference in measured f_s due to the use of different cone types in obtaining data for the selected papers and the present study.

Tab. 1. Results of regression analyses on the data

Equation	Results of regression analyses	n	R^2	S.E.
A	$\gamma_t = 4.077 - 0.5237 \log(f_s) + 5.377 \log(q_t) – 2.585 \log(z)$	194	0.698	0.396
B	$\gamma_t/\gamma_w = 1.357 - 0.1123 \log(R_f) + 0.2569 \log(q_t/P_a)$	195	0.551	0.049
C	$\gamma_t = 6.185 – 1.114 \log(f_s) +3.688 \log(q_t)$	194	0.56	0.478
D	$\gamma_t = 8.211 – 1.358 \log(z) + 3.191 \log(q_t)$	287	0.414	0.599

Note: q_t, P_a and f_s are in kPa, z in m and γ_t and γ_w in kN/m^3

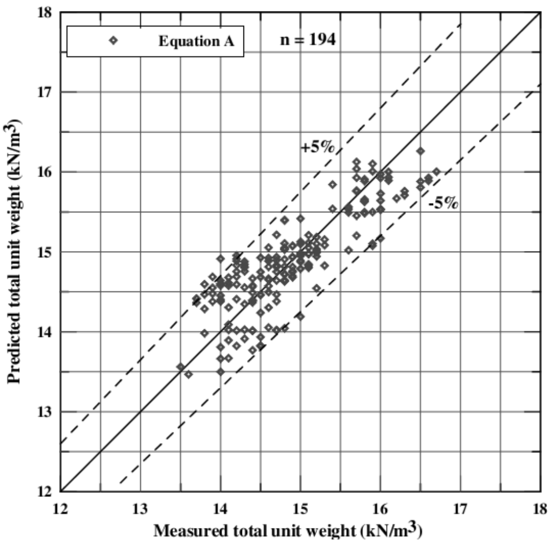

Fig. 7. Predictions from Equation A

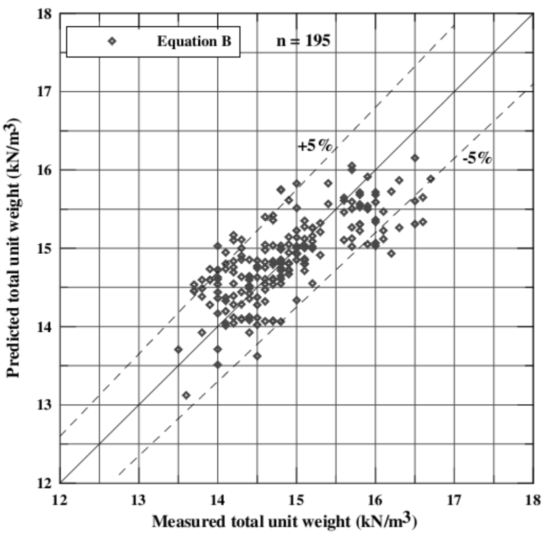

Fig. 8. Predictions from Equation B

Regression analysis using additional parameters

It is recognized that in very soft to soft clays measured CPT cone resistance and sleeve friction lack accuracy. That is why for deriving undrained shear strength, correlation with excess pore pressure is recommended. Also, a previous study on western Indian offshore calcareous clays (Ghanekar, 2014) had indicated that depth z is an important parameter in unit weight correlations based on CPT.

Hence, additional regression analyses were performed to see if the relationships can be improved by bringing in other parameters in addition to (or in place of) parameters used in the selected correlations from literature. Such attempts focused on involving measured pore pressure u_2 and depth z through the terms f_s/z ; Δu_2 (= u_2-u_0); "effective" cone resistance, q_E (=q_t-u_2); $\Delta u_2/u_0$; $\Delta u_2/z$ and $\Delta u_2/q_t$ where u0 is the initial in-situ pore pressure. The best results from such attempts are shown in Table 2.

It can be observed from equation E and F that inclusion of parameters involving Δu_2 improves the strength of relations as compared to equations A and B. Figures 9 and 10 present the predictions from equations E & F respectively against the measured values. The data mostly fall within ± 5% bounds.

Tab. 2. Results of regression analyses with additional parameters

Equation	Results of regression analyses	n	R^2	S.E.
E	γ_t= 5.177-0.5294log(f_s)+4.321log(q_t)-2.363log(z) + 0.6674log(Δu_2)	194	0.71	0.39
F	γ_t/γ_w= 1.377+0.0697log(R_f)+0.0.1971log(q_t/P_a) +0.144log($\Delta u_2/u_0$)	195	0.677	0.0413
G	γ_t = 10.76-0.4376log(f_s/z)+1.3log(q_t/z) +1.716log(Δu_2/z)	195	0.498	0.515

Note: q_t, P_a , Δu_2, u_0 and f_s are in kPa, z in m and γ_t and γ_w in kN/m^3

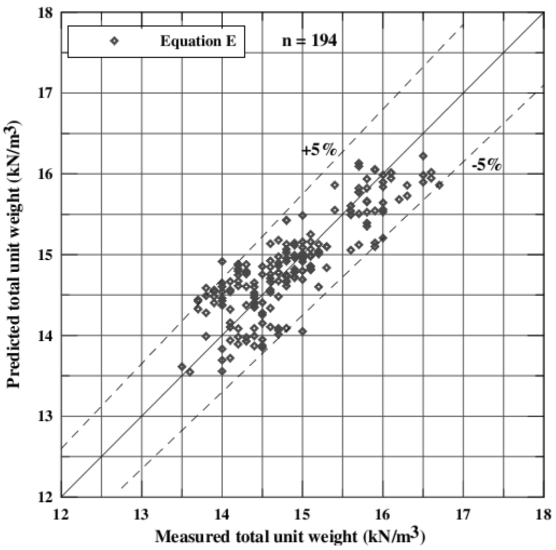

Fig. 9. Predictions – Equation E

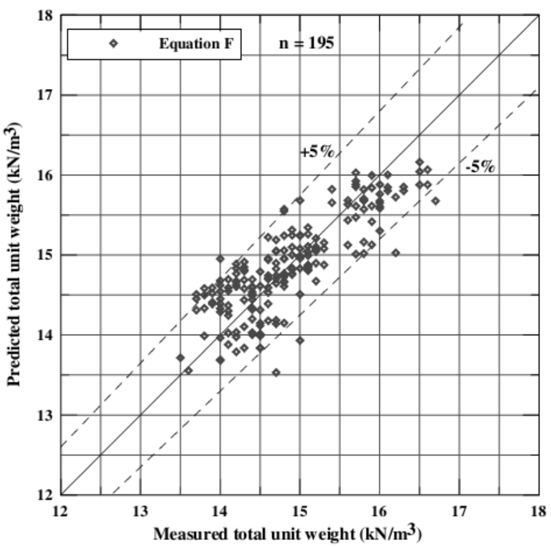

Fig.10. Predictions – Equation F

Discussion and recommendations for application in the field

From the results presented in the previous sections, it is observed that equations A, E and F all show comparable strengths. Despite problems with accuracy of measured CPT data for very soft to soft clays the strengths of relations are found to be reasonable. From the analyses presented on the limited data, it appears that excess pore pressure; Δu_2 may be an important parameter in CPT based unit weight correlations for very soft to soft clays. With additional data from future soil investigation in western Indian offshore this aspect needs to be further investigated. However, presently, equation A is recommended for application in western Indian offshore for estimating unit weights of very soft to soft calcareous fine grained soils.

To apply the relationships in the field will require identification of soils as silty clays/ clayey silts without the use of classification systems which require estimate of overburden pressure. This can either be done by simply observing the measured parameters and their variation with depth or by using non-normalized SBT chart proposed by Robertson (2010). To check if this chart identifies the soils in a similar way as Robertson & Wride (1998) system, data were plotted on Robertson (2010) non-normalized SBT chart and is presented in Figure 11. It can be seen that the soils are identified as fine grained, silty clay or clayey silt.

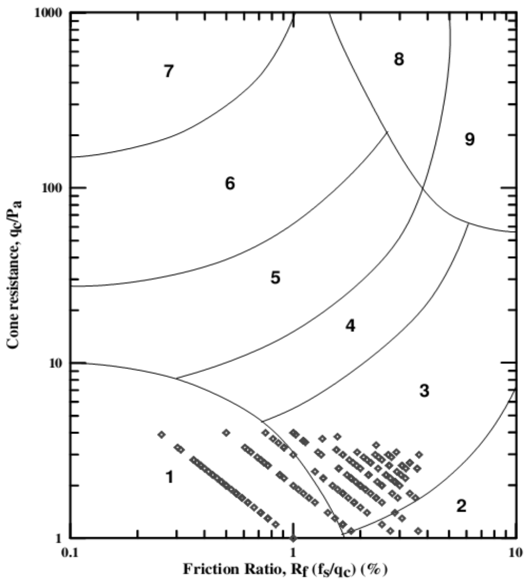

Fig.11. Data on Robertson (2010) chart

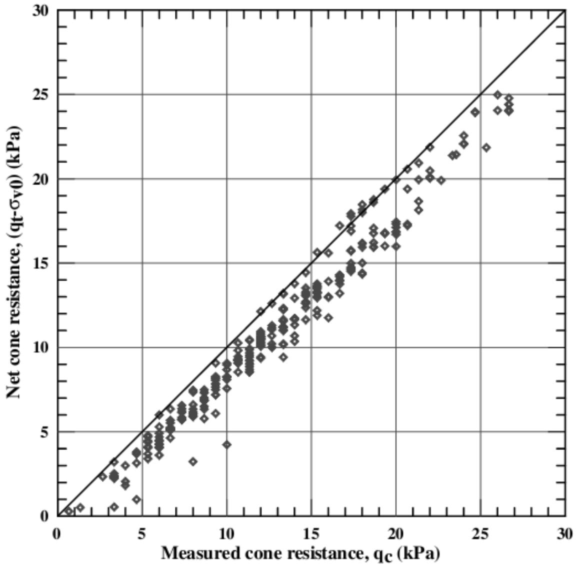

Fig.12. Comparison of q_c and q_{net}

The consistency of the clays also needs to be ascertained (as very soft to soft) before applying the identified correlation. Before the use of CPTU was introduced in Indian offshore, a direct correlation with uncorrected cone resistance (q_c) was being used to estimate undrained shear strength with 15 to 20 as the range of cone factors. A comparison of uncorrected and corrected cone resistance for the data used in this study is shown in Figure 12. Since this approach does not require estimate of overburden pressure, it is recommended for application in western Indian offshore using a cone factor of 15.

Conclusions

The data of calcareous clayey/silty very soft to soft soils from western Indian offshore locations have been used to evaluate applicability of four reported relationships between CPT measured parameters and unit weight. It is observed that two of the relationships i.e. by Mayne et al. (2010) and Robertson & Cabal (2010) generally predict within 10% of the measured total unit weight values although the scatter is high.

The results of attempts to refine the relations further by using multivariable regression analyses show that the best correlation for application to western Indian offshore calcareous clayey/silty very soft to soft soils is obtained involving the same parameters as those used by Mayne et al. (2010). It appears that Δu_2 may be an important parameter for such soils. However, presently, equation A is recommended for application in western Indian offshore for estimating unit weights of very soft to soft calcareous fine grained soils. The relevant soil types can be identified by either observing the measured CPTU parameters or through use of Robertson (2010) non-normalized SBT classification chart which does not require estimate of overburden pressure. As more and more data become available from future investigations in western Indian offshore, the relationship will have to be further examined and refined especially by involving parameter Δu_2.

It is also suggested that cone type used to gather CPT data should be identified and taken into account while proposing new correlations or applying existing correlations involving sleeve friction terms.

Acknowledgement

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Application of CPTU and SDMT to the characterisation of Irish silts

Michael Long
University College Dublin, Ireland

Roselyn Carrol
Norwegian Geotechnical Institute, Norway – formerly University College Dublin, Ireland

Abstract. Due to the increased pressures to develop marginal low lying sites and the engineering problems encountered when dealing with these materials in the past, there are significant economic benefits in understanding the characteristics of natural silts. This paper outlines the geotechnical characteristics of the two Irish silts, which although geologically very similar, have quite different geotechnical characteristics. The focus is on the use of in situ testing namely SDTM and CPTU to help characterise the materials and to provide data for 1D settlement analyses. Both the CPTU and SDMT "soil behaviour" charts work well for the materials with a chart recently developed for use of the CPTU in "intermediate" materials appearing particularly useful. Given the nature of the material behaviour it was found that data from the in situ testing was very useful in checking / augmenting data from laboratory tests. Measured settlements were much less than those initially predicted. This was mainly due to underestimation of the preconsolidation stress. A focus of any future projects should be accurate determination of this parameter using both laboratory and field techniques.

Key words: soft soil, silt, 1D compression, in situ testing; laboratory testing

Introduction

Deposits of soft silts are frequently found in low lying areas especially along rivers and in estuaries. Recent pressure on land space, especially in urban areas, has given new impetus for engineers to develop an understanding of the geotechnical behaviour of these materials. Frequently engineers make use of methods and techniques derived for clays or sands when working with silt but these may be inappropriate for intermediate materials such as silt. It is also well recognised that sample disturbance effects (both destructuration and densification) can be very significant in silts. Therefore use of in situ tests such as CPTU and DMT to both characterise silts and to give estimates of design parameters can be particularly useful. In this paper the focus is on the frequently encountered practical problem of 1D compression. Many natural silts are lightly overconsolidated and loading is often modest due to site filling or construction of access roads. Stresses are therefore mainly within the orverconsolidated zone and in the zone around the preconsolidation pressure (p_c') and therefore estimates of settlement are not straightforward. In order to attempt to address some of these issues detailed characterisation of two natural silt sites in the mid-west of Ireland are given. Work at the sites included CPTU and SDMT testing. Records of a trial filling at one of the sites are given. The overall aim is to provide guidance for engineers working in similar materials.

The Sites

The location of the two sites under study is shown on Figure 1. Both sites are located adjacent to the River Shannon; Foynes to the south and Shannon Town Centre to the north and are underlain by estuarine deposits associated with the river. The Foynes site together with two other Irish silt sites has been studied in detail by Carroll (2013). Investigation of the Foynes site was in association with the development of the Foynes Harbour Access Road. At Shannon a large housing scheme, together with associated access roads and other services was planned. Loading imposed on the silts was modest at both locations.

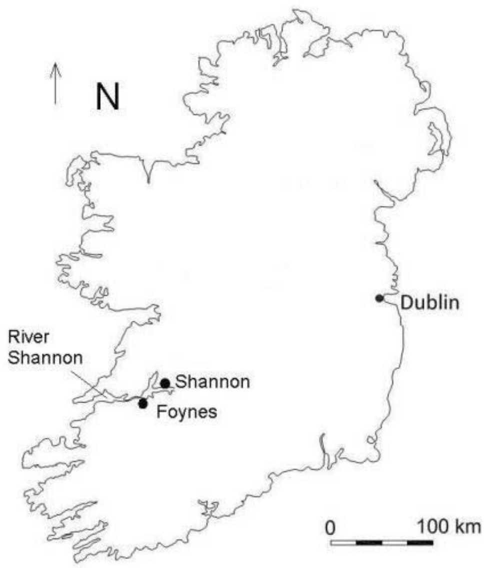

Fig. 1. Location of study sites

Ground conditions and soil index properties

The Foynes site is underlain by two distinct layers an upper sandy silt and a lower clayey silt, see Figure 2a. The transition between the two units is between 5.5 m and 6 m depth. These layers are up to 13 m thick in total and are underlain by Carboniferous limestone bedrock. For the upper layer the bulk of the data showed water content (w) ranged between 30% and 43%. Below 5.5 m w was relatively constant at about 28% but showed a slight increase with depth. Bulk density (ρ_b) was scattered in the depth range of 1.5 m and 4 m. Below 4 m there was a more consistent profile for ρ_b and in the lower layer ρ_b was roughly 1.87 Mg/m^3. There was a slight reduction in ρ_b with depth which was consistent with the slight increase in water content. The clay, silt and sand contents showed only a slight change with depth indicating that the upper layer samples had roughly 10 % to 15 % clay while the lower layer samples have between 13 % and 25 % clay.

The bedrock in the Shannon area also comprises Carboniferous Limestone. Overlying the rock is competent boulder clays. However above the boulder clay the superficial deposits are complex in nature. As well as a general accumulation of salt water saturated estuarine silts, river valleys appear to have been cut into the boulder clay and in filled with unconsolidated deposits which are generally peat rich accumulations. These buried valleys appear to be up to 14 m deep. The water content and bulk density values, shown on Figure 2b, confirm the complexity of the superficial deposits. The general (silty) body of the material has water content of some 50% to 60%. However there appears to be frequent zones where the material is highly organic or peat. At these locations water contents of 300% to 400% were measured. There seems to be a particularly peaty zone around 2 m to 5 m. Bulk density values are typically 1.6 Mg/m^3, but lower values, of about 1.2 Mg/m^3, were measured in the peaty material and higher values up to 2.2 Mg/m^3 in the sandy zones. Although the data is relatively limited the percentage clay is on average 15% and the silt is on average 80%.

Particle size distribution curves for Foynes and Shannon are shown on Figure 3a and 3c respectively. The Foynes silts are relatively uniform medium to coarse silt. It is clear from the particle size distribution curves that the upper silty layer is coarser than the lower clayey layer. The Shannon silt is also mostly a medium to coarse silt. A typical curve for a sandy layer is also shown and it can be see that this material is fine to coarse.

Atterberg limit test results, given on Figures 3b and 3d, show the values straddle the “A” line as has been found previously for silts. However the plot clearly distinguishes between the upper sandy silt and lower clayey silt at Foynes. The Shannon silts are generally more plastic and are characterised as being of “high” plasticity. The higher plasticity values are due to the higher organic content of the material.

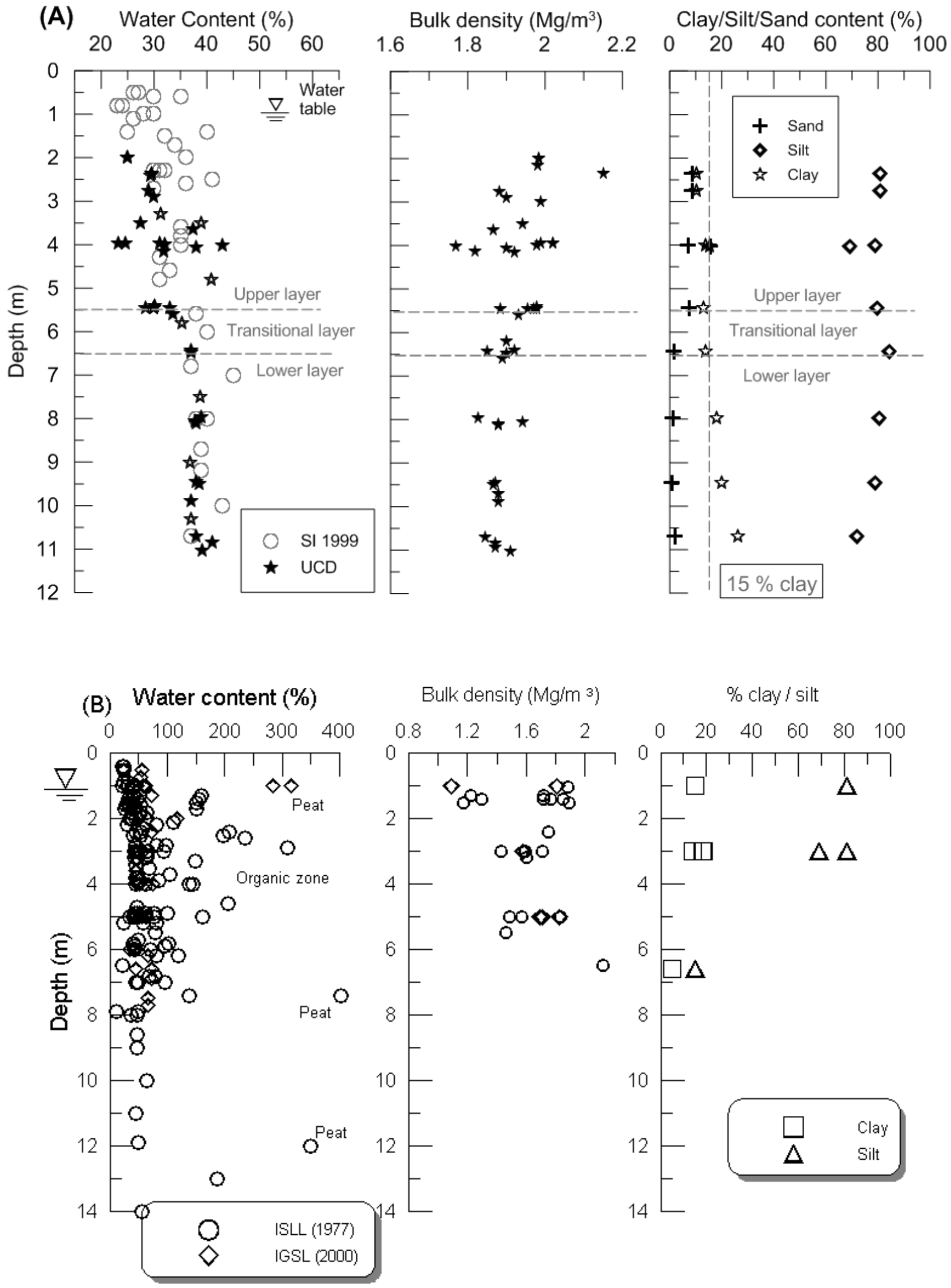

Fig. 2. Stratigraphy and index properties at (a) Foynes and (b) Shannon

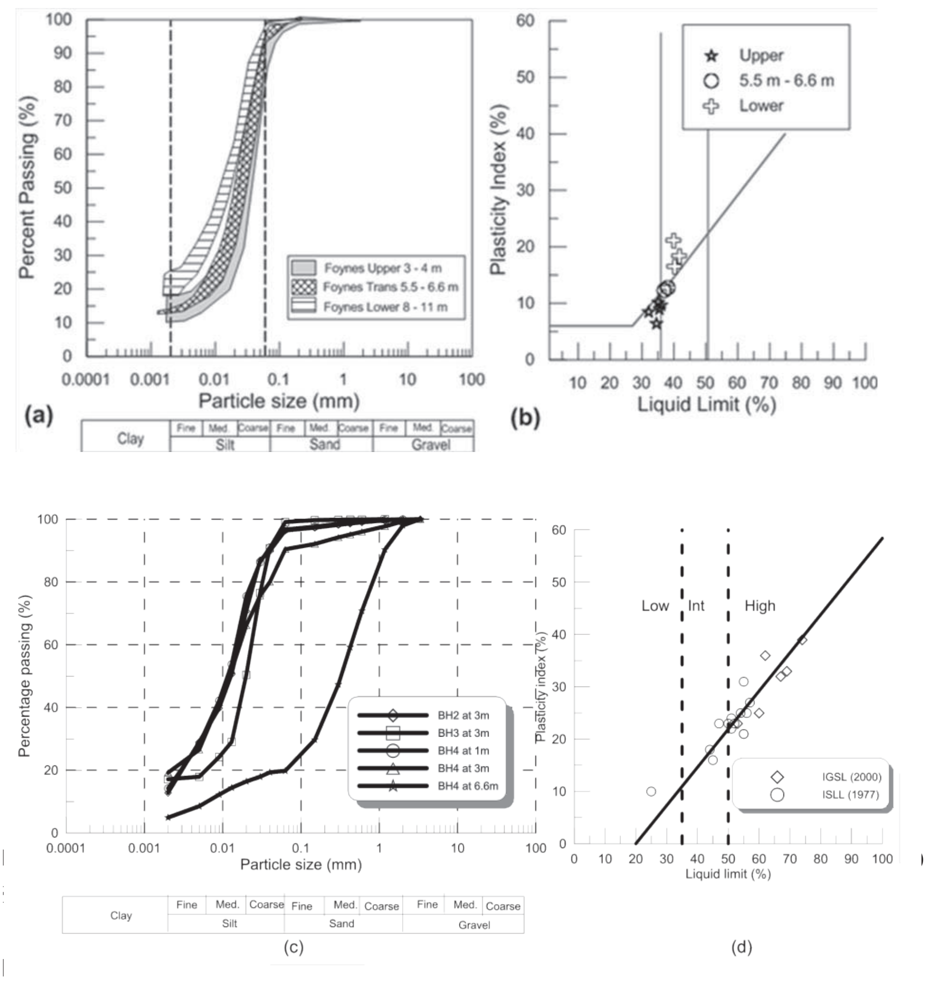

Fig. 3. Results of particle size distribution analyses (a and c) and plasticity chart (b and d) at Foynes and Shannon respectively

CPTU testing at Foynes

CPTU and SDMT testing was carried out at Foynes only. A total of three CPTU tests were carried out by In Situ Site Investigations Ltd., and the results are presented in Figure 4. There is generally good consistency between the three sets of tests. At 2.5 m q_t (corrected cone end resistance) was at a peak of 4 MPa in the upper sandy silt layer and reduced consistently with depth to about 0.5 MPa at 5 m. From 5.5 m to 6.5 m the transition in soil behaviour is evident at which point q_t reached a minimum value of less than 0.5 MPa. At 6.5 m there was a clear start of the lower clayey silt layer as q_t showed a steady increase with depth and is equal to approximately $5\sigma_{vo}$' (in situ vertical effective stress).

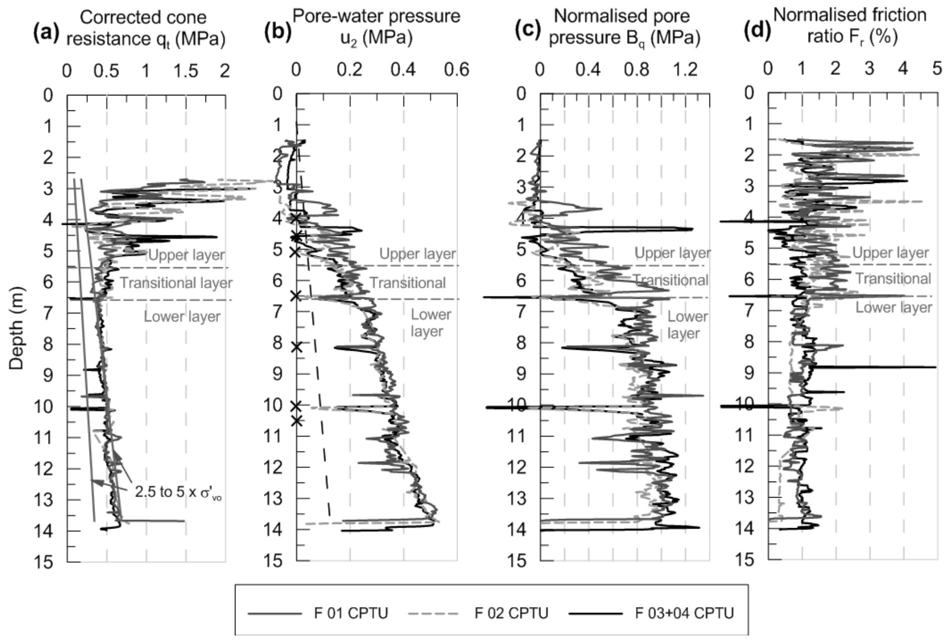

Fig. 4. Results of CPTU tests at Foynes

The u_2 (generated pore pressure) profile was close to hydrostatic until about 4 m after which point there was an increase in the excess pore pressure generated with increasing depth. Again transitional behaviour is evident between 4 m and 5.5 m. The corresponding B_q values were zero in the upper 4 m and an increase to 0.4 between 4 m and roughly 6 m. The X symbol on Figure 4b indicates locations where dissipation tests were carried out.

The F_r (normalised friction ratio) values in the upper sandy silt layer showed a range between 1 % and 2 %. In the lower clayey silt F_r values were close to 1 % consistently.

According to Robertson et al. (1986) or Schnaid et al. (2004) the B_q profile in the upper sandy silt is indicative of a drained penetration progressing to a partially drained penetration as B_q increases from 0 to 0.4. Tests F01 and F02 were run at the standard rate of 2 cm/s. In order to investigate the effects of partial drainage more closely Test F03+04 was run at a faster rate of roughly 11 cm/s between a depth of 2.2 m to 4.3 m. However no clear change in the measured parameters is evident.

SDMT testing at Foynes

Results of SDMT testing are presented on Figure 5 in the form of the two measured pressure parameters p_0 (“lift off pressure”) and p_1 (recording at 1.1 mm expansion of membrane) (Figure 5a) and the shear wave velocity (V_s) (Figure 5b). Two tests were carried out and gave very similar results. Values of p_0 and p_1 are generally higher and less uniform in the upper sandy silt while in the lower clayey silt they increase steadily with depth.

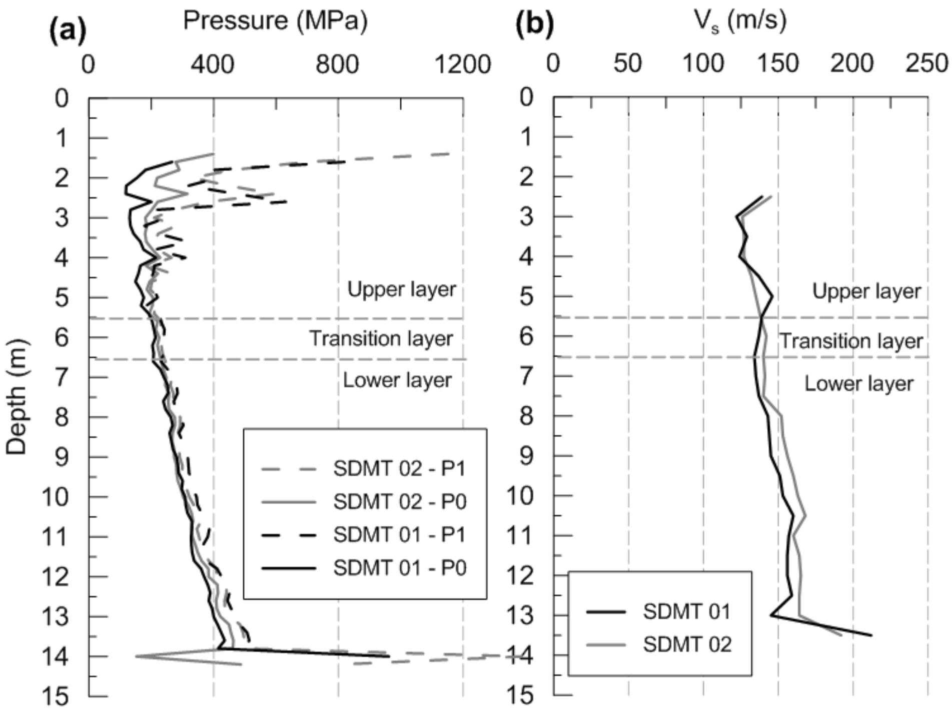

Fig. 5. SDMT test results – measured parameters (a) p_0 and p_1 and (b) V_s

Measured V_s values were typically 130 m/s and 150 m/s for the upper sandy silt and lower clayey silt respectively and these values are similar to those presented for the Os and Halsen silts in Norway by Long and Donohue (2007).

The secondary or derived SDMT parameters namely I_D, K_D and E_D, material index, horizontal index and dilatometer modulus respectively, are shown on Figure 6. Marchetti (1980) defines these parameters as:

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \tag{1}$$

$$K_D = \frac{p_0 - u_0}{\sigma_{v0}'} \tag{2}$$

$$E_D = 34.7(p_1 - p_0) \tag{3}$$

where:
u_0 = in situ pore water pressure
σ_{v0}' = in situ vertical effective stress

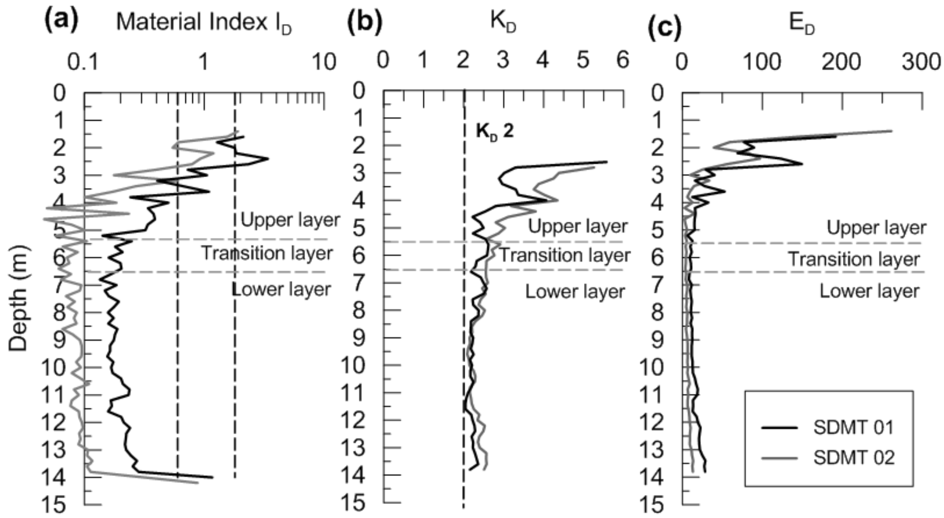

Fig. 6. Secondary SDMT parameters (a) I_D, (b) K_D and (c) E_D

The presence of the two distinct layers is clearly evident from the tests results, especially using I_D. There was a slight difference between I_D for SDMT tests 01 and 02. It is noted that the scale of results are in log form for this parameter. In general K_D and E_D are similar with depth for each of the tests performed. For a normally consolidated clay $K_D \approx 2$. K_D was somewhat greater than 2 in the upper layer and very close to 2 in the lower layer. Both the SDMT initial pressures and 'intermediate' stress parameters suggested the transition between upper and lower layers was slightly shallower than indicated previously. However below 5.5 m the behaviour of the material was uniform with depth.

Material characterisation from CPTU and DMT

Carroll et al. (2012) have presented a detailed analysis of the application of various "soil behaviour" charts to characterise the Foynes soils. An overview of the findings of these authors is given here.

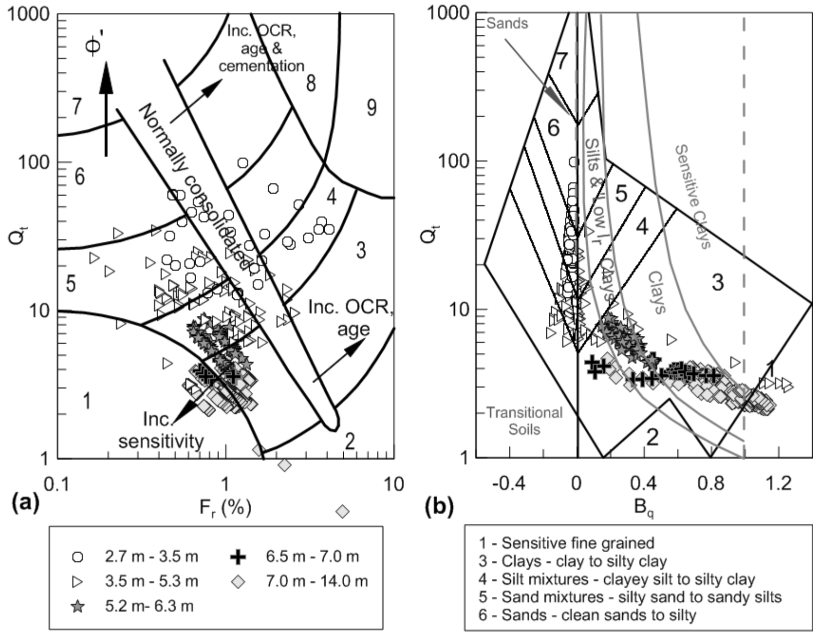

Fig. 7. Foynes CPTU data plotted on Robertson (1990) soil behaviour charts (a) Q_t versus F_r and (b) Qt versus B_q.

The Foynes CPTU data are plotted on the well know "soil behaviour" charts of Robertson (1990) on Figure 7. These charts involve plots of Q_t against F_r and Q_t against B_q. Despite some scatter in the data the charts clearly identify the presence of two distinct layers. The upper sandy silt data generally falls in Zones 4 and 5, i.e. silt mixtures or sand mixtures. There is considerable scatter in the normalised friction (F_r) data. The lower clayey silt data fall in Zones 1 or 3, i.e. sensitive fine grained or clay soils. The charts also successfully identify the presence of the transitional layer.

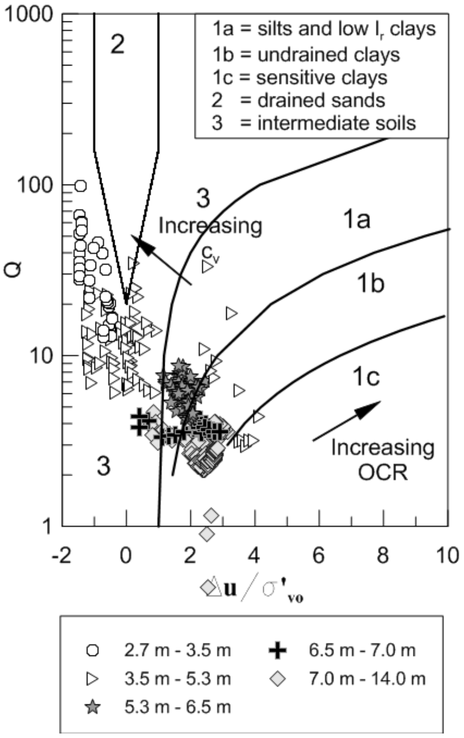

Fig. 8. Foynes CPTU data plotted on chart of Schneider et al. (2008).

Schneider et al. (2008) developed some charts designed to specifically deal with "intermediate" soils for example silts which may show drained or partially drained behaviour on cone penetration. It involves studying a plot of Q_t versus $\Delta u/\sigma_{v0}$'. The Foynes data are plotted on this chart on Figure 8. The chart works well for the site. The data for the lower clayey silt generally falls in Zones 1b, i.e. undrained clays. The transitional soils are classified as silts and low rigidity index clays (Zone 1a), while the lower sandy silts are classified as intermediate soils (Zone 3). Arguably this chart is more useful than that of Robertson (1990) as it is cleaner and easier to use as it has fewer zones.

It is also possible to superimpose the zones from the Schneider et al. (2008) onto Robertson (1990) chart for Q_t versus B_q, as shown on Figure 7b. This can help with identification of intermediate/partially drained soils.

The SDMT data in the form of a plot of the intermediate parameters I_D and E_D are plotted on the soil behaviour type chart proposed by Marchetti and Crapps (1981) on Figure 9. Overall there is a trend showing data points moving from top right to bottom left with depth on the site, indicating a full range of soil types from silty sand to "mud".

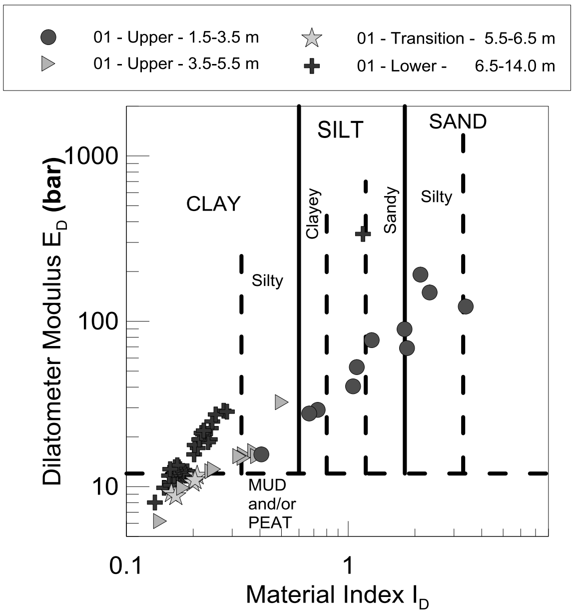

Fig. 9. Foynes SDMT data plotted on chart of Marchetti and Crapps (1981)

This reflects the transition from the upper silty sand layer to the lower silty clay layer. Hence the SDMT captured the stratigraphy of the site well. It seems however that the SDMT identified some of the upper and transition layer soils as more clayey than suggested by the index results.

Table 1 presents a comparison of the soil behaviour types identified using the CPTU and SDMT. In general all of the techniques work well and all provide useful data. It is clear that no single method should be used in isolation and a combination of several methods should be used at each new site.

Tab. 1. Comparison of soil behaviour types using different methods

Layer	Robertson (1990)	Schneider et al. (2008)	Marchetti and Crapps (1981)
Upper sandy silt	Sand / silt mixture	Transitional soil	Graduating from silty sand, through silt to mud
Transitional	Silt mixture / clay	Silts and low Ir clays	Mud and or peat
Lower clayey silt	Sensitive fine gra-ined or clay soil	Clays	Graduating from clay to mud and / or peat

Oedometer test results

The results presented here are based on laboratory work on the best available samples. Sampling was carried out using the 1 m long ELE 100 mm diameter fixed piston sampler, which has a 1.7 mm wall thickness. Studies by various authors such as Hight (2000) and Long (2006) have shown that sample quality is dependent on the sampler cutting edge angle. Therefore samples were taken with a modified version of this sampler, in which the sample tube cutting edge was sharpened from the normal 30° to about 10°.

For silts data can be plotted in the classical Terzaghi format, i.e. log of vertical effective stress (σ_V') against axial strain (ε) format or using the resistance concepts of Janbu (1963). It has been shown by several other authors that the log σ_V' versus ε curve for silts can be very flat with no obvious evidence of a change in stiffness at the preconsolidation stress (p_c') and therefore the alternative method of analysis may be useful.

Data for the Shannon site are shown on Figure 10 using both techniques, i.e. log σ_V' versus ε, constrained modulus ($M = \Delta\sigma_V'/\Delta\varepsilon$) against σ_V' , coefficient of consolidation (c_V) against σ_V' and finally creep (in this case C_{sec} = strain / log cycle of time) versus σ_V' . The in situ vertical effective stress (σ_{V0}') is also shown on the plots.

In the log σ_V' v ε , plot the stress – strain behaviour is initially reasonably linear and then the strain accumulates much more rapidly after p_c'. For the 1 m test it would seem that p_c' is at least 30 kPa (σ_{V0}' = 17 kPa). For the 3 m test it seems p_c' exceeds 40 kPa (σ_{V0}' = 31 kPa). For the 5 m tests p_c' may be close to σ_{V0}' (= 45 kPa). These results suggest that the material can then sustain a modest loading (of say 10 kPa to 20 kPa) without excessive settlements occurring.

The plot of constrained modulus shows a gradual build up in M with increasing σ_{V0}'. This is characteristic for silts as the stiffness increases in response to increased inter-particle pressure between the silt grains. For clays Janbu (1963) suggested that values of c_V would be high in the overconsolidated range, drop to a minimum at p_c' before gradually rising again with increasing σ_V'. Similarly Janbu (1969) suggested C_{sec} should be very low before p_c' rise to a maximum at p_c' and then decrease gradually with increasing stress. No such pattern for c_V or C_{sec} is evident here. The values remain essentially constant.

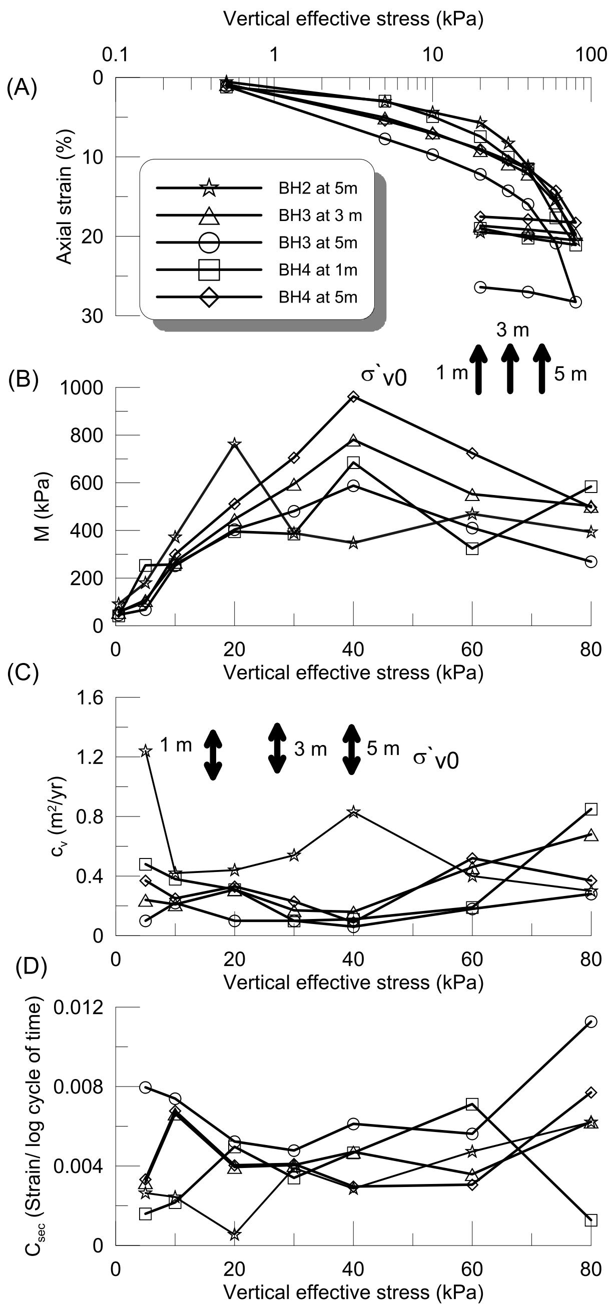

Fig. 10. Oedometer tests for Shannon presented in (a) conventional log σ_v' versus ε format (b) M versus σ_v', (c) c_v versus σ_v' and (d) C_{sec} versus σ_v'

1D compression parameters from CPTU and DMT
Overconsolidation ratio (OCR)

Carroll (2013) gives a detailed account of the determination of p_c' from oedometer tests in silt. Values of overconsolidation ratio (OCR), for the Foynes sites, determined using the techniques of Casagrande (1936), Janbu (1969) and from the "work" method of Becker et al. (1987) are compared to values obtained from seismic dilatometer tests (SDMT) on Figure 11a. These latter values are obtained from the empirical expressions derived by Marchetti (1980). Note that $K_D \approx 2$ corresponds to normally consolidated material:

$$OCR_{DMT} = \left(0.5K_D\right)^{1.56} \tag{4}$$

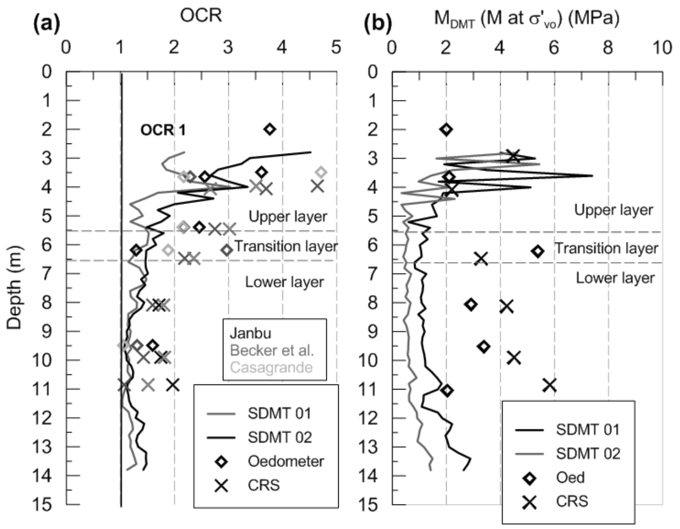

Fig. 11. 1D compression parameters from SDMT for Foynes (a) OCR and (b) M_0.

There is considerable scatter in the oedometer test data for the upper sandy silt layer. The scatter reduces with depth in the lower clay silt material. Broadly the trend from the lab tests follows that of the SDMT and suggests that the OCR reduces from about 3.5 at about 3 m depth to values close to 1.0 towards the bottom of the sequence (corresponding to $K_D \approx 2$, see Figure 6b).

It is also possible to empirically estimate OCR from CPTU results by comparing qt with lines representing 2.5 σ_{v0}' and 5 σ_{v0}' (Figure 4). Lunne et al. (1997) suggest that if q_t is greater than 2.5 σ_{v0}' to 5 σ_{v0}' then the material is likely to be overconsolidated. Again this suggests that the Foynes material below about 6 m is close to being normally consolidated. Mayne (1991) also suggested an expression for deriving OCR in clays from CPTU results based on cavity expansion and critical state theory. This appears to underestimate OCR for the lower silty clay at Foynes (Figure 12a).

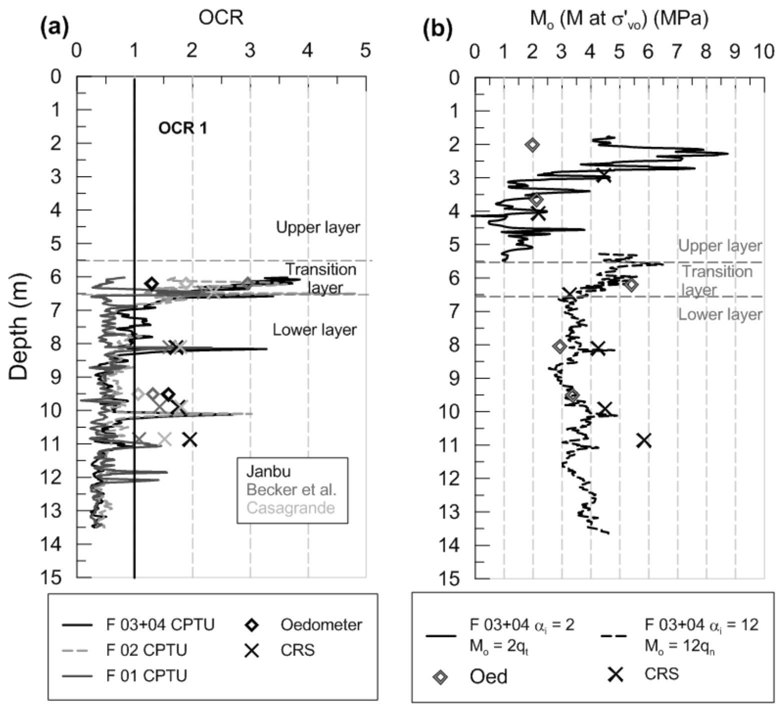

Fig. 12. 1D compression parameters from CPTU for Foynes (a) OCR and (b) M_0.

Constrained modulus at in situ stress (M_0)

The constrained modulus at in situ stress (M_0 at σ_{v0}') is a key soil property for estimating settlement in modestly loaded, lightly overconsolidated materials, such as those at Shannon or Foynes. M_0 can be estimated from SDMT results using the expression:

$$M_{DMT} = R_M E_D \tag{5}$$

where:

R_M = correction factor = $0.14 + 2.36 \log K_D$ if $I_D < 0.6$ (6)

M_{DMT} values are very similar to those obtained from oedometer testing in the upper sandy silt (Figure 11b) but underestimate the oedometer values in the lower clayey silt. This is because of the low E_D values determined for this material (Figure 6c)
Similarly M_0 can be determined from CPTU tests using the empirical correlation of Senneset et al. (1988):

$$M_0 = \alpha q_t \tag{7}$$

where:
$\alpha = 2$ if $q_t < 2.5$ MPa

It can be seen from Figure 12b that a good fit for the upper sandy silt can be obtained if α is chosen as equal to 2 as suggested by Senneset et al. (1988) but a much larger value of 12 needs to be used to obtain a good fit in the lower clayey silt.

Field settlement data for Shannon

Carroll (2013) presents a detailed analysis of the settlement data at Foynes. Here the focus is on the similar data recorded for a trial fill at Shannon. Although only modest loading due to filling for site access roads and hardstandings was proposed at Shannon there was significant concern about the likely settlements given the thick sequence of compressible organic silts present on the site. Settlements of the orders of several hundred mm were predicted assuming normally consolidated conditions pertained at the site
Therefore a trial loading comprising 1.5 m of fill ($\sigma_v' \approx 27$ kPa) placed over an area of some 13 m x 13 m was constructed in 2005, see Figures 13. The fill was placed on the natural ground having first stripped the topsoil.

Fig. 13. Trial filling at Shannon

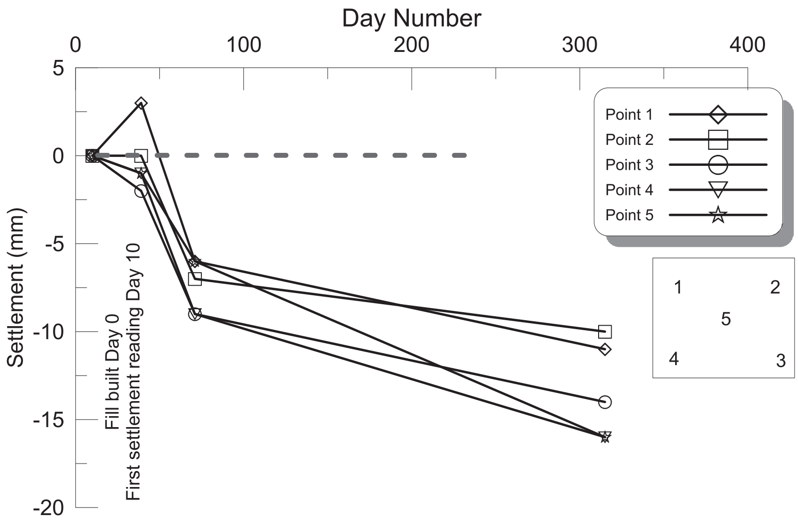

Fig. 14. Trial loading at Shannon

The measured settlements are shown on Figure 14. Although no data is available for the first 10 days of settlement, it can be seen that the measured settlements are very modest and did not exceed 16 mm. The borehole nearest to the trial shows approximately 6.5 m of soft compressible material over limestone bedrock. Therefore the strain in this stratum due to the filling was some 0.25%. It is clear that the loading imposed did not exceed the preconsolidation stress of the material and hence the modest settlements encountered.

Backanalysis of field settlement

It is difficult to backanalyse M_0 but from the data above it is roughly 10MPa which is much greater than that measured in the oedometer tests (Figure 10b) and suggests significant sampling disturbance effects occurred. Creep settlement seem to suggest C_{sec} is of the order of 0.0025 strain/log cycle of time which is at the lower bound of the values measured in the oedometer tests (Figure 10d)

Conclusions and recommendations

- Detailed characterisation of two silty materials from two sites in Ireland is given here. Although the sites are geologically very similar the geotechnical characteristics of the material are significantly different.
- In Ireland the focus of many practical projects in silty soils is in the characterisation of the materials and in the assessment of settlement due to relatively modest loading.
- Both the CPTU and SDMT "soil behaviour" charts work well for the materials at Foynes where two distinct layers were observed.
- The chart developed by Schneider et al. (2008) specifically for use of the CPTU in "intermediate" materials appears to be particularly useful.
- Given the "rounded" nature of the stress – strain plots for oedometer testing in silts, it is recommended that data are analysed using several schemes such as the classical Terzaghi approach and that of Janbu as utilised here.
- As preconsolidation stress is a key parameter in settlement analyses, no single technique should be used for obtaining the design value. Again data from both the SDMT and CPTU can be very useful in checking / augmenting data from laboratory tests.
- Settlements measured at the Shannon site were much less (perhaps an order of magnitude) than those expected from oedometer testing. This was particularly due to underestimation of p_c' but also M_0 and the overestimation of C_{sec} from the laboratory tests probably due to sample disturbance effects. In situ testing is therefore important to support any laboratory test data.

Acknowledgement

The authors are grateful to Mr. Darren Ward of In Situ Site Investigations Ltd., who carried out the field testing at Foynes and to Mr. Caimin Jones of Thomas Garland and Partners who provided the data for the Shannon site.

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Subsoil characteristics of the Vistula river dikes

Lech Bałachowski, Zbigniew Sikora
Gdańsk University of Technology, Faculty of Civil and Environmental Engineering, Poland

Abstract. The results of CPTU tests are used to determine the strength parameters of a dike subsoil. Stress state and stress history of the subsoil under the flood embankment was evaluated with CPTU and DMT tests. Normally consolidated soil was found under the central part of the dike, while the subsoil near the dike toe is found to be overconsolidated. Due to consolidation the undrained shear strength of the subsoil under the central part of the dike is generally higher than near the dike toe. The undrained shear strength estimated with CPTU tests depends also on the complex phenomena of the filtration under the dike. The CPTU dissipation tests in soft soils and the monotonic decay of water pressure induced during penetration under the central part of the dike confirm the presence of normally consolidated subsoil. Dilatory pore pressure response, typical for overconsolidated soils, was mainly observed in the tests performed at the toe of the dike at the waterside and downstream side. The shape of pore water dissipation curve reflects stress state and stress history of the subsoil.

Key words: CPTU, DMT, undrained shear strength, stress history, soft soil, dissipation test

Introduction

Cone penetration test with pore pressure measurements (CPTU) together with dilatometer test (DMT) permit a reliable and comprehensive evaluation of soft soil parameters including strength and deformation characteristics (Mayne, 2001). The dissipation test completes the complementary character of the both tests with a measure of filtration and consolidation parameters. As the CPTU penetration in less permeable soils is held in undrained conditions, the pore pressure is generated around the probe. One can monitor the dissipation of the induced pore pressure to the hydrostatic one (Sikora et al., 2004). Two shapes of the dissipation curve, i.e. monotonic decay or dilatory pore pressure response are recorded, related to the soil stress state and history. A series of CPTU and DMT were performed for renovation of the dikes in the mouth of the Vistula river (Bałachowski, 2008). The improvement works considered the reconstruction of drainage system, local access roads and the construction of vertical grouted curtain in the dike body and the subsoil. The grouting reinforces the central part of the dike corps, improves the overall dike stability and decreases the amount of seepage. Some other results of dike and subsoil improvement with hybrid methods for the dikes in the delta of the Vistula river are given in Ossowski, Wyroślak, 2012.

Subsoil characterization with CPTU and DMT

CPTU and DMT soundings were performed at the crest and at the waterside and downstream toe of the dike in order to characterize its corps and the parameters of the soft subsoil. The dike itself is generally built from silty soils, like sandy silts, silty or sandy clay, clayey silts or sands, silty sand and silts, and it can contain also some clay layers or fine sand. The height of the embankment in the analysed section is about 7,0 m. In the subsoil, one can distinguish superficial organic clayey layer, covering sandy or silty mud and a local peat layer. Some sandy interbeddings can be found in this mud layer as well. Fine and medium dense sands deposits, at larger depths, form a continuous well permeable layer. An example of the geotechnical cross--section is given in Fig. 1. The penetration tests were performed with Geotech 220 rig, property of Gdańsk University of Technology. In this paper, mainly the results of CPTU analysis are presented. The soundings were performed from the crest of the dike and at the toe from the waterside and the inner slope. Fifteen cross-sections of the dike were analysed.

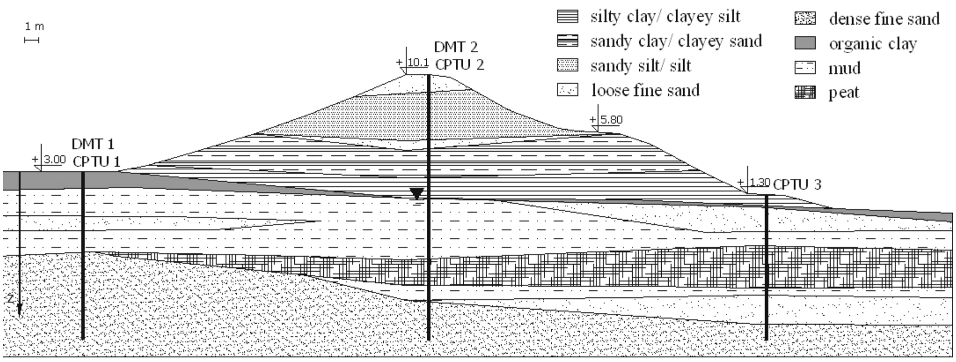

Fig. 1. The dike and its geotechnical cross-section (Bałachowski, 2006b)

The attention was placed on both the strength and the stress state in the subsoil as well as on the flow characteristics of the soft organic layers. The compaction state of the embankment itself, its permeability and shear strength are out of the scope of this paper. Standard CPTU probe with the pore pressure u_2 measurement was used. Values of the corrected cone resistance q_t, the normalized friction ratio F_r and the pore pressure measurements u_2 recorded in the dike subsoil from the crest and at the toe from the waterside and downstream side are given in Fig. 2. Here, the measurements in the core of the embankment were omitted and the results are presented from the same reference level, i.e. the elevation of the dike toe at the upstream side. For the test at the downstream side the measurements from 1,7 m below this reference level are given. For a given layer under the central part (crest) of the dike higher cone resistance is measured than near the toe. Due to crust phenomena one can already notice a cone resistance (from 0,5 to 1,8 MPa) mobilized in clayey layers from the very beginning of penetration. The pore pressure measurements are influenced by the local sand inclusions. In the less permeable zones, the pore overpressures are mobilized, especially under the central part of the dike. Filtrated CPTU data are presented in Fig. 2. The corrected cone resistance q_t and the friction ratio F_r together with the results of borings were used to delineate the soil layers within the subsoil. While the data filtration can be useful for the determination of the average soil parameters it could however mask some important details of the soil structure. In very heterogeneous alluvial soils the presence of local inclusions of more permeable material will be even more relevant to the analysis of dissipation tests (Marchetti et al. 2004) as the laminations will considerably change the soft soil response during dissipation test. A careful analysis of the non-filtrated data is thus essential in the proper analysis and in the interpretation of CPTU dissipation curve. A series of CPTU dissipation tests was performed in the mud under the central part of the embankment and at the upstream side and the downstream one.

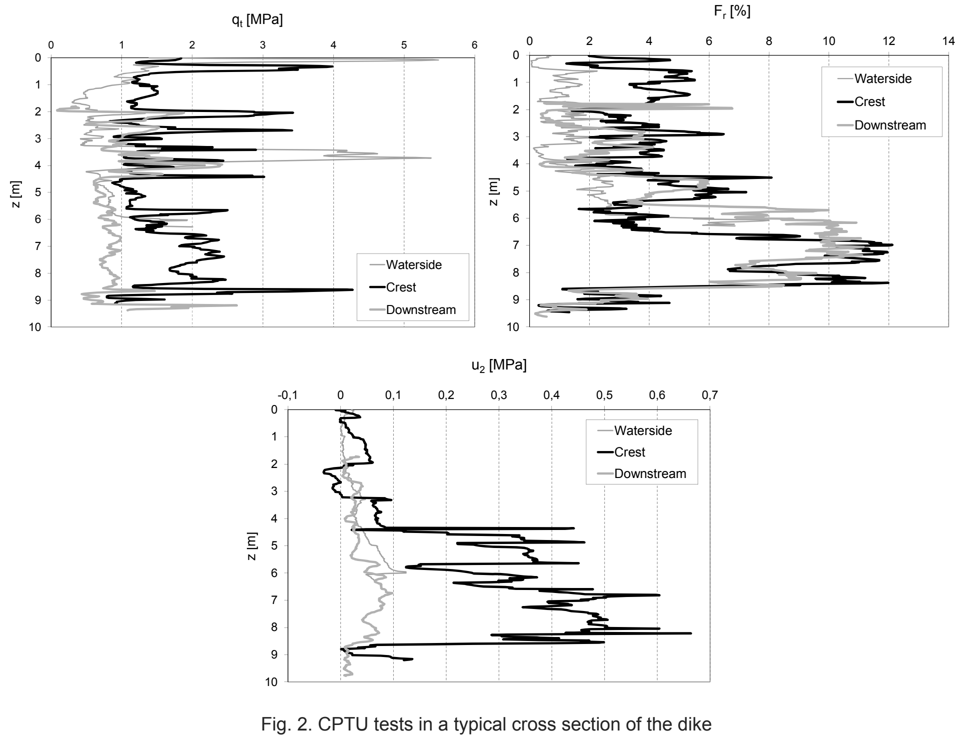

Fig. 2. CPTU tests in a typical cross section of the dike

Stress state and history

Stress state and history in the soft subsoil under the dike were evaluated with CPTU and DMT data. Soil overconsolidation can be determined from CPTU data according to formula (Lunne et al. 1997):

$$OCR = \frac{a(q_t - \sigma_{v0})}{\sigma'_{v0}} \tag{1}$$

where:
a – coefficient from 0,2 to 0,5 related to plasticity index,
q_t – corrected cone resistance,
σ_{v0} – initial total overburden stress.

According to Mayne et al. 1998 the coefficient α of a given clay layer decreases with plasticity index and void ratio. Lunne et al. 1997 suggest the average value of the coefficient a to be 0,3 and this was assumed in the present calculation.
For the soft soils under the embankment the *OCR* values slightly exceeding 1 were found from CPTU results (Fig. 3). At the embankment toe the *OCR* values vary mainly from 4 to 6. At small depth even higher *OCR* values are obtained, which could be related to crust phenomena and dessication.

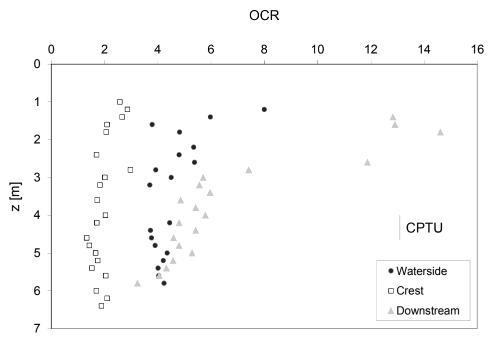

Fig. 3. Overconsolidation ratio of the dike subsoil from CPTU

The stress history was also evaluated with DMT tests. The lateral stress index K_{DMT} reflects stress history of the soil and can be regarded as earth pressure coefficient K_0 amplified by the penetration (Marchetti et al. 2001). Profile of K_{DMT} resembles *OCR* profile, with K_{DMT} value equal to 2 corresponding to normally consolidated cohesive soils. The following formula – valid for the soils with material index I_D smaller than 1.2 – is applied for overconsolidation ratio (Marchetti, 1980):

$$OCR = (0.5K_{DMT})^{1.56} \tag{2}$$

Some other correlations have been proposed to determine stress state and history in organic soils (Lechowicz et al. 1997). K_{DMT} values close to 2 are observed in the subsoil under the dike while higher values are found in the subsoil at the embankment toe (Fig. 4). Soft soil under the central part of the dike is thus normally consolidated, while the soil at the embankment toe is overconsolidated. This overconsolidation can be related to the ground water fluctuations, the complex stress history during the dike construction and to the lateral stress increase in the subsoil, due to the embankment load. Apart from the mechanical overconsolidation a structural overconsolidation can be also observed in organic soils (Wolski, 1988). The subsoil under the central part of the dike is normally consolidated since the weight of the embankment exceeded the previous preconsolidation pressure in the soft soil, what can be checked with a simple calculation. An analysis of CPTU and DMT data confirms that the soft soil under the central part of the dike is normally consolidated, while the subsoil under the dike toe is in overconsolidated state.

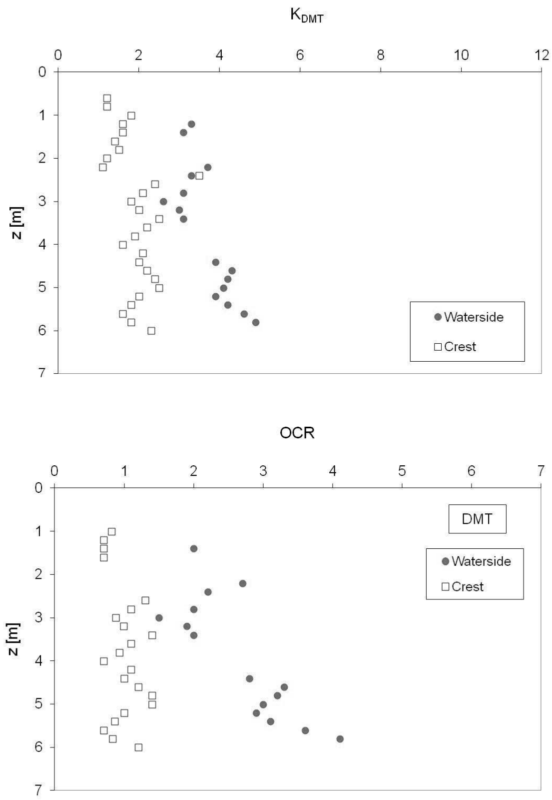

Fig. 4. Lateral stress index K_{DMT} and overconsolidation ratio *OCR* of the dike subsoil

Undrained shear strength in the week subsoil under the dike (Fig.5 and Fig.6) is calculated using the following formula:

$$c_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{3}$$

where:

N_{kt} – cone factor assumed equal 20 for mud and peat.

For a given section one can notice higher undrained shear strength of the peat layer and mud layer under the central part of the dike (crest) than that near the toe. It is systematically lower at soft layers at the downstream side than at the waterside. In some sections extremely small cu values of a few kPa are obtained at the downstream side. The detailed analysis of the undrained shear strength at the waterside and the downstream toe of the dike permit to attribute this difference to the filtration field and the seepage forces under the dike. For a specific site conditions the upward hydraulic gradient appears in soft soil layers behind the dike. Under the multiple action of upward hydraulic gradient during high water periods some internal erosion or degradation of the soil structure could appear. In addition, due to hydraulic gradient the effective stress state can be modified, especially in soft less permeable layers.

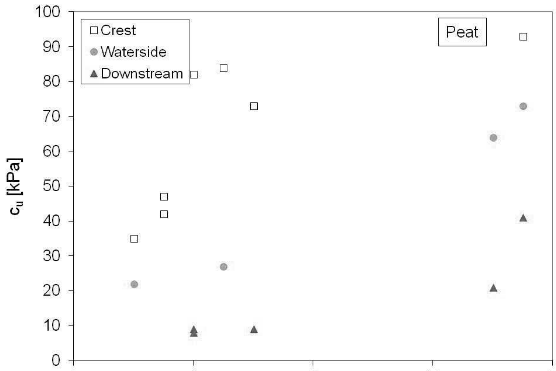

Fig. 5. Undrained shear strength in the peat layers under the dike

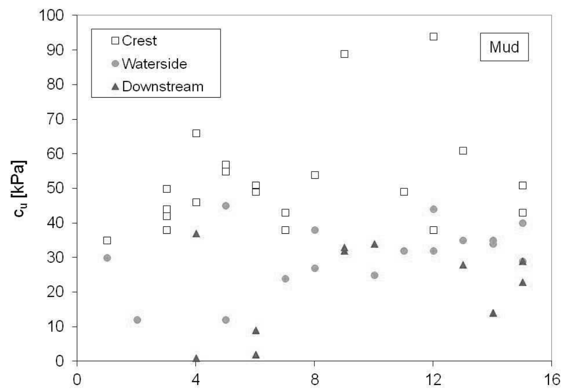

Fig. 6. Undrained shear strength in the mud layers under the dike

Dissipation test in normally consolidated and overconsolidated

During the cone penetration the nearby soil is subjected to complex loading depending on its position. The measurements of pore water pressure with different filter locations (u_1, u_2, u_3) reflect this complex loading (Sully et al. 1999). In the present tests typical CPTU probe with u_2 filter location was used. Water pressure u_2 at given depth can be presented (Burns and Mayne, 1998) as:

$$u_2 = u_0 + \Delta u_{oct} + \Delta u_{shear} \tag{4}$$

where:
u_0 – equilibrium pore pressure in-situ,
Δu_{oct} – octahedral component of the excess pore pressure corresponding to the zone of plastic soil around the cone tip (see Fig. 7),
Δu_{shear} – shear-induced component of the excess pore pressure generated in the interface.

Octahedric component of excess pore pressure is always positive and decreases during dissipation test (Mayne, 2001). Shear-induced component depends on dilative/contractive behavior of the soil within the interface. It can be negative in overconsolidated or fissured soils or positive in soft normally consolidated soils. Initial pore water pressure at the beginning of dissipation procedure can be lower or higher than the hydrostatic one (see Fig. 8).

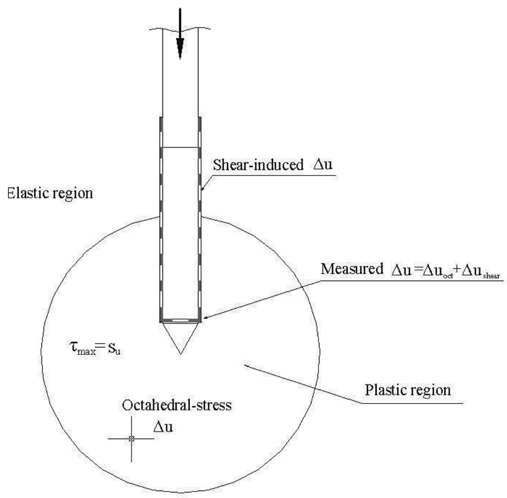

Fig. 7. Soil zones around CPTU probe (after Mayne, 2001)

When the penetration is stopped a local redistribution of water pressure occurs between the zone of predominant compressive stress and the shear stress zone within the interface and the dissipation of the excess water pressure begins. As a result the pore water pressure u_2 is initially increasing in overconsolidated soils (dilatory response) and a monotonic decay of pore water pressure is observed in normally consolidated soils.

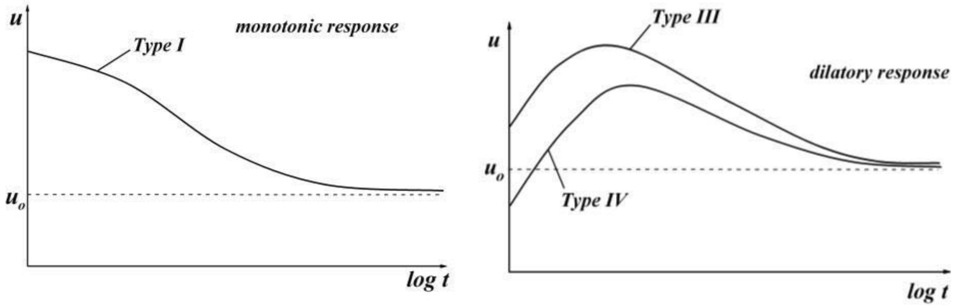

Fig. 8. Typical dissipation curves in normally consolidated and overconsolidated soils (after Sully et al. 1999)

Interpretation of dissipation test in normally consolidated soils

Interpretation is based on the cavity expansion theory with critical state soil models, with couple or uncouple consolidation analysis. Initial distribution of the pore water pressure around the probe is obtained with strain path method (Baligh and Levadoux, 1986), large strain finite element analysis (Houlsby and Teh, 1988) or cavity expansion (Burns and Mayne, 1998, Chang et al. 2001). For practical purpose the normalized excess pore water pressure is defined (Lunne et al., 1997):

$$U = \frac{u_t - u_0}{u_i - u_0} \tag{5}$$

where:
u_t – pore water pressure measured in time t,
u_0 – equilibrium pore water pressure in-situ,
u_i – initial pore water pressure at the beginning of dissipation t=0.

Value of water pressure u_i is determined by linear extrapolation of the initial portion of dissipation curve plotted in square root time. This procedure should be applied in case of dynamic redistribution of pore water pressure around the CPTU probe at the initial stage of the tests, immediately after the penetration stops. An example of the dissipation curve under the central

part of the embankment is given in Fig. 9. Extrapolated initial pressure u_i is equal 513 kPa. The hydrostatic pore pressure u_0 at the test depth is 52 kPa. The normalized excess pore water pressure is given in Fig. 10. The time t_{50}, i.e. when 50% of the normalized excess pore water pressure dissipates, can be read. For this curve the dissipation time t_{50}=195 s was estimated.
Flow parameters (i.e. coefficient of consolidation and hydraulic conductivity) can be derived from dissipation curves. As the dissipation undergoes predominantly in horizontal direction these parameters correspond to the radial flow. In normally consolidated soils the time t_{50} can be used to determine the value of hydraulic conductivity in horizontal direction. An empirical correlation (Parez and Fauriel, 1988) is given:

$$k_h = (251 \times t_{50})^{-1.25} \tag{6}$$

where:
t_{50} is introduced in seconds, and k_h is obtained in cm/s.

Hydraulic conductivity in horizontal direction as a function of t_{50} can be also evaluated with Robertson et al. 1992 method or estimated using the soil classification. The proper evaluation of hydraulic conductivity and consolidation coefficient should also include the knowledge of soil anisotropy.

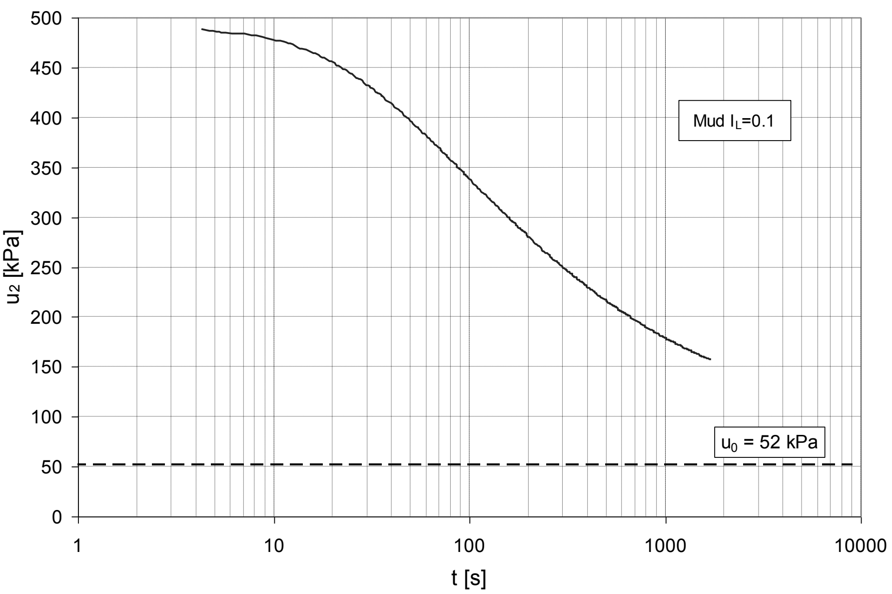

Fig. 9. Typical dissipation curve in NC soils under the central part of the dike (Bałachowski, 2006b)

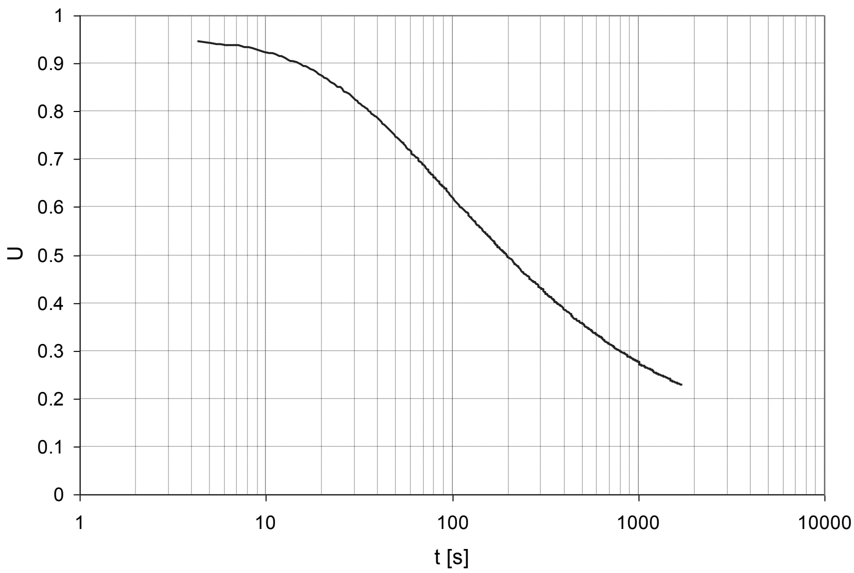

Fig. 10. Normalized excess pore water pressure (Bałachowski, 2006b)

Interpretaion of dissipation test in overconsolidated soils

A set of dissipation curves registered in overconsolidated plastic mud at the toe of the embankment is shown in Fig. 11. While in soft plastic mud a monotonic water pressure decay is observed, in plastic mud a dilatory response is found. The methods used for monotonic dissipation can not be applied directly. The dilatory curves are classified as a type III (Fig.8) dissipation curves according to Sully (1999).

There are two different approaches to the interpretation of the non-monotonic dissipation curves. The first one consists on transferring a dilatory response to the monotonic dissipation case. In the second one the overall dissipation curve with ascending and descending phases is described in one analytical model.

In the first approach two correction methods for the transfer of dilatory to monotonic dissipation curve are proposed (Sully et al. 1999), i.e. logarithm of time plot correction or square root of time plot correction. An example of these correction methods is given in Bałachowski, 2006b. One should note that the initial part of the dissipation curve is however lost when these two correction methods are applied. To overcome this limitation an analytical method based on cavity expansion theory was proposed (Burns and Mayne, 1998) describing both ascending and descending phases of the dissipation curve. In case of dilatory dissipation the normalized excess water pressure *U* can exceeds 1, especially in highly overconsolidated soils. An example of such set of normalized curves, based on the data recorded at the toe of the dike, is given in Fig. 12.

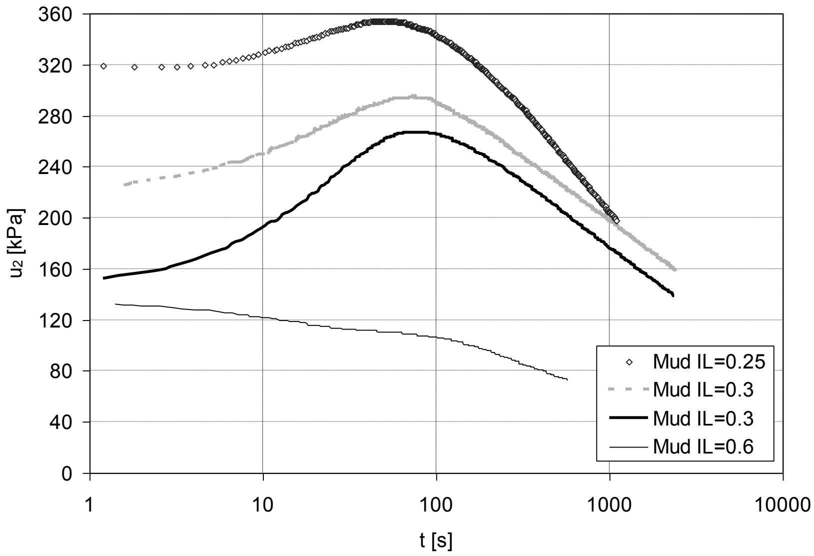

Fig. 11. Set of dissipation curves at the toe of the dike (Bałachowski, 2006b)

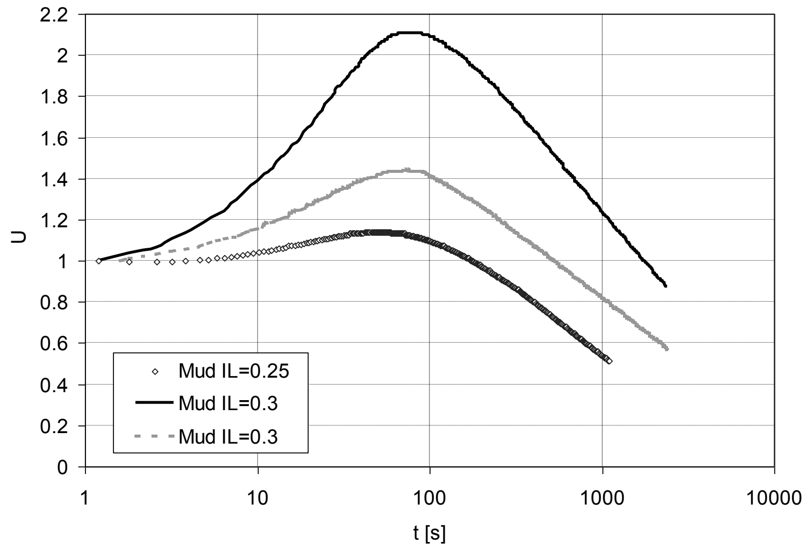

Fig. 12. Set of normalized excess pore water pressures at the toe of the dike (Bałachowski, 2006b)

Conclusions

The stress history in the soft soil under the dike was evaluated from enhanced in-situ tests. While the normally consolidated soil was found under the central part of the embankment the overconsolidated subsoil was detected at the embankment toe. The estimated subsoil overconsolidation is the complex phenomena involving both mechanical overloading and structural effect. The undrained shear strength of the soft layers, estimated with CPTU results, depends on the position under the dike corps. For a given layer this value is generally smallest for the tests performed at the dike toe on the downstream side. The CPTU dissipation curve provide additional information on the stress state and stress history in the soil. Monotonic pore water pressure response was found in the dissipation tests performed under the central part of the embankment, where the soil is normally consolidated. Dilatory pore water pressure response was observed in overconsolidated subsoil at the embankment toe. These observations are in agreement with the stress state and history determined with CPTU and DMT. Dissipation test with a sound knowledge of soil stress history and anisotropy can provide a method for in-situ evaluation of the consolidation coefficient and hydraulic conductivity in horizontal direction. Due to soil heterogeneities a larger number of dissipation tests will be necessary to provide a reliable estimation of the hydraulic conductivity and the consolidation coefficient in alluvial soils.

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Evaluation of soft clay properties from interpretation of CPTU data within a SHANSEP framework

Don J. DeGroot

Department of Civil and Environmental Engineering, University of Massachusetts Amherst, Amherst, MA, USA

Abstract. The paper presents a framework for determining piezocone (CPTU) empirical correlation factors k and N_{kt} for estimation of soft clay preconsolidation stress and undrained shear strength that are mutually consistent within the well-established relationship between normalized undrained shear strength (s_u/σ'_{v0}) and overconsolidation ratio (OCR). By way of background a general philosophy for conduct of site characterization programs in soft clays is first reviewed followed by general recommendations for performing CPTU testing and laboratory test programs using undisturbed samples. Equations for the commonly used CPTU empirical correlation factors k and N_{kt} are presented in the context of the SHANSEP equation, which relates s_u/σ'_{v0} and OCR, that provides for a consistent selection of k and N_{kt} values. Examples that demonstrate application of the methodology are given for several well characterized soft clay and silt sites.

Key words: soft clay, CPTU, SHANSEP, preconsolidation stress, undrained shear strength

Introduction

Geotechnical site characterization for soft, low overconsolidation ratio clays involves determination of soil stratigraphy, in situ pore water pressure conditions, and soil properties for analyses and design of geotechnical engineering projects. It is best conducted using a combination of in situ testing and laboratory testing of high quality undisturbed samples. The tools and procedures that can be used to perform an effective and reliable site investigation are well developed and should be thoroughly embedded in practice.

The focus of the paper is relevant to construction projects that involve relatively uniform deposits of soft saturated cohesive soils. Such soils include clays, silts, and organic soils of low to high plasticity, although for convenience the paper will often refer to all cohesive soils as "clays". These soils have usually been deposited in an alluvial, lacustrine or marine environment and typically have Standard Penetration Test (SPT) blow counts that are weight-of-rod or hammer, except within surface drying crusts. They can have highly varied and complex stress histories, which is the dominant factor controlling their compressibility and strength behavior. Projects involving construction on soft clays require estimates of the amount and rate of settlement and an assessment of undrained foundation stability. As such, determination of the relevant soil properties for analyses and design is the major objective of a site characterization program after first defining the soil profile and groundwater conditions.

The paper begins with an overview of the objectives of a site characterization program involving soft clays and the general methodology for fulfilling these objectives. Recommended laboratory methods for measurement of consolidation and undrained shear strength anisotropy are reviewed. Likewise, recommended Piezocone (CPTU) equipment and test procedures are also covered. Interpretation of CPTU data is described within a normalized soil behavior framework such that selection of CPTU empirical correlations can be made that are consistent with this framework.

The paper presents material abstracted from Ladd and DeGroot (2003) and DeGroot and Ladd (2012). Other related overview papers on soil behavior and site characterization include Terzaghi et al. (1996), Leroueil and Hight (2003) and Hight and Leroueil (2003).

Soft clay site characterization

The two key objectives of a site characterization program for soft clays are to define the site stratigraphy and to estimate the relevant soil properties needed for design. The first objective encompasses determining the location and relative state of principal soil types (stiffness if cohesive and density if granular) and ground water conditions (location of water table and possible deviations from hydrostatic pore pressures). The second objective quantifies the engineering properties of the foundation soils that are needed for settlement and stability analyses. All programs must include steps to properly classify the foundation soil types. In terms of properties, the in situ stress state (in situ vertical effective stress σ'_{v0}; in situ equilibrium pore water pressure u_0; preconsolidation stress σ'_p) is the most important and required in all cases, while the need for other parameters depends on the specific design objectives.

For settlement analyses the magnitude of the final consolidation settlement is always important and the key in situ parameters are stress history (SH = values of σ'_{v0}, σ'_p and overconsolidation ratio OCR = σ'_p/σ'_{v0}) and the values of compressibility parameters. For undrained stability analyses the key required design parameter is the variation in undrained shear strength (s_u) with depth. For problems involving consolidated-undrained shear behavior, such as stage construction, estimates of s_u for normally consolidated (NC) clay is also important because the first stage of loading should produce an in situ consolidation state $\sigma'_{vc} > \sigma'_p$ within a significant portion of the foundation. Most stability analyses account for undrained shear strength anisotropy by using the average or mobilized strength, that is $s_u = s_u(ave)$, while more complex anisotropic analyses explicitly model the variation in s_u with inclination of the failure surface.

To fulfill the above objectives, the optimal site characterization program should use a combination of in situ tests and undisturbed sampling for laboratory testing. In situ tests, such as the piezocone (CPTU), are best suited for soil profiling since they can rapidly provide a continuous record of the in situ penetration data that are used for identifying the spatial distribution of soil types and information about their relative state. In situ tests, however, are not as well suited for direct determination of strength-deformation properties because of the effects of installation disturbance and poorly defined and uncontrollable boundary conditions. Therefore, derived soil properties from in situ tests depend on empirical correlations that are at times quite scattered and often unreliable without site specific verification. Conversely, most laboratory tests have well controlled boundary and drainage conditions that enable direct measurement of strength-deformation properties. However, the reliability of laboratory data is strongly dependent on proper test procedures and good quality undisturbed samples. Otherwise, variable and often unknown degrees of sample disturbance can cause poor quality laboratory data that produce erroneous spatial trends in soil properties.

The scope of a site investigation program should follow the advice of Peck (1994) for characterizing a relatively homogeneous clay layer versus a highly heterogeneous deposit. If prior knowledge indicates a reasonably well-defined profile having soft clay, then careful undisturbed sampling and laboratory testing is warranted if the design concept has a significant potential for settlement and stability problems. However, for an ill-defined soil profile with heterogeneous layers, then the program should emphasize in situ testing to locate the weaker, more compressible soil layers with minimal collection of undisturbed samples for laboratory testing.

In summary, site characterization programs for soft clays should combine the best aspects of in situ and laboratory testing, with the scope depending upon the extent of prior knowledge about the nature of the foundation soils and the size and complexity of the project. For sites involving relatively uniform saturated clay deposits, the focus should primarily be on the use of multiple CPTU soundings for soil profiling and assessment of spatial variability. This should be coupled with collection of good quality undisturbed samples from selected boreholes for laboratory strength-deformation testing. The laboratory data can also be used to develop site specific correlations for more reliable estimates of σ'_p and s_u from CPTU.

Laboratory determination of stress history and undrained shear strength

Sample disturbance is the most significant issue affecting the quality and reliability of laboratory test data for clays. It causes changes to the stress state and structure of an intact soil and as a result all key design parameters are adversely influenced. While no definitive method exists for determining sample quality, valuable information can be obtained from making use of both qualitative and quantitative methods. Qualitative (visual) assessment of sample quality is best made by examination of sample X-rays (e.g., ASTM D4452). The most established quantitative method is the measure of volumetric strain (e.g., Terzaghi et al. 1996) or normalized change in void ratio (e.g., Lunne et al. 2006) upon reconsolidation to the in situ effective stress state and both parameters should be reported for all consolidated test specimens.

Specimens for consolidation and shear tests should avoid soil from within about 1 to 1.5 times the tube diameter at the top and bottom of the tubes due to the greater degree of disturbance that usually occurs at the ends. Specimen locations should ideally be selected using radiography in order to identify representative soil types and soils having minimal disturbance. Selected sample sections should be de-bonded using the procedure described by Ladd and DeGroot (2003) to break the bond at the soil-tube interface before extruding samples, this is especially important for prolonged storage times and with lower quality tube materials such as regular steel or galvanized steel.

Consolidation Testing

The one-dimensional consolidation test is typically performed using an oedometer cell with application of incremental loads (IL) as this technique is widely available and relatively easy to perform. However, the constant rate of strain (CRS) test (Wissa et al. 1971) has significant advantages over the IL test as it produces continuous measurement of deformation, vertical load, and pore pressure for direct calculation of the stress-strain curve and coefficients of permeability and consolidation. Computer-controlled load frames allow for automation of IL and CRS testing (with the additional use of a flow pump for back pressure saturation during CRS testing) which generally results in improved data quality and test efficiency. General requirements for the IL test are covered by ASTM D2435 and for the CRS test by ASTM D4186. Computer controlled stress path triaxial tests with K_0 consolidation of specimens beyond σ'_p also give reliable data for determining the compression curve (i.e., σ'_p, compressibility).

All methods for estimating σ'_p require test data from samples of reasonable quality. The compression curves used for interpretation should either be from CRS tests having an appropriate strain rate (with perhaps a 10% reduction in σ'_p to better match end of primary values for high quality samples) or by data plotted at a constant consolidation time (t_c) from IL tests with appropriate load increment ratios (see examples in Ladd and DeGroot 2003). Casagrande's method is the most widely used technique for estimating σ'_p, but it can be quite subjective and difficult to apply with rounded compression curves. The strain energy method of Becker et al. (1987) uses work per unit volume as the criterion for estimating σ'_p from a plot of strain energy versus σ'_v in linear scales.

Undrained shear strength testing

Laboratory testing for measuring undrained shear behavior of soft clays should rely on a consolidated-undrained (CU) shear test program that accounts for anisotropy, strain rate effects and sample quality. The Recompression and SHANSEP strength testing techniques were independently developed to address these important soil behavior issues. Both use CK_0U tests with shearing in different modes of failure, such as triaxial compression/extension (TC/TE) and direct simple shear (DSS), at appropriate strain rates to account for anisotropy and strain rate effects. But the two methods have a major difference in how they deal with sample disturbance. In the Recompression method, Bjerrum (1973) recognized the unreliable nature of the standard unconsolidated-undrained test and proposed using CU tests that are anisotropically (CAU) reconsolidated to the in situ state of stress (σ'_{v0}, σ'_{h0}), as shown by point 3 in Figure 1. This procedure assumes that the reduction in water content during reconsolidation to σ'_{v0} is sufficiently small to compensate any destructuring during sampling, so that the measured s_u data is representative of in situ clay for undrained stability cases. Berre and Bjerrum (1973) recommended that the volumetric strain during recompression should be less than 1.5 to 4 percent.
The SHANSEP method (Ladd and Foott 1974, Ladd 1991) is based on the experimental observation that the undrained stress-strain-strength behavior of most ordinary clays, for a given mode of shear, is controlled by the stress history of the test specimen. The method assumes that these clays exhibit normalized behavior and uses mechanical overconsolidation to represent all preconsolidation mechanisms. The procedure explicitly requires knowledge of stress history profiles for the clay layer and was developed to obtain s_u profiles for both the unconsolidated--undrained and consolidated-undrained stability cases. Test specimens are K_0 consolidated to stress levels (σ'_{vc}) greater than σ'_p to measure the normally consolidated behavior (Points A and B in Figure 1), which now can be readily performed using computer controlled systems that also produce 1-D compression curves for estimating σ'_p. Specimens are also unloaded to varying laboratory OCRs (Points C and D) to measure OC behavior. The s_u/σ'_{vc} vs. OCR data are then used to obtain the SHANSEP values of S and m such that

$$s_u/\sigma'_{vc} = S(OCR)^m \qquad (1)$$

The values of S will always have some inherent variability, in addition to a general trend of decreasing S with increasing σ'_{vc}/σ'_p (especially for structured clays).

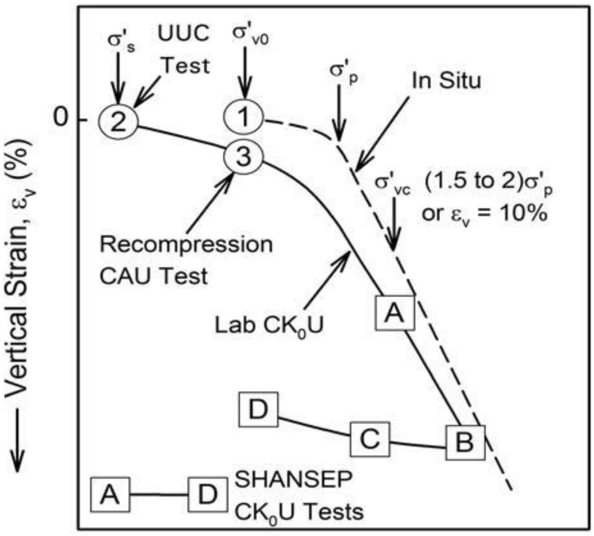

Fig. 1. Recompression and SHANSEP consolidation procedure for laboratory CK_0U testing (slightly modified from Ladd 1991)

For clay deposits that are or will become truly normally consolidated (e.g., due to construction loading), the SHANSEP procedure should be used because the Recompression method of reconsolidation to $\sigma'_{v0} = \sigma'_p$ for NC soil will result in high strains compared to the in situ soil (Figure 1), and thus lead to unsafe design strengths. SHANSEP CK_0U tests for predicting the behavior of in situ OC clay should use $t_c \approx 1$ day so as to allow secondary compression to help "restore" some of the clay structure. But for in situ NC soil, the tests should use $t_c \approx$ end of primary consolidation to minimize secondary compression.

The SHANSEP reconsolidation technique of consolidating test specimens beyond the in situ σ'_p will destructure all OC clays to varying degrees. Furthermore, the mechanical overconsolidation used with SHANSEP will never exactly re-produce the undrained stress-strain behavior of natural overconsolidated deposits, especially for highly structured clays where mechanical consolidation is not the primary mechanism causing σ'_p. With highly structured clays SHANSEP values of S are much too low. However, even though the SHANSEP technique gives imperfect results (except when the in situ OCR = 1 as noted above), it does offer several important practical advantages: 1) it forces the user to establish the initial stress history of the soil, which is needed to understand the deposit and is required for settlement analyses and stage construction; 2) the error in S should always be on the safe side and is probably < 5 to 10% for most cohesive soils of low to moderate sensitivity; 3) errors in m are not significant for low OCR deposits (thus can be estimated); and 4) with the advent of computer automation, SHANSEP tests yield continuous compression curves that provide values of σ'_p, plus those for compressibility, and K_0 vs. σ'_v (for triaxial tests). In any case, the SHANSEP equation can always be used to calculate the range in s_u profiles for likely variations in stress history and SHANSEP values of S and m.

For problems requiring reliable predictions of deformations caused by undrained shear in OC deposits, the Recompression technique is much preferable (assuming good quality samples) because SHANSEP tests will always underestimate the in situ undrained stiffness. However, Recompression tests cannot, by definition, measure values of σ'_p and give only depth specific values of s_u. Thus a separate program of consolidation tests (to get stress history data) is needed to check the reasonableness of the measured s_u/σ'_{v0} ratios and for settlement and consolidated-undrained stability analyses. Recompression tests can use a range of σ'_{vc} values (but $< \sigma'_p$) in order to compute s_u profiles via the SHANSEP equation.

Levels of laboratory strength testing

Ladd (1991) proposed three levels of sophistication for obtaining strength data from laboratory testing for undrained stability analyses. Selection of an appropriate level for a given project depends on many factors including complexity, risk, cost, and preliminary vs. final design. Level A is the most sophisticated and involves performing CK_0UC/E and DSS tests to develop anisotropic strength parameters. Level B uses either DSS tests or a combination of CK_0U TC and TE tests to obtain s_u(ave) values. Level C is the least sophisticated and involves using empirical correlations for the SHANSEP S and m parameters as summarized in Table 1 to compute s_u(ave). These recommended S and m values are based primarily on experience with North American clays although they should be relevant to other clays of similar characteristics.

Tab. 1. Level C values of S and m for estimating s_u(ave) via SHANSEP equation (modified from Ladd 1991)

Soil Description	S	m†	Remarks
Sensitive cemented marine clays (PI < 30%, LI > 1.5)	0.20 nominal SD = 0.015	1.00	Champlain Sea clays of Eastern Canada
Homogeneous CL and CH sedimentary clays of low to moderate sensitivity (PI = 20 to 80%)	S = 0.20 + 0.05(PI/100) or simply 0.22	0.80	without shells or sand lenses-layers
Northeastern US varved clays	0.16	0.75	assumes that DSS mode predominates
Sedimentary deposits of silts & organic soils (AL plot below A--line) and clays with shells	0.25 nominal SD = 0.05	0.80	excludes peat

Note: PI = plasticity index, LI = liquidity index, CL = low plasticity clay in Unified Soil Classification System (USCS, ASTM D2487), CH = high plasticity clay, AL = Atterberg Limits, A-line = Casagrande plasticity chart, SD = standard deviation.

Piezocone (CPTU) testing

Equipment and procedures

The CPTU is excellent for soil profiling as it can rapidly provide detailed subsurface information and estimates of s_u and σ'_p if reliable empirical correlations are available. ISO/FDIS 22476-1, ASTM D5778, and Lunne et al. (1997) present detailed guidelines on piezocone equipment and test procedures. Four significant equipment and test procedure issues that commonly lead to poor quality CPTU data for soft clays are: 1) low cone area ratio (which is approximately equal to the area of the tip load cell divided by the projected area of the cone), 2) inaccurate friction sleeve data, 3) unreliable sensor measurements, and 4) poor saturation. Cones should have as high an area ratio as practical (e.g., ~ 0.8) because cones with lower area ratios (down to 0.5 or less) greatly decrease the accuracy of the corrected tip resistance (q_t) because of the large pore pressure (u_2) correction. Sleeve friction (f_s) values are notoriously inaccurate (e.g., see Lunne 2010) due to poor equipment design and the fact that the values are very low in soft clays (and even close to zero in soft, sensitive clays). As a result, f_s values are often unreliable although f_s is far less important than q_t and u_2 for interpretation of CPTU data in soft clays.

Quantitative interpretation of piezocone data in soft clays requires very accurate cone resistance (q_c) and u_2 measurements, yet it is not uncommon to see very high capacity "universal" cones with insufficient data resolution being used in soft clays. Zero drift due to temperature changes is another sensor problem with some cones being quite sensitive to changes in temperature. It is thus important that zero readings are recorded (both before and after each sounding) at a temperature that is representative of in situ conditions.

Poor quality u_2 data is a pervasive problem mainly caused by poor initial saturation of the cone and desaturation due to cavitation in soil above the water table and in dilating sands below the water table. Water is the ideal saturation fluid, but because of difficulties with freezing and maintaining saturation (especially with coarse filters), more viscous fluids such as silicone oil and glycerine are used. Different filter materials can be used although polypropylene plastic is the most common in practice. DeJong et al. (2007) studied the performance of various saturation fluids using coarse (15 to 45 μm) polyethylene filters and concluded that a low to moderate viscosity (100 – 1000 cS) silicone oil may be the preferable fluid for testing in saturated soils. For penetration above the water table and if dilating sands are anticipated, then higher viscosity silicone oil, possibly in combination with finer filters, should be considered. ASTM D5778 provides guidelines on proper saturation procedures for various saturation fluids and filter materials including the use of high vacuum (< - 90 kPa) deairing for a minimum acceptable duration, and protecting the filters from desaturation during cone assembly and commencement of testing. Additionally it is recommended that filters be replaced after each sounding.

CPTU data interpretation

CPTU soundings provide a rapid and detailed approach for developing soil profiles as the data can be used to readily differentiate between soft cohesive and free draining deposits and for detecting the presence of interbedded granular-cohesive soils. Soil behavior type classification charts, such as those developed by Roberston (1990), are widely used and are especially useful for determining if the site consists of relatively uniform deposits versus complex heterogeneous foundation conditions. Multiple soundings across a site are also very useful for detecting spatial variations in soils units and their relative state.

CPTU data can be used to estimate σ'_p (Lunne et al. 2007)

$$\sigma'_p = k(q_t - \sigma_{v0}) = k(q_{net}) \tag{2}$$

where q_{net} is the net penetration resistance and k is typically within the range of 0.25 to 0.35. The undrained shear strength can be estimated from CPTU data using empirical correlations that were developed using strength data from other testing methods via

$$s_u = (q_t - \sigma_{v0})/N_{kt} = q_{net}/N_{kt} \tag{3}$$

where N_{kt} is the cone factor that varies with the selected reference undrained shear strength. Historically, recommended N_{kt} values have spanned a large range [e.g., 10 to > 20 for s_u(ave)] which presumably reflects differences in the nature of the clay (lean and sensitive vs. highly plastic) and its OCR, the reliability of the reference strengths, and the accuracy of q_{net}. Continued research using high quality CPTU and reference s_u data is resulting in a narrowing range of recommended N_{kt} values (e.g., Low et al. 2010).

SHANSEP framework for interpretation of CPTU data

It is common practice to independently select k and N_{kt} values for interpretation of σ'_p and s_u from CPTU data without consideration of the well-established relationship between s_u/σ'_{v0} and OCR for clays. Mesri (2001), Ladd and DeGroot (2003), Been et al. (2010), and Robertson (2012) discuss selection of k or N_{kt} values that are mutually consistent in a normalized soil behavior framework such as SHANSEP or Critical State Soil Mechanics.

Interrelationship between k and N_{kt}

If σ'_p is estimated from CPTU data using a selected value for k in Equation 2, then the normalized soil behavior framework implies that a consistent value of N_{kt} should be determined via

Equation 1 as a function of S and m for the relevant mode of reference undrained shear strength of interest, i.e.,

$$N_{kt} = k^{-m}S^{-1}[(q_t - \sigma_{v0})/\sigma'_{v0}]^{1-m} = k^{-m}S^{-1}[q_{net}/\sigma'_{v0}]^{1-m} \quad (4)$$

This relationship could also be rearranged with k as a function of the undrained shear strength parameters N_{kt} and S, however, both these latter parameters depend on which mode of undrained shear strength is of interest (e.g., CAUC, field vane, ave, etc.) whereas σ'_p is a unique in situ state parameter. In any case, Been et al. (2010) note that the selection of CPTU interpretation parameters k and N_{kt} should satisfy the relationship

$$N_{kt}Sk^m = [q_{net}/\sigma'_{v0}]^{1-m} \quad (5)$$

As an example application of Equation 4, the Table 1 recommended Level C values of S = 0.22 and m = 0.8 for s_u(ave) for homogeneous CL and CH sedimentary clays together with the commonly used k = 0.30 imply

$$N_{kt,ave} = 11.9[q_{net}/\sigma'_{v0}]^{0.2} \quad (6)$$

with the key feature here being that N_{kt} is not a constant but rather a function of normalized q_{net}. For low OCR CL and CH clays of low to moderate sensitivity, q_{net}/σ'_{v0} values at depth are typically within the range of 2 to 8 thus yielding $N_{kt,ave}$ values from Equation 6 that range between 14 to 18.

Table 2 presents Equation 4 $N_{kt,ave}$ values for several well characterized soils for each of the four Level C soil categories given in Table 1. The selected S and m values are the generic ones listed in Table 1 for each soil category while the k values are from the references cited and are based on site specific correlations between q_{net} and σ'_p with the latter values all determined from laboratory tests performed on high quality block samples. The range of q_{net}/σ'_{v0} values listed are from CPTU profiles performed at each site and the range of corresponding Equation 4 $N_{kt,ave}$ values are listed (except for the sensitive Louiseville clay for which m = 1). The range of $N_{kt,ave}$ values listed for each soil bracket the universal value of 16 often recommended for soft clays, with the exception of the highly anisotropic Connecticut Valley Varved Clay for which, as noted in Table 1, s_u(DSS) is the critical mode of shear for stability problems.

The k values for the five clays listed in Table 2 are within the narrow range of 0.28 to 0.31. Given that these were determined from site specific correlations based on σ'_p data determined from testing of high quality block samples further validates the common recommendation of a universal value of k = 0.30 for soft clays.

The N_{kt} values listed in Table 2 are for s_u(ave) as the reference undrained shear strength. Of course, other reference undrained shear strengths can be used and N_{kt} modified accordingly. For example, Figure 2 presents SHANSEP S values as a function of PI for K_0 consolidated

TC, DSS and TE modes of shear for a large collection of soft clays and silts. While there is some variation within each mode of the shear, the TC values show little to no trend with PI and average approximately S_{TC} = 0.32. In this case Equation 6 becomes

$$N_{kt,TC} = 8.2[q_{net}/\sigma'_{v0}]^{0.2} \tag{7}$$

and for low OCR CL and CH clays of low to moderate sensitivity with q_{net}/σ'_{v0} values within the range of 2 to 8 yields $N_{kt,TC}$ values that range between 9.4 and 12. As for an example using site specific data, Lunne et al. (2006) report SHANSEP CK_0UC (= TC for simplicity of notation) values of S_{TC} = 0.31 and m = 0.76 for the Onsøy clay which together with k = 0.29 (Table 2) and a depth averaged q_{net}/σ'_{v0} ~ 5.0 for the site yields $N_{kt,TC}$ = 12.2 in comparison to the site calibrated average value of $N_{kt,CAUC}$ = 11.5 reported by Low et al. (2010) which was based on a series of CAUC Recompression tests performed on high quality block samples. Alternatively, without the measured Onsøy SHANSEP test data, just using the Figure 2 generic S_{TC} = 0.32 and Table 1 m = 0.8 yields $N_{kt,TC}$ = 11.6 for k = 0.29 and q_{net}/σ'_{v0} = 5.0.

Tab. 2. Range of Equation 4 $N_{kt,ave}$ values for several well characterized soft clays

Soil	S†	m†	k‡	Range q_{net}/σ'_{v0}	Range $N_{kt,ave}$
Louiseville, Quebec, Canada, PI = 45%, Sensitivity = 22, Leroueil et al. (2003)	0.20	1.00	0.31	11 to 14	16
Boston Blue Clay, MA, USA, PI = 20%, DeGroot (2003)	0.21	0.80	0.30	3 to 8	16 to 19
Onsøy, Norway, PI = 30 to 50%, Lunne et al. (2003), Landon (2007)	0.22	0.80	0.29	4 to 6	16 to 18
Connecticut Valley Varved Clay, MA, USA. DeGroot and Lutenegger (2003)	0.16	0.75	0.30	3 to 8	20 to 26
Bothkennar, UK. Clayey silt w/organics. Nash et al. (1992), Hight et al. (2003)	0.25	0.80	0.28	6 to 8	16 to 17

Note: †generic S and m values from Table 1 based on relevant soil category. ‡k and q_{net}/σ'_{v0} values from references cited in Col. 1 and are site specific; all k values based on site specific calibration with σ'_p data from laboratory testing of high quality block samples.

The generic m = 0.8 values given in Table 1 for homogeneous CL and CH sedimentary clays and sedimentary deposits of silts and organic soils can be refined for specific soils if the compression and swelling ratios (C_c and C_s) are available from 1-D consolidation tests (e.g., IL or CRS) using Ladd and DeGroot (2003) as

$$m = 0.88(1 - C_s/C_c) \tag{8}$$

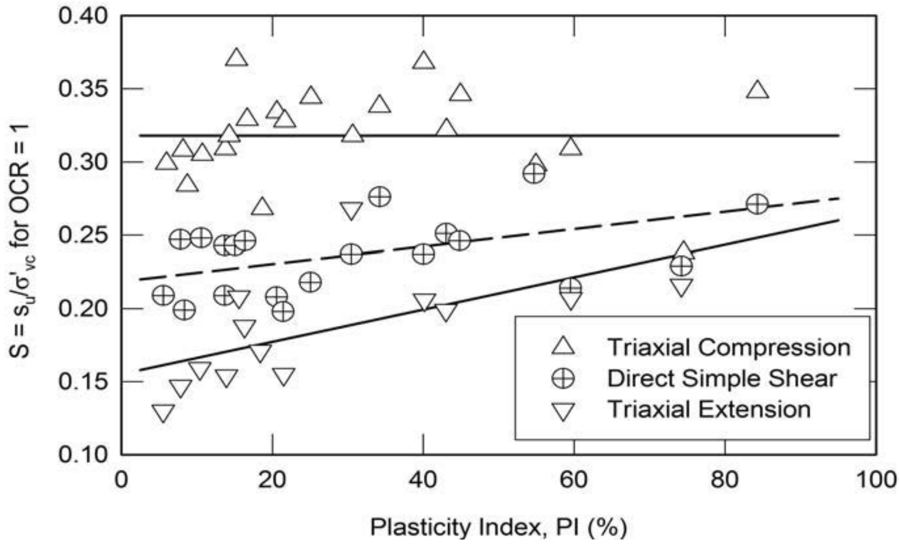

Fig. 2. Undrained strength anisotropy from SHANSEP CK_0U tests on NC clays and silts (from Ladd 1991).

Interrelationships with m = 1

Terzaghi et al. (1996) states that for in situ undrained shear strength profiles, m = 1 for all practical purposes such that Equation 1 for some soft clays and silts (with an exception being chemically bonded lean clays and silts) becomes

$$s_{u0}/\sigma'_p = s_u/\sigma'_{vc} \tag{9}$$

where s_u0 is defined by Terzaghi et al. (1996) as the in situ undrained shear strength at the in situ σ'_p while s_u/σ'_{vc} is the SHANSEP laboratory measured normally consolidated S value. Mesri (1975) based on field vane data and Terzaghi et al. (1996) based on laboratory data determined that $s_u(ave) = 0.22\sigma'_p$, independent of plasticity index for low to medium OCR clays, which is the equivalent of S = 0.22 and m = 1 in the SHANSEP equation.

If m is indeed taken as equal to one, then Equation 4 reduces to

$$N_{kt} = [kS]^{-1} \tag{10}$$

and N_{kt} is no longer a function of q_{net}/σ'_{v0}. For k = 0.30 and S = 0.22 for s_u(ave) and 0.32 for s_u(TC) the corresponding Equation 10 values are $N_{kt,ave}$ = 15 and $N_{kt,TC}$ = 10. This outcome is similar to the Mesri (2001) recommendation of a universal $N_{kt,ave} = 16 \pm 2$ that is applicable for stability analysis of embankments, footings, and excavations on both inorganic and organic soft clay and silt deposits. Mesri (2001) derived this range of recommended values by analyzing the CPTU as a bearing capacity problem, considering the triaxial compression test as the closest laboratory mode of shear to the cone, using different $s_u(TC)/\sigma'_p$ values for OCR < 5 inorganic clays and silts vs organic clays and silts, and a rate correction factor to account for the difference in strain rate between the cone and triaxial test.

SHANSEP framework for estimation of σ'_p from CPTU

For soft clay sites that have little variation in OCR the use of Equation 2 for estimating σ'_p is appropriate. However, if a site has large variations in OCR and reliable site specific laboratory σ'_p data are available, then a SHANSEP type of equation for q_{net}/σ'_{v0} vs. OCR is preferred for developing site specific correlations for σ'_p. Substituting s_u from Equation 3 into Equation 1 results in

$$q_{net}/\sigma'_{v0} = N_{kt}S(OCR)^m \tag{11}$$

for which rearranging and setting $N_{kt}S = S_{CPTU}$ and $m = m_{CPTU}$ results in

$$\sigma'_p = \sigma'_{v0}[(q_{net}/\sigma'_{v0})/S_{CPTU}]^{1/m_{CPTU}} \tag{12}$$

where values of S_{CPTU} (= q_{net}/σ'_{v0} for OCR = 1) and m_{CPTU} are determined from a power function regression to the q_{net}/σ'_{v0} versus OCR data. If m_{CPTU} is taken as equal to one then Equation 12 reduces to Equation 2.

Recommendations

The overarching recommendation for performing site characterization programs for soft clay projects is to combine the best attributes of both in situ and laboratory testing. Depending on project scope this should involve multiple CPTU profiles and some select borings for collection of high quality samples for performing advanced consolidation and undrained shear strength testing. The collective in situ and laboratory data sets will then allow for development of site specific CPTU correlations. The final site specific k and N_{kt} values should be selected such that they are mutually consistent in a normalized soil behavior framework as described by Equations 4 and 5.

For preliminary design purposes, low risk projects, or projects without the opportunity to develop site specific correlations the use of k = 0.30 and $N_{kt,ave}$ = 16 is a good starting point. More refined, soil specific values of N_{kt} can be estimated via Equation 4 with k = 0.30 and using the recommended S and m values for the four different soil categories given in Table 1. This approach is somewhat analogous to the empirical correlations derived by Karlsrud et al. (2005) for N_{kt} as a function of OCR and to some extent soil type as quantified by soil sensitivity. Although reliable OCR and S_t values are necessary to use the Karlsrud et al. (2005) correlations which generally requires good quality samples for laboratory testing.

Another important consideration is for projects where samples collected for laboratory testing and to develop site specific CPTU N_{kt} correlations turn out to be of marginal to poor quality due to excessive sample disturbance. In such cases, laboratory determination of σ'_p and Recompression method s_u data will be unreliable and likewise the site specific calibrations of k and N_{kt} will also be unreliable. In such cases a select number of SHANSEP tests can nevertheless be performed on the marginal to poor quality samples to determine site specific S and m values, with S being the more important variable especially if the OCR values are low (e.g., < 2). The site specific laboratory measured S and m values can be used together with a reasonable value of k (e.g. 0.30 for soft clays) to estimate a site specific N_{kt} via Equation 4. It is important when performing the SHANSEP tests, especially on marginal to poor quality samples, to make sure that specimens are consolidated to high enough stress levels to ensure that the virgin compression line has been reasonably approached (Figure 1).

Summary and Conclusions

Site characterization programs for soft clays should focus on performing multiple CPTU soundings for soil profiling and assessment of spatial variability combined with collection of good quality undisturbed samples from selected boreholes for laboratory strength-deformation testing. The laboratory data can also be used to develop site specific correlations for more reliable estimates of σ'_p and s_u from CPTU. The CPTU empirical correlation factors k (for σ'_p) and N_{kt} (for s_u) should be selected such that they are mutually consistent in a normalized soil behavior framework such as SHANSEP. For preliminary design purposes, low risk projects, or projects without the opportunity to develop site specific correlations, the use of k = 0.30 and $N_{kt,ave}$ = 16 is a prudent approach. Although more refined soil specific values of N_{kt} in such cases can be estimated for several different categories of soft clays using the SHANSEP equation and the recommended S and m values presented in Table 1.

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In – situ tests in organic soils

Geotechnical properties of peaty ground in Hokkaido, Japan

Hiroyuki Tanaka
Hokkaido University

Abstract. Wide areas in Hokkaido are covered by peat, which has caused many geotechnical problems, such as large settlement and lack of stability of foundations. Also, since Hokkaido Island is located at the most active seismic areas, dynamic properties of the ground are a very important parameter to predict the performance of structures during earthquake. In this investigation in situ as well as laboratory tests were carried out for peaty grounds. A unique relationship is found between the point resistance from Cone Penetration test (CPT) and the shear modulus measured by seismic cone, but it is completely different from the relation for usual clayey ground. In this paper, to find possible explanations for such a different result, various tests were conducted, focusing on shear strength, sample disturbance, rigidity index, anisotropy and dependency on strain levels.

Key-words: Peat, Cone Penetration test, Small strain stiffness

Introduction

Most flat areas in Hokkaido Island are covered by peat, as shown in Fig. 1. The ground consisting of peat soil has brought many kinds of geotechnical problems such as large settlement, large lateral deformation, and in extreme cases, failure of the foundations. In addition to these problems relating to the construction, damages caused by seismic vibration during an earthquake have become a serious social issue, especially after 2011 Tohoku earthquake. Nowadays it is required for geotechnical engineer to estimate the ground motion triggered by a great earthquake. Dynamic properties of the ground are a key parameter in the prediction. However, still the investigation or testing method for obtaining these parameters are not commonly carried out in practice and its cost is inevitably expensive, so that it is needed to estimate them from conventional testing method, such as Cone penetration test (CPT). Fig. 2 shows relationships, measured in Hokkaido areas, between shear modulus (G) measured by seismic cone (SCPT) and the net resistance q_{net} (q_t-σ_{vo}) from CPT, where q_t is the point resistance considering the effective cross-sectional area and σ_{vo} is the total overburden pressure at a measured depth. Tanaka and Tanaka (1998) have presented the relation of G=50q_{net} for clayey ground. Although this relation seems valid for clayey ground as shown in Fig. 2, the relation for peaty grounds is located in the bottom, and its coefficient is much smaller than that for usual clay and around 5, which is as small as 1/10 for the usual clay. In this paper, reasons for such a small coefficient for peaty grounds will be discussed.

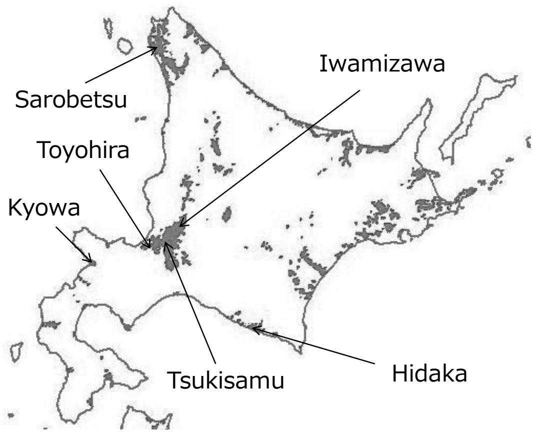

Fig. 1. Distribution of peat and location of tested sites

Characteristics of peat soils

Point resistance for peaty ground

Fig. 3 shows q_t distribution against depth measured at several sites, whose locations are indicated in Fig. 1. In Fig. 3, calculated q_t is also drawn by a thick solid line with following assumptions: cone factor (N_{kt}) is 20; undrained strength incremental ratio (s_u/p') is 0.5; the unit weight (γ_t) is 10.5 kN/m3; the ground water table is located 1m below the ground surface. Although such a large N_{kt} and s_u/p' may give large q_t, measured values of q_t except for Sarobetsu are greater than the calculated values, especially the q_t values are significantly large at the ground surface. Fig. 4 shows comparison of q_{net} measured by the conventional CPT (with 60 °conical angle and 10 cm^2 cross sectional area) and ball penetrometer (BPT) with a large cross sectional area (miniBPT: 50 cm^2; BPT: 100 cm^2). A tendency can be observed that the BPT with a large diameter provides smaller q_{net} than the conventional CPT or with a smaller ball. This indicates that because of relatively small cross sectional area for the CPT, q_t or q_{net} is likely affected by local resistance generated by still not discomposed peat material, such as fibers or fraction of woods.

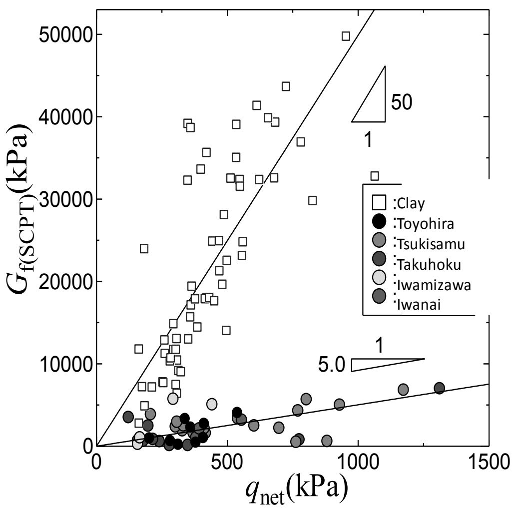

Fig. 2 Relation between G and q_{net}

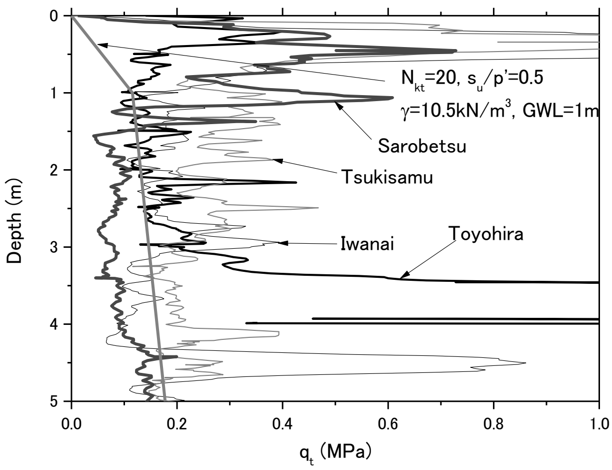

Fig. 3. CPT test result

Using the BPT with 100 cm² ball, the cyclic penetration was conducted for estimating the sensitivity of the peaty ground at Toyohira site. Test results are shown in Fig. 5 in comparison with clayey grounds (Nakamura et al., 2009). The vertical axis in the figure indicates the reduction of q_t due to cyclic, normalized by q_t at the first penetration ($q_t 1$). Reduction of q_t due to cycling penetration for peaty ground at the Toyohira site (site 1 and 2) is similar to that at the Mihara site whose liquidity index (I_L) is about 0.9.

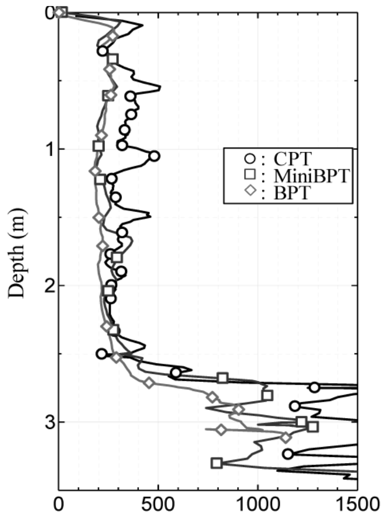

Fig. 4. Comparison of CPT and BPT

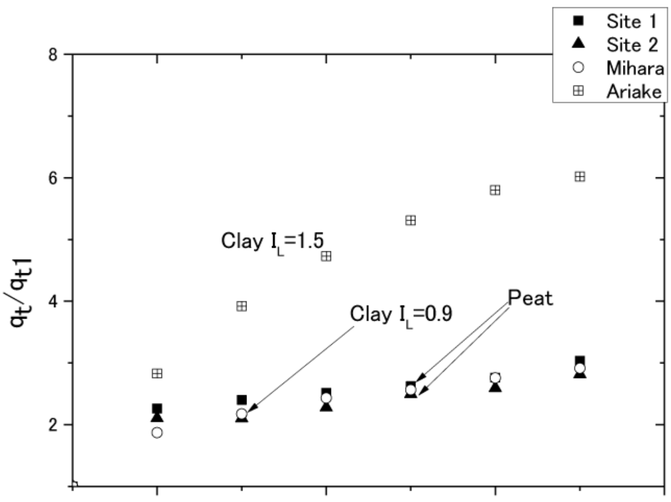

Fig. 5. Cyclic penetration of BPT

Sampling disturbance of peat soil

Since peat soil consists of fibers of plants and factions of woods, it seems difficult to obtain a high quality sample. Sample quality was evaluated by the bender element test. The testing and evaluation method may be referred by Horn, et al. (2010). Fig. 6 shows comparison between shear wave velocity measured by laboratory bender element (ν_{BE}) and that by seismic cone (ν_{SCPT}). All samples of clay as well as peat soil were retrieved by the Japanese thin wall sampler with fixed piston (Tanaka, 2000). The ν_{BE} values were measured under unconfined conditions. ν_{BE} is roughly 0.7 ν_{SCPT} and when this ratio is converted to G, the ratio of G becomes about 0.5. This reduction of G is considered to be due to loss of the effective stress in the sample and destruction of the soil structure by sampling. Before this experiment, it was anticipated that ν_{BE}/ν_{SCPT} for peat soils should be much smaller than that for usual clays because of difficulty in the sampling. However, this anticipation is not correct but the sample quality of peat soil is equivalent to the quality for usual clays.

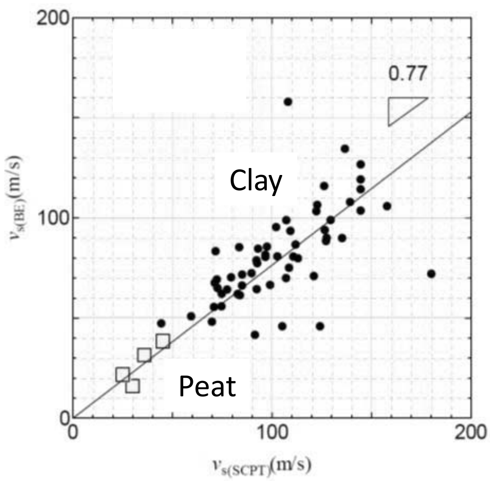

Fig. 6. Sample quality evaluated by shear wave velocity

Small strain stiffness of peat soil

Effect of consolidation pressure

Using the bender element, the stiffness was measured under different effective burden pressures (p') at laboratory in a consolidation cell, using the bender element. The size of the consolidation cell is 100 mm in diameter and about 150 mm in initial height. The bender element for transmitter is equipped in the cap and the receiver is in the pedestal. The peat sample was completely remolded and mixed with additional distill water. As the peat layer is located at shallow depths and its unit weight is very small, the range of the consolidation pressure is as small as between 2 and 20 kPa.

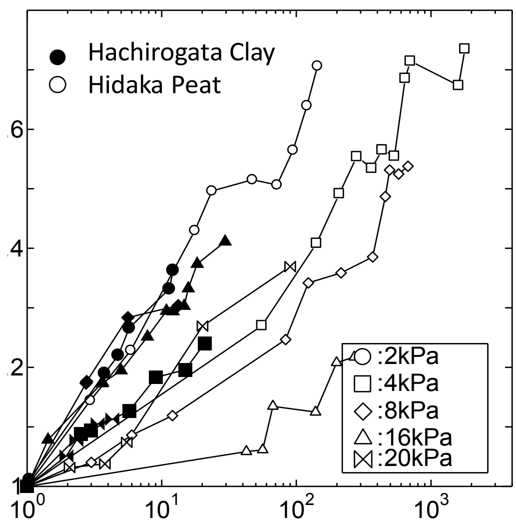

Fig. 7 Increase in G during secondary consolidation

It is known that for clay including peat, strength and shear modulus increases with time even after the end of primary consolidation. Such an increase in G after the end of primary consolidation (EOP) is compared for usual clay (Hachirogata clay) and peat (Hidaka peat) in Fig. 7. Time of EOP was obtained by the square root time method. As seen in the figure, the increase in G for one log time is less than 1.3 for both soils and that for Hidaka peat seems smaller than that for Hachirogata clay. Though the comparison was made for only two samples, it can be concluded that the effect of the secondary consolidation on the G increase for peat soil is not particularly significant.

Dependency of G on p' is shown in Fig. 8, where data measured by Ogino, et al. (2009) are also used for comparison. In this figure, all data for clay as well as peat are measured under normally consolidated state and G for Toyoura sand is measured for relatively dense conditions, i.e., $D_r = 0.74$. For all samples in Fig. 8, there is a liner relation between G and p' in logarithm scale. It is recognized that G for peat ground is considerably smaller than other clays and sand if it is compared at the same p'. This fact implies to give a reasonable explanation to the smallness of G/q_{net} ratio for peaty ground (see Fig. 2).

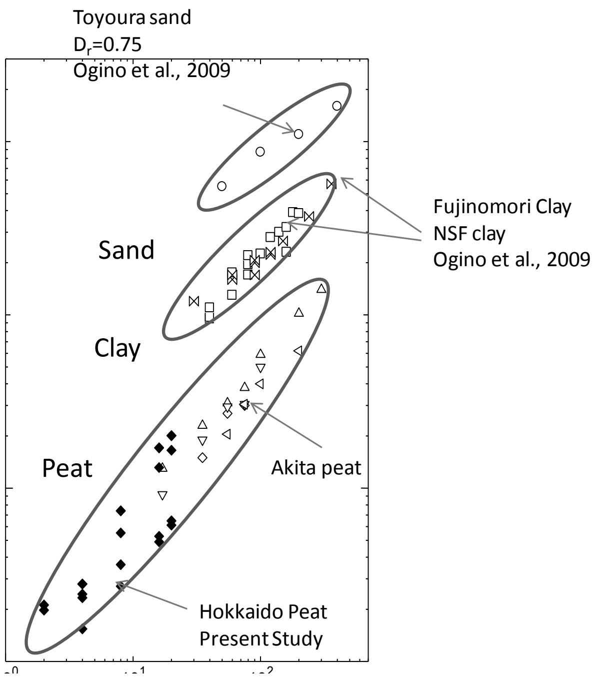

Fig. 8 Relationships between *G* and *p* for various soil materials

Strain dependency of *G*

It has been found from recent advanced measurements that deformational characteristics including *G* are strongly dependent of strain levels. The strain dependency for peat soil is shown in Fig. 9, compared with granular and clayey soil (Mexico clay with high I_p). The *G* for peaty soils was measured by torsional shear apparatus for intact sample under the in situ effective stress conditions (Hayashi, et al., 2011). The strain dependency of *G* for pear soil is similar to clayey soil with high plasticity. That is, the *G* for peat does not decrease until a certain large strain level. A similar result has also reported by Wehling et al., (2003).

Anisotropy of shear modulus

Anisotropy of stiffness was measured by the bender element for a sample of Sarobetsu peat. The specimen was obtained from a block sample excavated at about 0.5m depth and trimmed in the field as shown in Fig. 10. A pair of bender elements was inserted into the block for the vertical direction (G_{vh}), which is the same direction as the SCPT. In addition, the shear wave velocity was measured in the horizontal direction, where the bender element was inserted horizontally (G_{hh}) and vertically (G_{hv}). The wave patterns were sine and rectangular shape and the arrival time of the shear wave was the average of these waves. Measured results are indicated in Table 1. Theoretically, if the soil element is uniformly deformed during the propagation of the shear wave, G_{hv} should be equal to G_{vh}. However, as shown in the table, G_{hv} is slightly larger than G_{vh}. This indicates that the shear wave is propagated though stiff fibers or woods that are piled horizontally. The G_{hh} is about 2 times larger than G_{vh}, being the same as most soils.

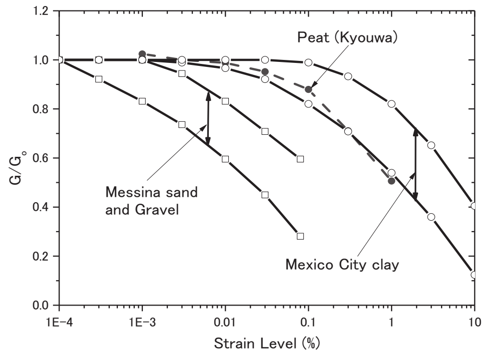

Fig. 9 Strain dependency of small strain stiffness. Lines other than peat are referred from Leroueil and Hight (2003).

Discussion and conclusions

Based on characteristics of peat soil described above, reasons for the extremely small ratio of G/q_{net} in comparison with that for usual clays will be considered. Soil disturbance is not so significant from sensitivity measured by cyclic BPT and the shear wave velocity measurement under unconfined conditions. The strain dependency of small stiffness is the same as clays with high I_p. These facts cannot explain the small G/q_{net} ratio. Instead, the reasons should be found in two other characteristics of peat soils: large q_{net} and small G. For example, as seen in Fig. 6, G for peat soil is about 1/5 times of G for usual clays. The q_{net} values at most sites are much larger that those estimated with large N_{kt} and s_u/p (Fig. 3). That is, q_{net} for peaty ground is considerably greater than that for usual clays. Den Haan and Kruse (2007) reported that s_u/p for Dutch peat is between 0.4 and 0.6, whose range is considerably larger than the usual clay. In addition, the possibility of drainage during the CPT penetration can be supposed, because q_{net} from BPT with large diameter is small and Bq parameter (not be presented in this paper) is rather small. These facts imply the excess pore water pressure caused by the CPT penetration is partially dissipated and observed q_t may be larger than that assuming that the penetration is executed completely under undrained conditions.

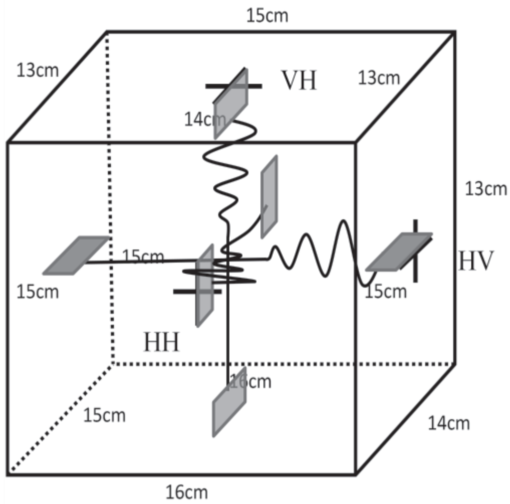

Fig. 10 Setting the bender element for measurement of anisotropy of *G*

Tab. 1 Measured *G* for different direction

	v_1(m/s)	v_2(m/s)	G/G_{vh}
G/G_{vh}	56.3	56.8	1
G/G_{vh}	76.2	88.7	2.13
G/G_{vh}	61.6	68.8	1.33

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Use of SDMT for the evaluation of the geotechnical parameters of organic soils

Zbigniew Lechowicz, Marek Bajda, Simon Rabarijoely,
Grzegorz Wrzesiński
Warsaw University of Life Sciences – SGGW, Faculty of Civil and Environmental Engineering, Department of Geotechnical Engineering, Warsaw, Poland

Abstract. The paper presents the results of field and laboratory tests performed on organic soils from the "Zoliborz channel" on the route of the II underground line in Warsaw. Seismic Dilatometer Tests (SDMT) were carried out in order to evaluate the geotechnical conditions in one of the most difficult sites in Warsaw. For comparison, results of SCPT test as well as oedometer tests and triaxial tests were analysed. The problem of evaluating the stress history, undrained shear strength and deformation parameters of organic soils from SDMT is discussed.

Key Words: organic soils, Seismic Dilatometer Tests SDMT, stress history, undrained shear strength, deformation parameters.

Introduction

In situ tests allow the investigation of soils in their natural intact state and stress condition, thus giving more accurate quantification of the stress history, shear strength and deformation parameters in soil profiles.

The overconsolidation ratio OCR, undrained shear strength and deformation parameters are the basic parameters for the geotechnical design of the structure. Determination of these parameters using dilatometer tests is usually based on empirical formulae. However, it should be noted that most of these formulae were established in different countries, therefore regional geotechnical conditions could have substantially affected the empirical relationships.

Due to their high compressibility and non-linear behaviour, organic soils require the application of a different description of behaviour and thus other values of coefficients used in the existing formulae for mineral soils. Experience from organic soils indicates that the stress history and current state of effective stress have significant influence on shear strength and deformation parameters (Bihs et al. 2013, Młynarek et al. 2013). This requires modifications of the existing formulae as proposed by Marchetti that could be used during the determination of geotechnical parameters in organic soils.

This paper presents site investigation and geotechnical characteristics of the area for the Plocka C08 Station of the II underground line in Warsaw. In this site occur organic soils, mainly organic muds and gyttja with a thickness reaching up to 10 m. Because of difficulties with the collection of undisturbed soil samples of soft soils, in situ tests, i.e. the Seismic Dilatometer Test (SDMT), were used besides drilling to provide shear wave velocity (V_S) measurements supplementing conventional readings. The use of SDMT was applied mainly to obtain a better interpretation of soil stratification and to provide information on the stress history, shear strength and deformation parameters of the organic soils. Stratigraphy and soil parameters were evaluated from three pressure readings whereas small strain stiffness (G_0) was obtained from in situ V_S profiles. Analysis of the test results have allowed presenting methods of evaluating stress history, undrained shear strength, and deformation parameters of organic soils from SDMT.

Site characteristics

Field investigations using SDMT were performed in the “Zoliborz channel”, one of the most difficult sites along the route of the II underground line in Warsaw. The construction of the Plocka C08 Station is planned in this site (Fig. 1). The “Zoliborz channel” is located in the western part of Warsaw; sedimentation of organic soils took place here during the Eemian Interglacial. The channel is about 12 km long and nearly 800 m wide in its central part. In the “Zoliborz channel”, the organic soils i.e. organic mud and gyttja, reach a thickness of up to 10 m.

The examined area is located within the Warsaw Basin, composed of Upper Cretaceous deposits, developed as marls and high plasticity marly clays, the top of which lies at approximately 250 m below ground level. The first subsurface layer in the tested subsoil is formed by fills with thickness varying between 0.5 m to 4.0 m. The fills are underlain by sand and mud deposits of the Vistulian Glaciation to the depth of approximately 4–6 m below ground level. The sand and mud layers cover the continuous layer of gyttja and organic mud from the Eemian Interglacial. The top of this layer was noted at the depth of approximately 6 to 16 m below ground level. Organic soils of the Eemian Interglacial are overconsolidated with an overconsolidation ratio OCR varying in the range between 2.0 and 3.5. The grain-size composition of the gyttja points to silts, silty clays and more plastic firm and stiff, silty clays, with the organic matter content from 10% to over 30%, locally even up to 50%. The index properties of organic soils are given in Table 1.

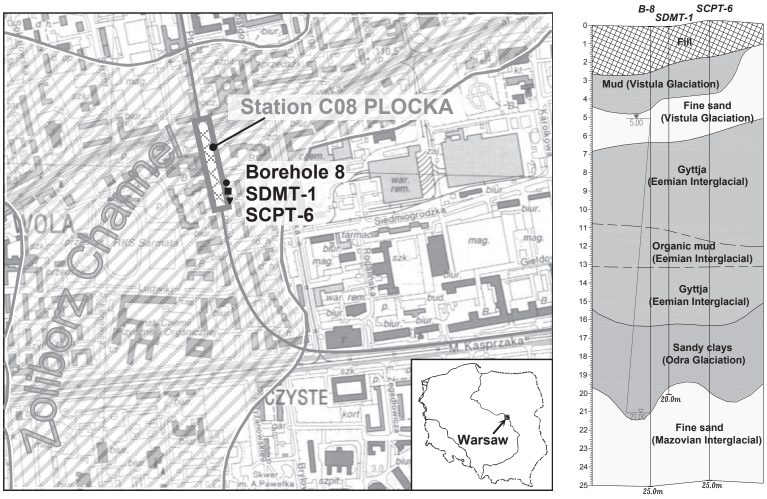

Fig. 1 – Location and typical cross-section of the test site

Tab. 1 – Index properties of the examined organic soils

Properties	Vistulian Glaciation (lacustrine deposits)	Eemian Interglacial (lacustrine deposits)		
	Mud M	Gyttja Gy (upper layer)	Organic mud M_{or}	Gyttja Gy (lower layer)
Water content w_n (%)	33-38	88-100	32-34	75-82
Plastic limit w_P (%)	28-30	80-85	30-32	70-75
Liquid limit w_L(%)	45-50	135-145	50-55	115-125
Plasticity index I_P (%)	17-20	55-60	20-23	45-50
Consistency index I_C (-)	0.60-0.70	0.75-0.85	0.90-0.95	0.85-0.90
Unit density ρ (t/m^3)	1.65-1.70	1.40-1.45	1.60-1.65	1.50-1.55
Specific density ρ_s (t/m^3)	2.50-2.55	1.95-2.00	2.40-2.50	2.05-2.10
Organic content I_{OM} (%)	6-9	40-48	8-12	35-37

The bottom of the channel is filled with moraine deposits of the Odra Glaciation, mainly comprising sandy clays, underlain by sandy deposits of the Mazovian Interglacial represented by dense, fine, medium and silty sands.

Profiles of data from the SDMT test with two geophones spaced at 0.5 m carried out in the subsoil of the Plocka Station, presented as profiles of the material index I_D, lateral stress index K_D, and dilatometer modulus E_D are shown in Figure 2. Profiles of pore pressure index U_D and shear wave velocity V_s are presented in Figure 3.

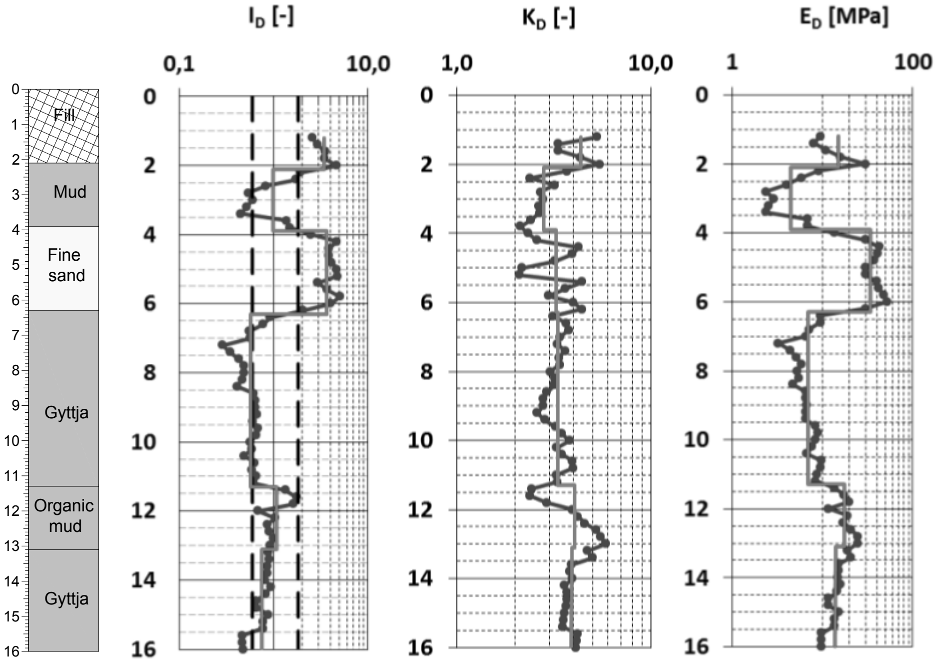

Fig. 2 – Profiles of I_D, K_D and E_D indexes from the SDMT test carried out in the subsoil of the Plocka Station

Lutenegger and Kabir (1988) have shown that in mineral soils the pore pressure index U_D estimated on pressure p_2 is very useful for the evaluation of the pore water pressure conditions. Experience from organic soils indicates that the dilatometer pore pressure index U_D is useful for identifying subsoil stratigraphy and pore water pressure conditions (Lechowicz and Rabarijoely 2000). In case of the subsoil of the Plocka Station, the values of the U_D index indicate that the layers of gyttja are in a preconsolidated state, whereas there is a possibility of horizontal drainage in more permeable interbeddings within the Eemian organic mud.

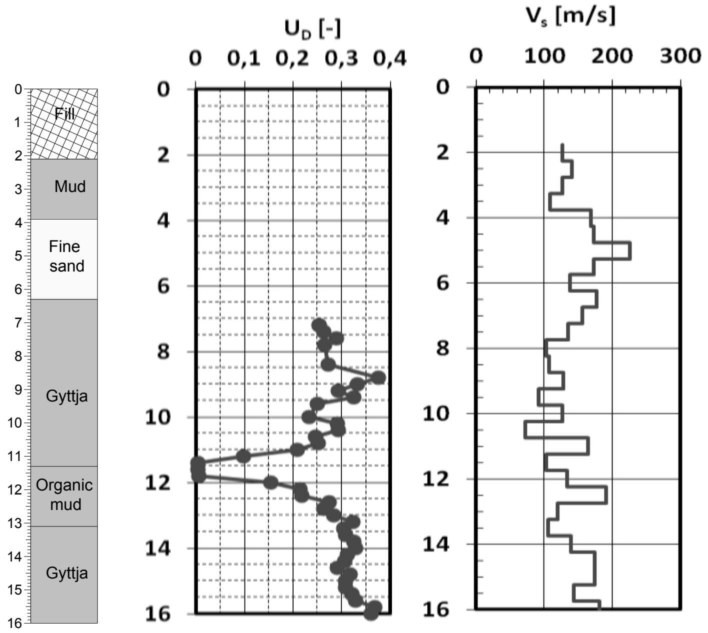

Fig. 3 – Profiles of the U_D index and V_s from the SDMT test carried out in the subsoil of the Plocka Station

Field and laboratory testing programme was carried out to evaluate soil parameters for foundation desing. Oedometer tests as well as CK_0U and CD traixial tests were performed on undisturbed soil samples. In the test site SCPT tests were also carried out.

Determination of the stress history

In practical geotechnical engineering, evaluation of stress history is based on the overconsolidation ratio OCR defined as:

$$OCR = \frac{\sigma'_p}{\sigma'_v} \quad (1)$$

where:
σ'_p – preconsolidation pressure,
σ'_v – vertical effective stress.

The value of the preconsolidation pressure σ'_p is usually determined based on the results of oedometer tests. In situ tests, i.e. dilatometer tests, which can characterize the variation of OCR with depth are valuable tools for geotechnical engineers.

Using the correlation between the OCR and the lateral stress index K_D for soils with the material index $I_D > 2.0$ and for cohesive soils where the material index I_D is smaller than 1.2, the following correlations were proposed by Marchetti (1980):

$$OCR = (0.67 \cdot K_D)^{1.91} \tag{2}$$

$$OCR = (0.5 \cdot K_D)^{1.56} \tag{3}$$

It is important to note that the estimation of the overconsolidation ratio OCR from dilatometer tests depends on empirical and local experience. Many studies have been performed to improve the original correlations proposed by Marchetti, however they were mostly limited to mineral soils (Briaud and Miran 1991, Kamei and Iwasaki 1994).

Experience from organic soils indicates that the relation between the OCR and the lateral stress index K_D is as follows (Lechowicz 1997a):

$$OCR = (0.45 \cdot K_D)^{1.40} \tag{4}$$

Using equation (4), the SDMT profiles, shown as values of the overconsolidation ratio OCR and the preconsolidation pressure σ'_p, were determined in the subsoil of the Plocka Station (Fig. 4).

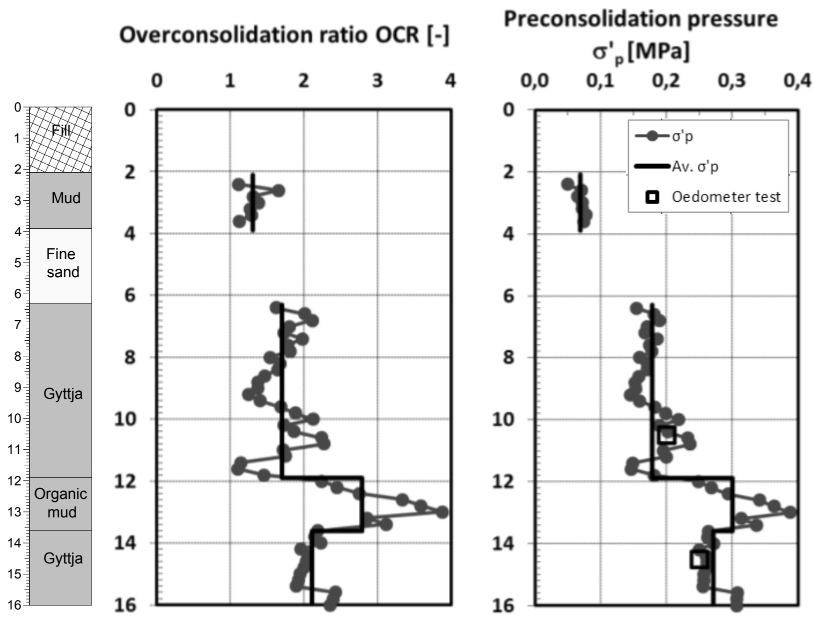

Fig. 4 – Profiles of the overconsolidation ratio OCR and the preconsolidation pressure σ'_p from the SDMT test and values obtained from the oedometer test for the subsoil of the Plocka Station

Undrained shear strength

The methodology of standard DMT and SDMT tests is widely known and a detailed procedure for conducting the test has been presented by Marchetti (1980), Marchetti et al. (2001), Marchetti et al. (2008). Marchetti (1980) proposed the following correlation between normalized undrained shear strength and lateral stress index K_D for cohesive soils ($I_D < 1.2$):

$$\frac{\tau_{fu}}{\sigma'_{v0}} = 0.22 \cdot (0.5 \cdot K_D)^{1.25} \tag{5}$$

where:
σ'_{v0}– in situ vertical effective stress.

The analysis carried out by Lechowicz (1997b) indicates that, particularly for organic soils, the relationship between normalized undrained shear strength and lateral stress index K_D differs from that proposed by Marchetti (1980) and can be modified as follows:

$$\frac{\tau_{fu}}{\sigma'_{v0}} = S \cdot (0.45 \cdot K_D)^{1.20} \tag{6}$$

where:
S – normalized undrained shear strength for a normally consolidated state.

Research on Holocene organic soils indicates that normalized undrained shear strength for the normally consolidated state S for mud and organic mud equals 0.35 and 0.40, whereas for calcareous gyttja, calcareous-organic gyttja and amorphous peat it equals 0.40, 0.45 and 0.50, respectively (Lechowicz 1997b). Analysis of laboratory test results has shown that values of the S coefficient for Eemian organic mud equals 0.30 and for gyttja equals 0.35.

Experience from organic soils indicates that for the evaluation of undrained shear strength from the dilatometer test, the following multifactor formula can be used (Rabarijoely 2000):

$$\tau_{fu} = \alpha_0 \cdot \sigma_{v0}^{\alpha_1} (p_0 - u_0)^{\alpha_2} \cdot (p_1 - u_0)^{\alpha_3} \tag{7}$$

where:
u_0 – in situ pore water pressure,
α_0, α_1, α_2, α_3 – empirical coefficients.

Analysis of the test results indicates that the obtained values of empirical coefficients for organic mud from the Plocka Station area are $\alpha_o = 1.12$, $\alpha_1 = 0.13$, $\alpha_2 = 0.10$,$\alpha_3 = 0.44$, and for gyttja – $\alpha_o = 1.25$, $\alpha_1 = 0.30$, $\alpha_2 = 0.12$, $\alpha_3 = 0.30$. Figure 5 presents profiles of undrained shear strength evaluated by the modified Marchetti as well as Rabarijoely (2000) formulae.

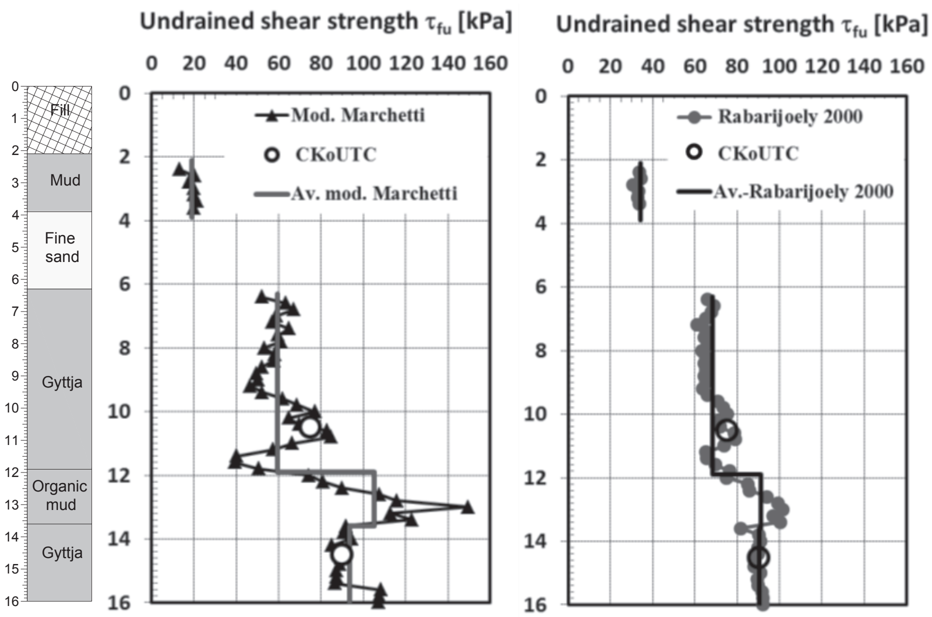

Fig. 5 – Undrained shear strength from the SDMT test and the CK_0U triaxial test for the subsoil of the Plocka Station

Deformation parameters

In 1980, Marchetti proposed a set of empirical correlations from which the constrained modulus M can be obtained by multiplying the dilatometer modulus E_D by the factor RM related to horizontal stress index K_D. For mineral soils formulae were determined that allow evaluating deformation parameters for different soil types. However, there are no such formulae for organic soils. Conducted research (Lechowicz et al. 2012) has allowed to determine the relations for the determination of the constrained modulus M for Eemian organic mud and gyttja:

$$M = R_M \cdot E_D \tag{8}$$

where:
$R_M = 0.14+2.36 \cdot \log K_D$ (for organic mud),
$R_M = 0.12+2.10 \cdot \log K_D$ (for gyttja).

In order to evaluate the deformation modulus $E_{0.1\%}$ at 0.1% of strain, the following relationship was proposed for Eemian organic soils (Lechowicz et al. 2012):

$$E_{0,1\%} = R_E \cdot E_D \tag{9}$$

where:
$R_E = 2.4 + 2.36 \cdot \log K_D$ (for organic mud),
$R_E = 2.15+2.10 \cdot \log K_D$ (for gyttja).

Figure 6 presents profiles of the constrained modulus M and the deformation modulus $E_{0.1\%}$ determined on formulae (8) and (9).

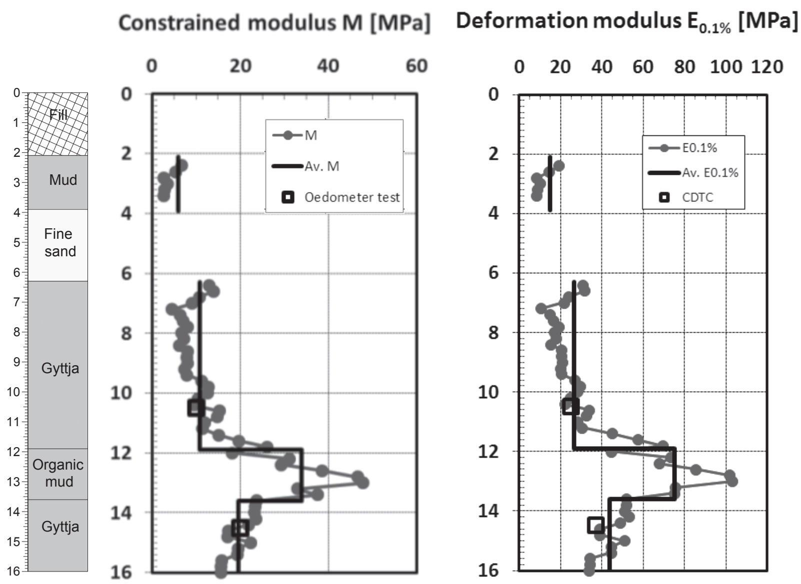

Fig. 6 – Constrained modulus M and deformation modulus E0.1% from the SDMT test and values obtained from the oedometer test and CD triaxial tests for the subsoil of the Plocka Station

The measurement of shear wave velocity gives the possibility to obtain the initial shear modulus of soil G_0 at a strain level less than 0.0001%. The elastic wave theory relates small strain shear modulus G_0 in MPa to shear wave velocity and unit density using the following equation:

$$G_0 = \rho \cdot V_s^2 \tag{10}$$

where:
ρ – soil mass density,
V_s – shear wave velocity.

Figure 7 shows the G_0 moduli obtained from equation (10) using shear wave velocity measurements during the SDMT test. For comparison, values of the G_0 modulus for gyttja obtained from triaxial tests with shear wave velocity measurements and from SCPT test with two geophones spaced 1.0 m are also presented.

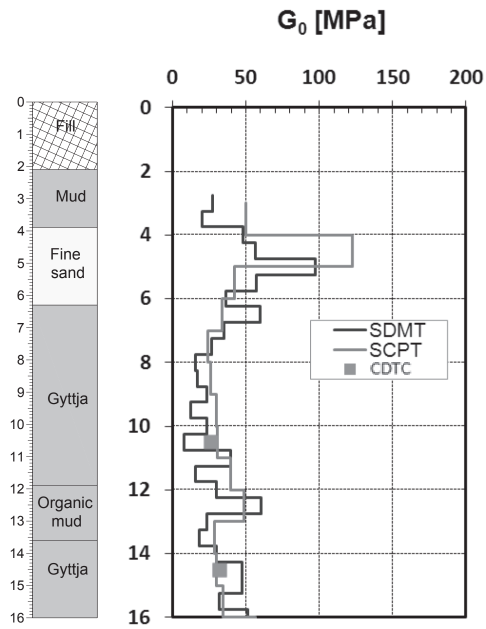

Fig. 7 – G_0 moduli obtained from the SDMT test and the SCPT test as well as the CD triaxial test for the subsoil of the Plocka Station

Conclusions

Based on SDMT tests carried out in one of the most difficult sites of the II underground line in Warsaw, the stress history, undrained shear strength and deformation parameters of Eemian organic soils were evaluated. A comparison between results obtained from SDMT tests and laboratory tests has been made.

On the basis of the presented test results, the formulae used for the interpretation of the dilatometer test to estimate the stress history, undrained shear strength τ_{fu}, constrained modulus M and deformation modulus $E_{0.1\%}$ of gyttja (Eemian Interglacial) were evaluated. The values of initial shear modulus G_0 at small strain obtained from SDMT and SCPT were compared with values evaluated by laboratory tests.

Further studies are necessary in order to verify the proposed correlations in other types of organic soils.

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Seismic DMT in a very soft organic clay

Graziella M.F. Jannuzzi, Fernando A.B. Danziger, Ian S.M. Martins
Federal University of Rio de Janeiro, Brazil

Abstract. Two series of seismic dilatometer tests have been performed in a very soft organic plastic clay deposit, at Sarapuí II test site, Brazil. The test procedures were presented, with special emphasis on the difficulties related to testing in a deposit with very low bearing capacity and high water level. The hammer was observed to be a key factor in the test results. The obtained values of shear wave velocity and maximum shear modulus are in the lower bound of the values found in the literature.

Key Words: SDMT, soft clay, soil investigation, shear wave velocity, shear modulus.

Introduction

The dilatometer test (DMT) was performed the first time in Brazil in 1985, by Tom Lunne and the late Marcio Miranda Soares in the very soft clay deposit of Sarapuí, near Rio de Janeiro (Soares et al., 1986). The tests have been performed in a joint research project on in situ tests in very soft clays between the Norwegian Geotechnical Institute (NGI) and Instituto Alberto Luiz Coimbra de Pós-Graduação e Pesquisa de Engenharia from Federal University of Rio de Janeiro (COPPE/UFRJ). Piezocone penetrometers used in the North Sea, from different manufacturers, as well as the dilatometer have been taken to Brazil by Tom Lunne. The first COPPE/UFRJ piezocone penetrometer was tested in the same research. Another series of DMT tests were carried out in 1992 (Vieira, 1994, Vieira et al., 1997) in the same test site. Recently, seismic dilatometer tests (SDMT) have been performed in Sarapuí II test site, 1.5 km from the early test site (Jannuzzi, 2013), as described below. The tests are presented herein, with special emphasis on the execution of the tests in a test site with some difficult conditions.

The test site

The Sarapuí soft clay test site has been used since the 1970s as a research site, and a number of in situ and laboratory tests have already been performed (e.g., Lacerda et al., 1977, Werneck et al., 1977, Ortigão et al., 1983). A comprehensive report about the deposit has been provided by Almeida and Marques (2002). Geotechnical characteristics of the soil are included in Fig. 1, based on investigations carried out near the trial embankments sites. The very soft organic clay layer is about 11 m thick, and overlyes sand layers. The plasticity index (IP) of the Sarapuí clay decreases with depth, from around 100% to 50%. Stress history and compressibility characteristics of the deposit are shown in Fig. 2.

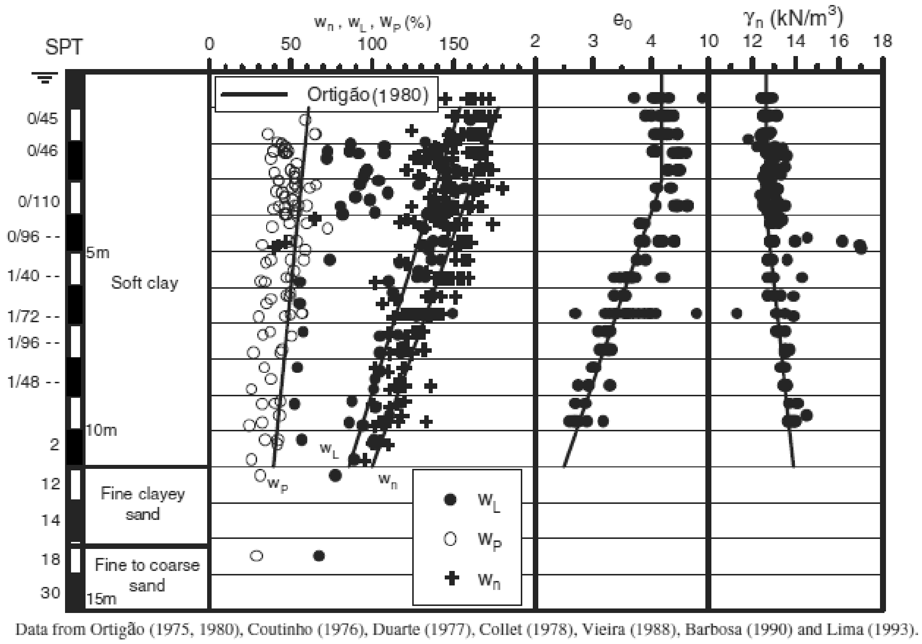
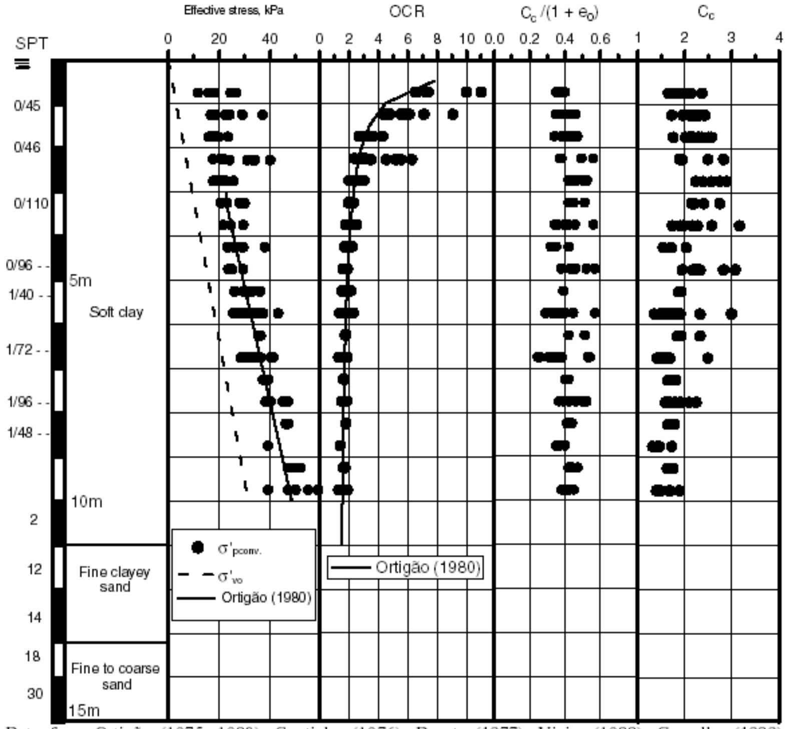


Fig.1. Characteristics of Sarapuí soft clay deposit (Almeida and Marques, 2002).



Data from Ortigão (1975, 1980), Coutinho (1976), Duarte (1977), Vieira (1988), Carvalho (1989), Barbosa (1990), Lima (1993) and Bezerra (1996).

Fig. 2. Stresses and compressibility parameter profiles (Almeida and Marques, 2002).

In the last fifteen years, however, security reasons have prevented the use of the test site. A new area (named Sarapuí II) in the same deposit, 1.5 km from the previous area and inside of a Navy Facility, has been used since then (Fig. 3). Two researches on pile behaviour have been carried out at Sarapuí II site (Alves, 2004, Francisco, 2004, Alves et al., 2009). The initial tests with the torpedo piezocone (Porto et al., 2010, Jannuzzi et al., 2010, Henriques Jr. et al., 2010) have already been performed at Sarapuí II test site. Although the whole deposit can be considered fairly homogeneous in horizontal directions, a number of in situ tests have been performed in this new area. In fact, 6 deployments of SPT's (performed each meter in Brazil), 7 CPTUs, 51 vane tests (in 5 deployments) and 4 T-bar tests have been performed (Jannuzzi, 2009). The very soft clay layer in this particular area varies from 6.5 m to 10 m. This new area is been used by the Research Center of the Brazilian Oil Company (CENPES/PETROBRAS) and Federal University of Rio de Janeiro as a state-of-the-art test site on very soft organic clay. Fig. 4 shows corrected cone resistance q_t, pore-pressures at cone face u_1 and cone shoulder u_2 v_s. depth from a typical piezocone test. It can be seen that the very soft clay layer is around 8 m deep, and a clayey-silt layer underlies the very soft clay. Further details of the in situ tests can be obtained in Jannuzzi (2009).

Fig. 3. Sarapuí II test site with respect to the early Sarapuí I test site.

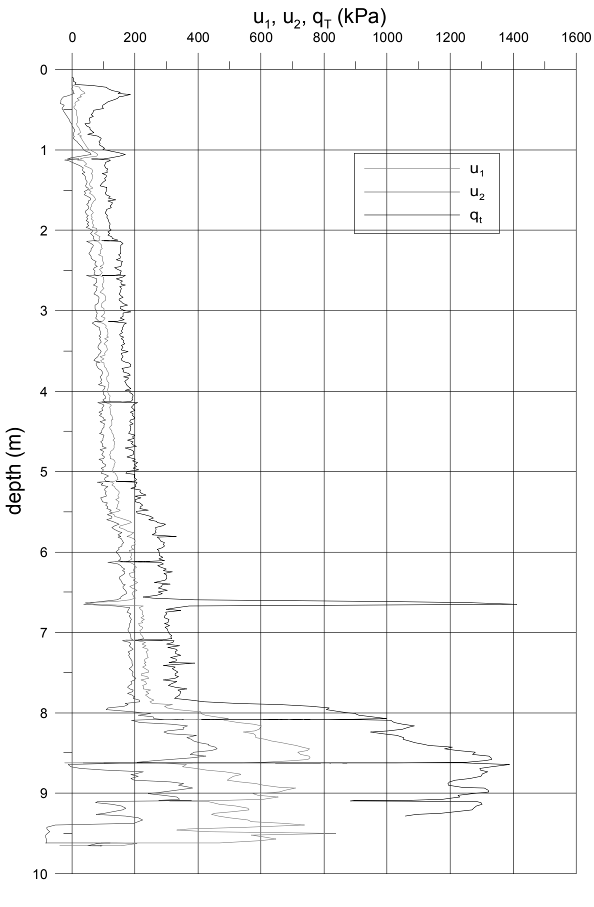


Fig. 4. Typical results of cone resistance (q_t) and pore pressure at cone face (u_1) and cone shoulder (u_2) vs. depth, Sarapuí II soft clay test site (Jannuzzi,2009).

Tests performed

Two series of SDMT tests have been performed in 2012, by Jannuzzi (2013) as indicated in table 1. The second series of tests aimed at the performance of tests after some problems verified in the first test series. In both series the dilatometer tests have been performed every 0.20 m, and the seismic tests every 0.50 m.

Tab. 1 – SDMT tests performed.

Test series	Test designation	Date	Maximum depth (m)
1	01/01	13 and 14.06.2012	9.2
	02/01	15.06.2012	9.4
2	01/02	12.12.2012	8.4
	02/02	13 and 14.12.2012	9.0

First Series of Tests

The water table at the test site during the first series of tests was approximately 0.30 m above ground level, which was an additional difficulty related to the performance of tests in the Sarapuí II test site. Fig. 5 illustrates the support usually used to allow the rig to move in the very soft clay area. The seismic dilatometer poses an additional difficulty with respect to the dilatometer test, because the seismic device requires a larger distance between the rig and the soil, as illustrated in Fig. 6. Besides, to place the hammer in the proper position is a cumbersome operation. Initially, the roots of the vegetation must be removed (Fig. 7), then the hammer is placed on the soil surface under water, with extreme care in order not to induce failure on the very soft material (Fig. 8). To complete the operation a load must be added to the hammer in order to keep it stable during hammer striking (Fig. 9). Fig. 10 illustrates the initial operation of the test.

Fig. 5. Wood support needed to move and hold the rig in the test position.

Fig. 6. Rig during SDMT test in Sarapuí II clay.

Fig. 7. Careful removal of the roots of the vegetation to provide room for the hammer.

Fig.8. Placement of the hammer on the soil surface, under water.

Fig. 9. Hammer in position for the test, with some weight on the top to keep it stable during striking.

Fig. 10. Handling the seismic dilatometer in the beginning of the test.

The operation of the test is also difficult, provided the hammer head splashes water around. Fig. 11a illustrates the beginning of the seismic test and Fig. 11b a situation where the weight on the hammer was not enough to keep it stable and the weight of a person was needed for that purpose. Fig. 12 shows a general view of the whole equipment during a test.

(a) (b)

Fig.11. (a) Beginning of a seismic test; (b) Protection against water splashing when the weight of a person is required to keep the hammer stable.

Fig.12. Test operation.

During the first test the dilatometer membrane failed due to an error in the execution of the test. However, the seismic tests have been performed anyway. The seismic tests performed in the second deployment provided essentially the same results of shear wave velocity (v_S) obtained in the first deployment, even with the membrane failure. Despite the similarity between the v_S values of the first and the second deployments, in both cases the shear wave did not present a smooth behavior, as shown in Fig. 13. This was attributed to gaps in the hammer, which therefore needed to be improved.

A curious observation from the tests is that the seismic test may be affected by magnetic fields from mobile phones, which therefore must be kept far away from the equipment.

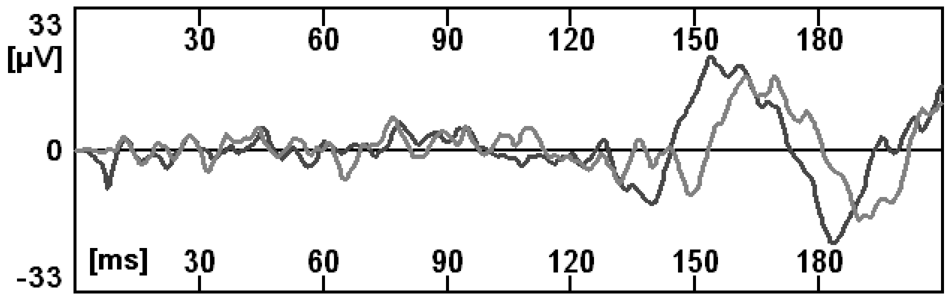


Fig. 13. Shear wave in SDMT02/01, 6.0 m depth, v_S = 56 m/s.

Second Series of Tests

As mentioned before, the hammer needed to be improved, and special care was taken with respect to gaps that prevent the generation of an adequate wave. Fig. 14 shows the hammer after the adjustments carried out, in a simple test of stability during striking.

Fig. 14. Hammer during test in the laboratory.

The water level was approximately coincident with the ground level during the second series of tests, thus the execution of the tests was not so difficult as in the first series of tests. Fig. 15 shows the onset of a seismic test and Fig. 16 shows the detail of the hammer with more weight than in the first series of tests, also replacing the need of a person to supply weight.

Fig. 15. Onset of a seismic test.

Figure 16 – Detail of the hammer with weight on its top, during a SDMT test.

Test results

The “intermediate” DMT parameters, material index I_D, dilatometer modulus E_D, and horizontal stress index K_D, are presented in Fig. 17. The interpretation of the dilatometer test results is outside the scope of the present paper, which refers only to the seismic tests. However, it must be noted that a significant scatter was obtained in the case of I_D and E_D, but not with K_D. This is due to the fact that both I_D and E_D depend on the difference between p1 (the corrected first reading) and p_0 (the corrected second reading), very small for the case of the Sarapuí II very soft clay, while K_D depends only on p_0.

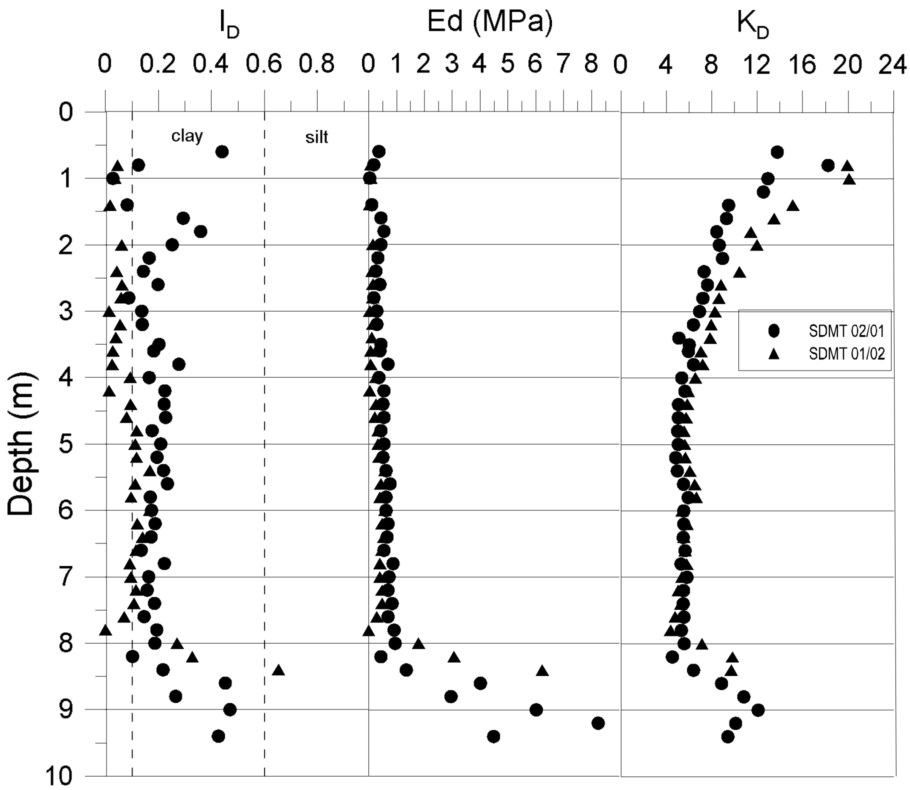


Fig. 17. I_D, E_D and K_D versus depth.

Fig. 18 presents the results of a SDMT test performed at 2.0 m depth in the first series of tests. Fig. 18a shows the recorded original signals already filtered, Fig. 18b the part of the signal used for calculation of v_s (windowed signals) and Fig. 18c the rephased signals. Fig. 19 presents the results of a SDMT test performed at 2.00 m depth in the second series of tests, i.e. after the improvement of the hammer, where Figs. 19a, 19b and 19c correspond to Figs. 18a, 18b and 18c. The improvement of the quality of signals is quite clear.

The values of vs obtained in the tests from both series are presented in Fig. 20. The vs values of the the first series of tests are in the upper range of the results of the second series of tests. Due to the quality of the signals, the second series of tests have been used to the calculation of G_0. In general, however, the difference in the values of v_s between both series of tests was not as large as it would be expected from the difference in the aspect of the signals, showing that the software used to obtain the v_s values is quite efficient.

It must also be noted that the scatter of v_s values was larger at 1.0 m depth. In fact, it is recommended (Marchetti et al., 2008) that the seismic tests must be performed from 1.5 m depth. However, the authors of the present paper decided to perform the test also at 1.0 m depth, which is only possible if a suction is applied to the membrane, with the syringe used for the calibration, provided the soil in this depth has a very small horizontal stress, not enough to bring the membrane to the initial position, as requested in the seismic test. This also occurred in other tests performed in very shallow depths.

Values of G_0 (or G_{max}) obtained from the tests performed in the second series of tests are shown in Fig. 21, in a magnified scale, where it can be observed that they decrease with depth to 3m, then increase to the bottom of the very soft material. The obtained values are in the lower bound of the values found in the literature, as presented in Fig. 22, where the values obtained herein are plotted in the chart which correlates G_0 versus q_t from data in a number of clays, collected by Mayne and Rix (1993).

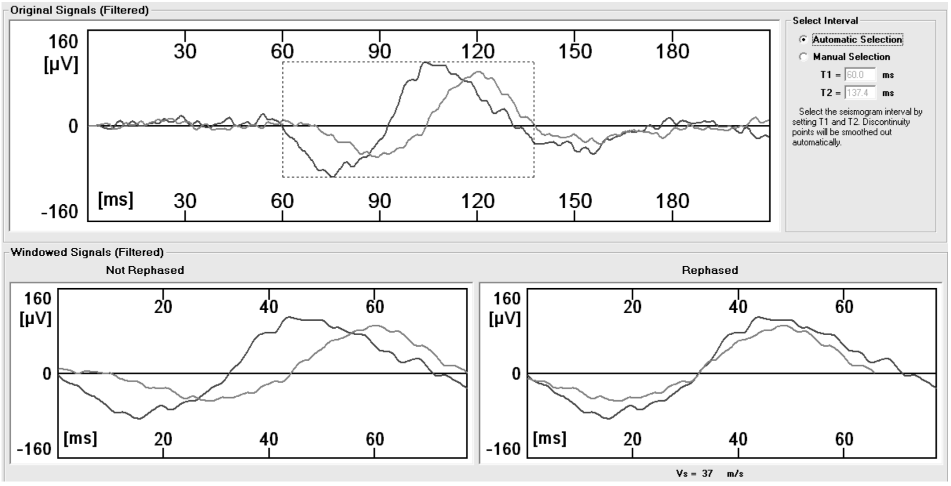


Fig.18. Seismic dilatometer test performed in the first series of tests, 2.0 m depth.

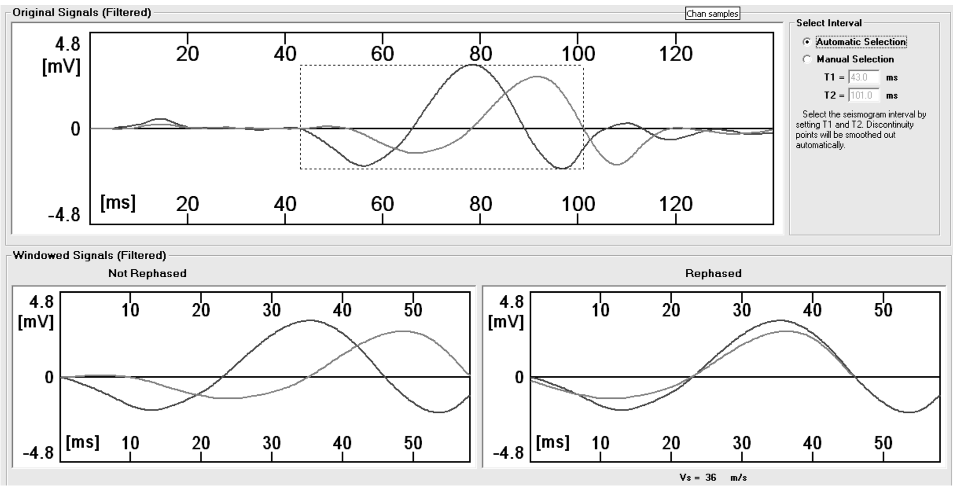


Fig. 19. Seismic dilatometer test performed in the second series of tests, 2.0 m depth.

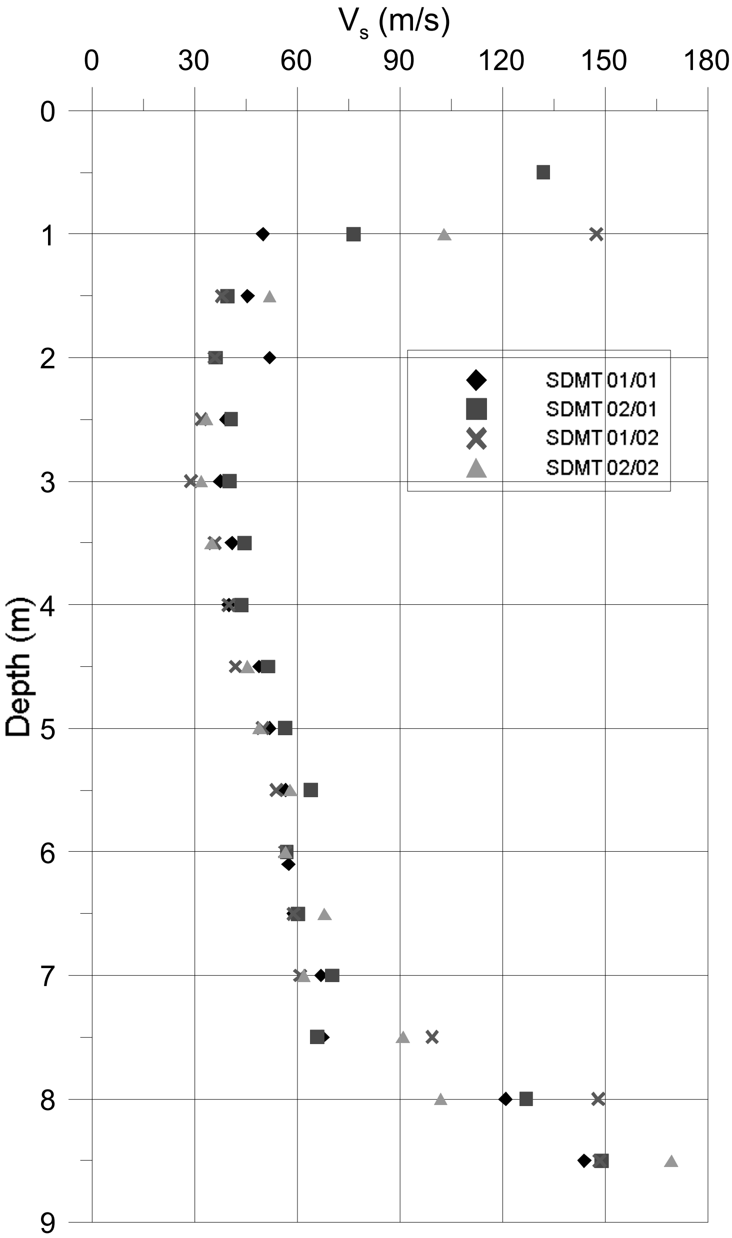


Fig. 20. v_S values from two series of SDMT tests.

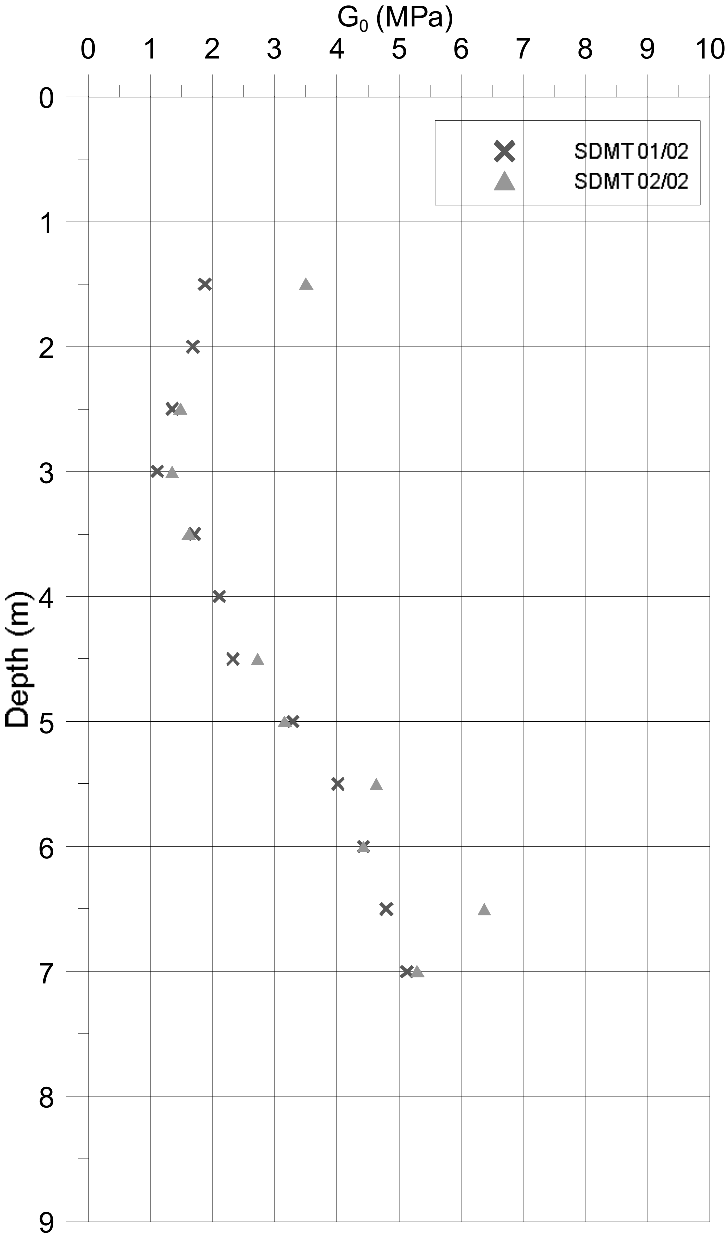


Fig. 21. G_0 values, second series of SDMT tests.

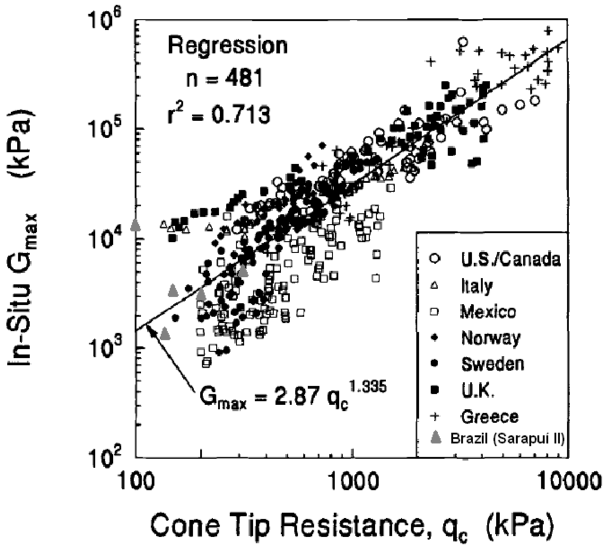


Fig. 22. Apparent relation between maximum shear modulus (G_{max}) from in-situ measurements and cone resistance (q_c) for clays located worldwide (modified after Mayne and Rix, 1993).

Conclusions

Two series of seismic dilatometer tests have been performed in a very soft organic plastic clay deposit, at Sarapuí II test site, near Rio de Janeiro, Brazil. The test procedures were presented, with special emphasis on the difficulties related to testing in a deposit with very low bearing capacity and high water level. The hammer was observed to be a key factor in the test results. However, the software used to calculate the values of shear wave velocity was able to provide consistent results even without good quality signals. The obtained values of shear wave velocity and maximum shear modulus are in the lower bound of the values found in the literature.

Acknowledgments

Diego Marchetti, from Marchetti Inc., for help in the performance and interpretation of SDMT tests.

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Some aspects of site characterization by flat dilatometer test (DMT) and cone penetration test (CPTU)

Specific aspects, not widely known, of DMT/SDMT

Silvano Marchetti
University of L'Aquila, Italy

Abstract. Aim of this paper is to possibly clarify questions that from time to time have arisen during the application of the geotechnical parameters estimated by DMT to the design of geotechnical works. In general the TC16 DMT Report (2001) has proved to be a rather complete document, describing in detail the use and interpretation of the DMT. However, in the years, a number of specific aspects have been recognized in need of clarification.

Legitimacy of treating the constrained modulus as a constant even when the applied load is not small

It is well known that oedometer moduli M are not constant, but vary with the applied vertical load. In particular the oedometer modulus increases up to the maximum past pressure p_c. At p_c, break point in the e-log p curve, the modulus decreases, to increase again at higher loads. Therefore the average modulus to be used for predicting settlement should in principle be chosen as the average modulus in the interval between the initial and the final vertical load. This can be done if the e-log p curve from the oedometer is available, but cannot be done if only the constrained modulus at geostatic stress is available. Since the target of DMT is specifically the 1-D modulus at vertical geostatic stress, and since DMT does not provide information on modulus at stresses higher than geostatic, predicting settlements using M_{DMT} involves approximation.

Fig. 1 shows schematically two typical e-log p oedometer curves, and the values of the moduli M at various applied vertical load p. In many natural soils, with the exception of highly structured clays, where the break is sharp, the variation of the modulus across p_c is moderate. Hence the error in assuming $M \approx$ constant is often relatively acceptable for practical purposes. This assertion is supported by the large number of case histories in the recent decades indicating good agreement between observed and DMT-predicted settlements. On the other hand moduli estimated by alternative methods are not rarely affected by errors (e.g. disturbed samples) much larger than the mentioned approximation. It is reminded that M_{DMT} provides an estimate of the operative modulus during the consolidation. Hence the predicted settlement is the primary settlement, and does not include the secondary settlement.

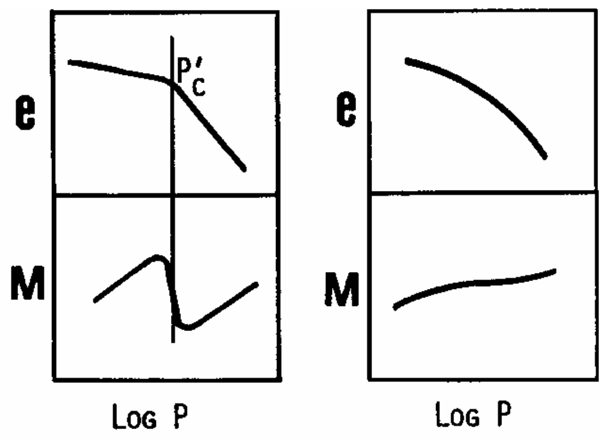

Fig.1.1 Schematic variation of the oedometer (constrained) moduli with applied load

Possibility of deriving the constrained modulus in clay from the undrained expansion of the membrane

In clay, the expansion of the membrane occurs in undrained conditions. Therefore the dilatometer modulus E_D is an undrained modulus. Thus, according to logic, the correlation to be investigated should be between E_D and the undrained modulus E_u. Attempts of this kind were carried out in the early days of the DMT development. However a big obstacle, precluding such possibility, was the high variability of the undrained moduli provided by different laboratories, at least in part due to the high sensitivity of E_u to the disturbance. Hence, as a second attempt, the correlation E_D - M was investigated. This correlation involves many soil properties, including material type, anisotropy, pore pressure parameters etc. Hence no unique E_D - M correlation can be expected. On the other hand the DMT provides, in addition to E_D, also the parameters I_D and K_D containing at least some information on material type and stress history. This availability provides some basis to expect at least some degree of correlation E_D - M , using I_D and K_D as parameters. Moreover, while the correlation E_D - M is, at least in principle, weaker than E_D -E_u, at least E_D - M can be tested, because M by different laboratories have much less variability than E_u.

Obviously the final word goes to real world observations. Large number of case histories have generally proved the favorable comparisons between observed and DMT-predicted primary settlements, thereby supporting the use of M_{DMT} as operative constrained modulus.

Note also Lambe et al. (1977): “Drained moduli of saturated clays are typically about one-third to one-fourth the undrained values”. Hence a broad connection drained-undrained stiffness had already been invoked in the past.

Reliability of the material index I_D to identify sand, silt, clay

The DMT Material Index I_D is a useful index to identify the soil composition (sand, silt, clay). Even a non experienced operator, after a few minutes, notes that in sand the two readings p_o and p_1 are distant (e.g. 5 and 20 bar), while in clay they are close (e.g. 5 and 6 bar). Obviously I_D is not the equivalent of a sieve analysis. The sieve analysis defines soil composition based on grain size, I_D based on the mechanical response in terms of p_o and p_1. Note also that a mixture of sand and clay would probably be „wrongly” interpreted by I_D as a silt. On the other hand such mixture will probably behave mechanically as a silt. Since the engineer is often interested in the grain size distribution not „per se”, but indirectly, just to infer from the grain size the mechanical properties, perhaps, in some cases, it could be more expressive, for association with the mechanical behavior (a sort of Soil Behavior Type Index), the soil description based on I_D than the description based on the sieve analysis. I_D is a sort of ratio stiffness to strength, i.e. a mechanically connected index, hence it is indicative of aspects of the soil mechanical behavior rather than of the grain size, though the two things are to some extent correlated.

Current status of the methods for estimating Ko and OCR in sand

In the last 30 years many efforts have been carried out to develop correlations between DMT results and Ko and OCR in sand. These studies have been described in various successive papers. This section aims at providing a summary of the successive advancements. Only clean sand is being considered.

It has been often pointed out in the past that, while the correlation $K_0 = (K_D / 1.5)^{0.47} - 0.6$ (Marchetti 1980) provides reasonable, though approximate, K_0 estimates in clay, in sands a similar unique one-to-one correlation does not exist, because the K_0- K_D correlation is substantially dependent on ϕ (or D_r).

This dependency was pointed out by Schmertmann (1982, 1983) who, based on calibration chamber (CC) results, developed the following K_0 - K_D -ϕ correlation equation

$$K_o = \frac{40 + 23K_D - 86K_D(1-\sin\phi'_{ax}) + 152(1-\sin\phi'_{ax}) - 717(1-\sin\phi'_{ax})^2}{192 - 717(1-\sin\phi'_{ax})} \quad (4.1)$$

Fig. 4.1, the graphical equivalent of Eq. 4.1, clearly shows that the K_0 - K_D correlation is not unique, but depends on ϕ. Fig. 4.1 is physically understandable, because, in a dense (high ϕ) sand, only part of the high K_D is due to K_0 , the remaining part being due to the high density.

It may be noted that Eq.1, to provide a value for K_0, requires the knowledge of ϕ, usually unknown too. Therefore Schmertmann (1983) suggested to supplement K_D from DMT with an additional information, namely q_c from CPT (or q_d, the dilatometer tip resistance). Once K_D and q_c are known, both the unknowns *Ko* and ϕ can be simultaneously determined. For such determination Schmertmann suggested to combine Eq.1 with the Durgunoglu & Mitchell (D&M 1975) theory, which establishes an interrelationship between q_c and the same two unknowns *Ko* and ϕ, i.e. q_c = f(*Ko*, ϕ).

The D&M equations are rather complex, but the relationship q_c = f(*Ko*, ϕ) can be readily understood in Fig. 4.2, which is a slightly simplified graphic equivalent of D&M. Fig. 4.2 provides an estimate of q_c as a function of ϕ but also taking into account *Ko*.

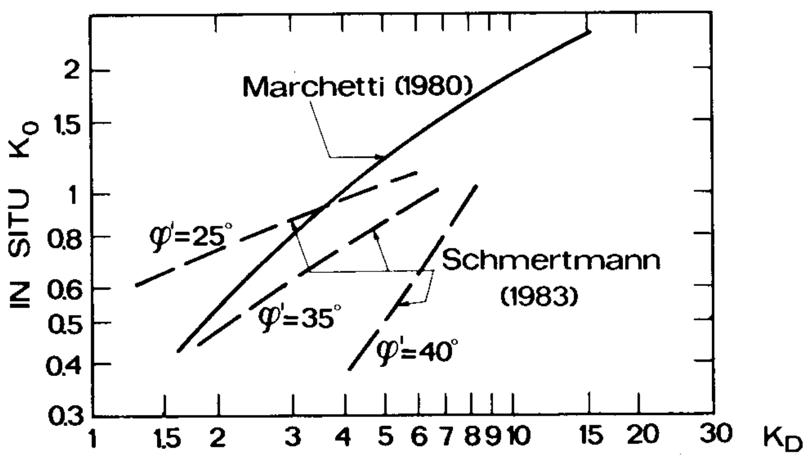

Fig. 4.1 Correlations Ko vs Kd as a function of Ø

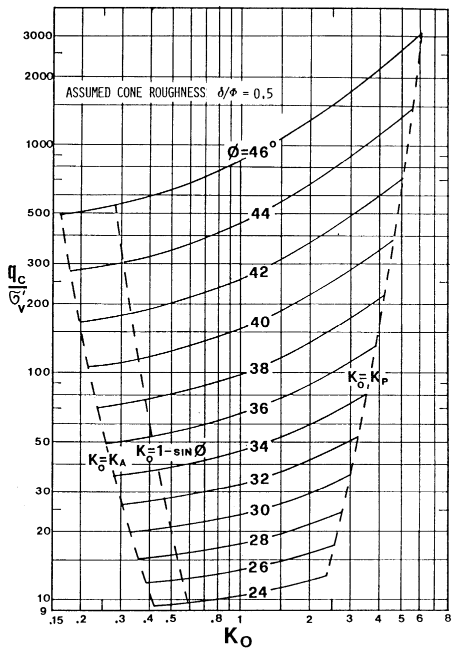

Fig. 4.2 Interrelationship Qc-Ø-Ko by Durgunoglu and Mitchell (1975). Converted in graphical format by Marchetti 1985.

In order to find the two unknowns Ko and ϕ, the following system of two equations has to be solved :

$$\begin{cases} K_D = f_1\,(Ko, \phi) & \text{(is the Eq. 4.1 above, solved for } K_D\text{)} \quad (4.2) \\ q_c = f_2\,(Ko, \phi) & \text{(D\&M theory)} \quad (4.3) \end{cases}$$

The system is well conditioned because, while q_c and K_D depend on both Ko and ϕ, q_c reflects more ϕ, K_D reflects more Ko (Marchetti 1985).

The system can be solved by an iterative procedure described in detail by Schmertmann (1983). Since the D&M equations are rather complex, the iterative procedure is generally performed by computer. In principle the iterative procedure proceeds as follows. K_D and q_c have been determined in the field. Then one assumes an initial trial value of ϕ. By inserting K_D and the trial value of ϕ in Eq. 4.1, a value for Ko is obtained. Then Eq. 4.3, entered with q_c and Ko, provides an estimate of ϕ. If this ϕ does not coincide with the initial trial value of ϕ, a new trial value of ϕ is assumed. The sequence is stopped when there is coincidence between the trial estimate of ϕ and the subsequently calculated value of ϕ.

To avoid the iterative procedure and simplify the calculations, Marchetti (1985) prepared a K_0 - q_c -K_D chart (Fig. 4.3), obtained by eliminating ϕ in the above system of Eqns. 4.2 and 4.3.

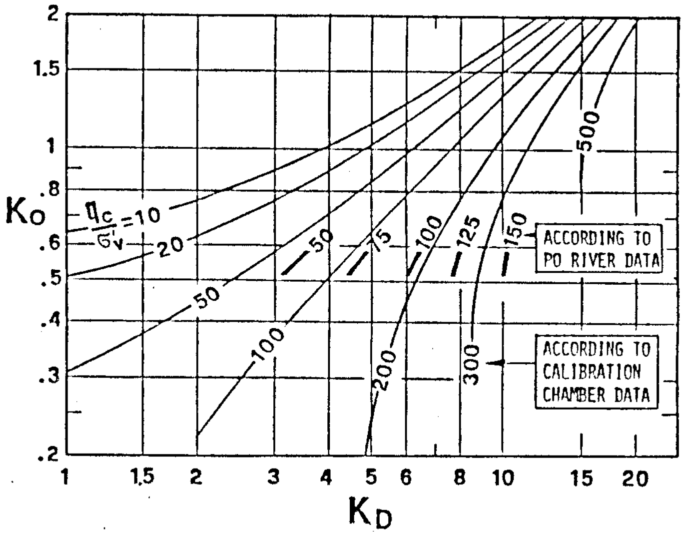

Fig. 4.3 Chart for evaluating $Ko = f(K_D, q_c/\sigma'_v)$. Marchetti 1985

Fig. 4.3 provides K_0, once q_c and K_D are given. Once Ko has been estimated, then ϕ can be read from the chart in Fig. 4.2. The compact Ko chart in Fig. 4.3 is very similar to Fig. 4.1, with the replacement of the parameter $Ø$ with the parameter q_c/σ'_v (of course the curves are not the same). The chart expresses Ko as a function of two highly reproducible measurements (K_D and q_c), leaving not involved ϕ (or worse D_r), an unneeded potential source of uncertainty.

Fig. 4.3 shows, besides the continuous curves, derived from calibration chamber tests, an additional scale for the parameter q_c/σ'_v, derived from 25 DMT datapoints in the Po river sand (Marchetti 1985). Fig. 4.3 may be used to obtain an approximate estimate of the expectable Ko range.

Baldi et al. (1986) enriched such K_0 -q_c -K_D chart with additional CC work. Moreover the chart was converted into simple algebraic equations:

$$K_0 = 0.376 + 0.095\, K_D - 0.0017\, q_c/\sigma'_{v0} \tag{4.4}$$

$$K_0 = 0.376 + 0.095\, K_D - 0.0046\, q_c/\sigma'_{v0} \tag{4.5}$$

Eq. 4.4 was determined as the best fit of CC data, obtained on artificial sand, while Eq. 4.5 was obtained by modifying the last coefficient to predict „correctly" K_0 for the natural Po river sand. Eqns. 4.4 and 4.5 should be used with the following values of the last coefficient: 0.005 in „seasoned" sand, 0.002 in „freshly deposited" sand. This choice involves appreciable subjectivity. Possibly a help for this choice could come from using the value of K_D, e.g. assuming that the sand is fresh if K_D = 1-2, or is seasoned for K_D = 5-6. However no specific study has been conducted on this possibility.

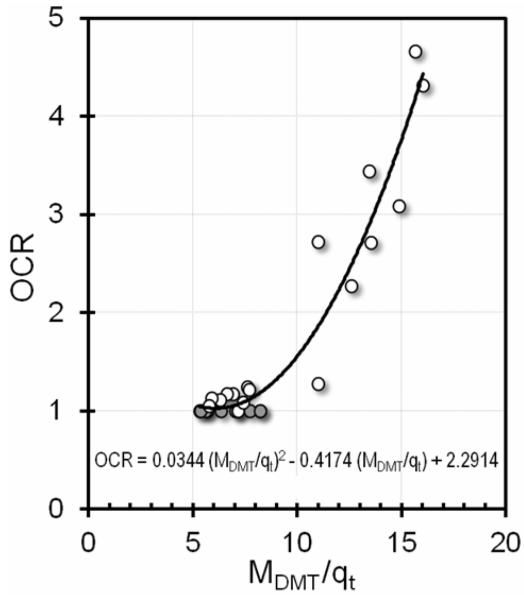

Fig. 4.4 Correlation OCR = f(M_{DMT} / q_c) for the Venice Lagoon Sandy layers (Monaco et al. 2014)

A third way for estimating *Ko* is to first estimate OCR, then to infer *Ko* from OCR. The currently preferred method for estimating OCR in sand, though highly approximate, is the method making use of the ratio between the constrained modulus *M* from DMT (M_{DMT}) and the cone penetration resistance q_c from the cone penetration test (CPT). Note that even this method needs both CPT and DMT. The semiquantitative guidelines reported in Marchetti et al. (2001) are M_{DMT}/q_c = 5-10 in NC sands, M_{DMT}/q_c = 12-24 in OC sands.

Based on OCR - M_{DMT}/q_c datapoints from an experimental research in Venice area, Monaco et al. (2014) found the correlation OCR - M_{DMT}/q_c in Fig. 4.4 , which is in quite good agreement with the above indicated semiquantitative guidelines. Fig. 4.4 is the currently recommended curve to estimate OCR in sand.

Once having estimated OCR, the Schmidt equation can be used to estimate *Ko*:

$$Ko = K_{o,nc}\, \mathrm{OCR}^{m} \quad \text{with m= 0.4 to 0.5} \qquad (4.6)$$

Conclusion of this section. Estimates of *Ko* in clean sand can be obtained in three possible ways: (a) Using Fig. 4.3 (b) Using Eqns 4.4 and 4.5 (c) Estimating first OCR using Fig. 4.4, then inferring *Ko* from OCR. All the three estimates are highly approximate. All the three estimates are based on a multi-parameter approach, i.e. the combined use of DMT and CPT.

Reason of the central deflection of the membrane S=1.10 mm

The central deflection S of the membrane cannot be excessive, otherwise the expanded membrane is no longer flat and flexible. The steel will start working in tension - rather than in bending - and the membrane correction *DeltaB* increases rapidly with S. In this case the majority of the inside gas pressure is spent to expand the membrane and only a small part will act to move the soil. On the other hand if the central deflection is very small, then the difference p_1 - p_0 will also be very small, with appreciable potential error. Trials were executed with S in the range 0.8 and 1.2 mm. Finally the deflection 1 mm was selected, as a good compromise between the two previously outlined values.

It is noted that experiments carried out with instrumented research dilatometers have shown that the pressure-deflection curve during inflation is essentially linear, hence S could indifferently be any of the above values, in all cases obtaining the same E_D - obviously inserting in the elasticity formula the appropriate value of S.

A few years after the start of the DMT a mechanical simplification was introduced, which had as a consequence the necessity of modifying the 1 mm into 1.10 mm, which has never been changed thereafter. Thus the modification 1 mm into 1.10 mm has by no means geotechnical motivations.

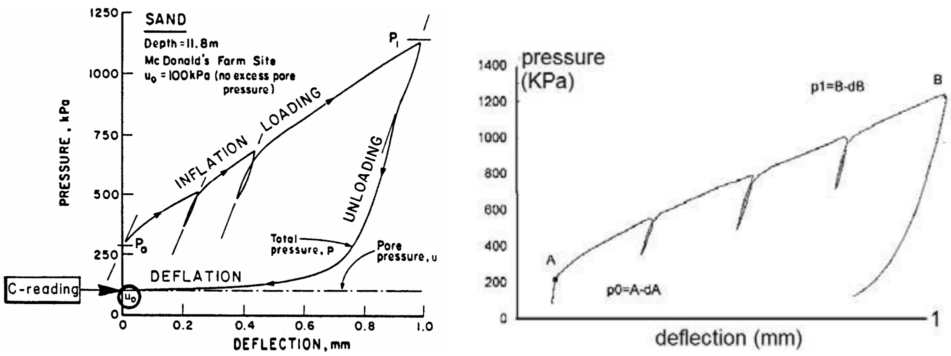

Fig. 5.1 Examples showing the linearity of the pressure-deflection dilatometer curve during inflation. (a) Campanella et al. 1985 (b) Fretti et al. 1992

Compaction specifications in terms of D_r or in terms of modulus

Determining D_r in situ - generally based on the use of q_c – involves a big uncertainty. q_c is indeed sensitive to D_r , but there is no unique $q_c - D_r$ correlation applicable to all sands. As pointed out by Robertson and Campanella (1983) Hilton Mines sand at D_r =60% has the same q_c as Monterey sand at D_r =40%, i.e. each sand has a different $q_c - D_r$ correlation (Fig. 6.1). Therefore setting compaction specifications in terms of D_r, difficult to measure, may later generate controversy.

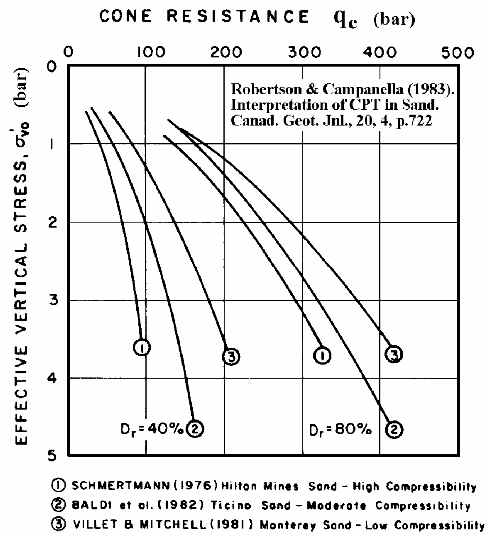

Fig. 6.1 Example of different curves for interpreting D_r from q_c.

As noted by Schmertmann et al. (1986) since the primary objective of compaction is to reduce settlements, it might be more rational to set the specifications in terms of M (e.g. estimated by DMT), since M relates more closely to the objective than the $q_c - D_r$ criteria.

In plastic clays C_u by field vane is higher than C_u by DMT

C_u by field vane needs to be corrected before it can be used in design. In plastic clays the correction – a reduction factor to be applied to C_u by field vane – is substantial. For instance for a clay with a plasticity index PI = 70 the Bjerrum correction is 0.70.

The 1980 DMT correlations for C_u were developed by calibrating vs operative Bjerrum--corrected C_u values, i.e. the Bjerrum reduction is already included in C_u by DMT, which is then to be used without correction.

Penetration resistance of the cone and of the blade in sand

Campanella and Robertson (1991), based on parallel DMT and CPT data at McDonald's farm, found in the clean sand layers (Id>2) the following approximate relationship between the DMT and CPT penetration resistances:

$$q_d = 1.1\, q_c \tag{8.1}$$

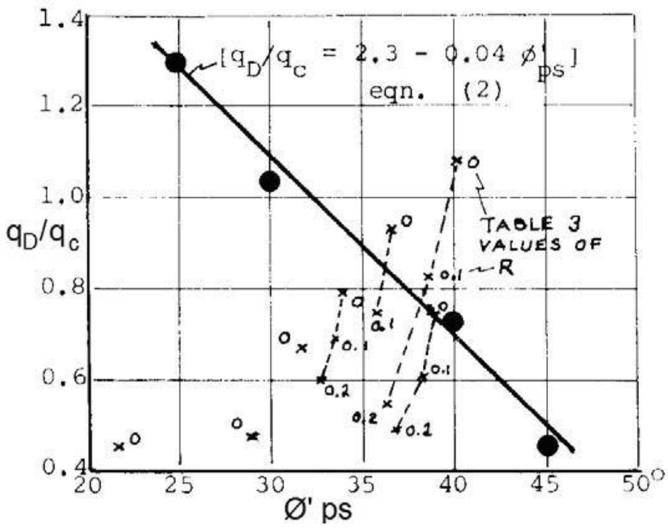

Fig. 8.1 Predicted q_d/q_c from D&M theory (Schmertmann 1982)

Campanella and Robertson attributed the 10% difference to the frictional stress on the sides of the blade, because the cone only measures bearing capacity, whereas the DMT bearing resistance includes friction. Considering that the total exposed end area of a dilatometer blade is about 18 cm², the total force required for penetration of the dilatometer is approximately twice that required for a 10 cm² cone.

Schmertmann (1982), using the D&M theory, plotted for sand the ratio q_d/q_c vs ϕ_{ps}, the plane strain friction angle (Fig. 8.1). This diagram confirms the proximity of q_c to q_d, and suggests $q_d < q_c$ in high ϕ_{ps} sand (dense to very dense).

Additional specific information on the q_d/q_c range would be desirable. In the meanwhile the above information, for sand, suggests that q_c and q_d are not very different, while the total penetration force for the dilatometer may be 50% to 100% higher than for cone, due to the larger end area.

Penetration speed and inflation speed

Sometimes DMT specifications include the prescription of a penetration speed of 2 cm/sec, as with CPT. This rather strict limitation is unnecessary, because for DMT, unlike CPT, the penetration is just for inserting the instrument, while the real test occurs later. Penetration speeds in the range half or twice the 2 cm/sec are adequate and do not affect the results.

As to the inflation speed, the recommendation of taking the *A*-reading in orientatively 15 sec is, in general, adequate. However, when the specific purpose is to estimate C_u in silts, where the rate of consolidation is relatively fast, it is advisable to start the inflation immediately after reaching the test depth, taking the *A*-reading possibly in 5-10 sec, in order to possibly insure undrained conditions during the test. In particular in silts the inflation should never be slow, otherwise B, due to some consolidation during the expansion, will be too low, and consequently I_D, ED, *M* too low as well (more details in TC16 - 2001, section 11.4.4). A way of finding out if appreciable consolidation takes place during the time for determining *A* and *B* (say 0.5 min) is to take repeated *A*-readings, for say 1 min (every time deflating immediately after *A*). If the A-decay is appreciable, then the test occurs in partially drained conditions, and *B* is probably too low.

Absence of the *A*-reading in very soft soils

Sometimes, when testing in very soft soils, especially at shallow depths, the lateral stress applied by the soil to the blade is insufficient to close the membrane. In this case the syringe may be used to apply a suction to the system in order to close the membrane. In this case the *A*-reading is a negative pressure. This value, with the sign minus, is introduced in the usual formulae for p_o and p_1, which will then provide the correct p_o and p_1.

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Use of SDMT results for earthquake resistant design: examples of application in silty-clayey sites

Paola Monaco
University of L'Aquila, Italy

Sara Amoroso
Istituto Nazionale di Geofisica e Vulcanologia, L'Aquila, Italy

Abstract. This paper comments on the use of results of seismic dilatometer tests (SDMT) for the site characterization aimed at the definition of the seismic action on constructions, a key element in earthquake resistant design. Examples of application are presented with particular reference to silty-clayey sites. The following issues are addressed: (1) Use of the „surrogate" parameter c_u (undrained shear strength), in place of the „primary" parameter V_S (shear wave velocity), for the identification of ground types required to determine the seismic action according to the Eurocode 8. (2) Experimental interrelationship between small strain and working strain stiffness using SDMT. (3) Evaluation of V_S from mechanical D_{MT} data. (3) Use of SDMT results for deriving in situ G-γ decay curves.

Keywords: seismic dilatometer test, shear wave velocity, ground type identification, seismic action, in situ stiffness decay curves

Introduction

In the recent years the crucial role of the local ground conditions on the seismic action on constructions has been definitely recognized in current earthquake resistant design practice, also stimulated by the introduction of the Eurocode 8 (EN 1998-1:2004).

A key parameter, at the base of any method for quantifying the influence of the local ground conditions on the seismic action, is the shear wave velocity V_S. According to the EC8 simplified approach, the seismic action can be evaluated using elastic response spectra defined taking into account the „ground types" site classification, based on the equivalent shear wave velocity in the top 30 m of soil ($V_{S,30}$). A more rational approach is based on numerical site seismic response analyses, which require accurate soil input data including, besides the profiles of V_S, also the curves describing the non linear and dissipative behaviour of the soil under seismic loading (typically obtained by cyclic/dynamic laboratory tests).

This paper is intended to provide a contribution on this topic, based on the experience accumulated in the recent years using the seismic dilatometer (SDMT) for site characterization in seismic areas. In particular, the results presented in this paper were obtained by SDMTs executed at various silty-clayey sites in Abruzzo, a highly seismic region in central Italy, where many strong earthquakes occurred in the past. Only in the last century: on January 13, 1915 a magnitude $M_W = 7$ earthquake devastated the Fucino plain, causing over 30,000 victims; on April 6, 2009 a $M_W = 6.3$ earthquake destroyed the city of L'Aquila (regional capital), causing 309 victims, about 1,600 injured, 40,000 homeless and huge economic losses.

Seismic dilatometer test (SDMT)

The seismic dilatometer (SDMT) is the combination of the mechanical flat dilatometer (DMT), introduced by Marchetti (1980), with an add-on seismic module for measuring the shear wave velocity V_S (conceptually similar to the seismic cone penetration test). First introduced by Hepton (1988), the SDMT was subsequently improved at Georgia Tech, Atlanta, USA (Martin and Mayne 1997, 1998; Mayne et al. 1999).

A new SDMT system (Figure 1) has been recently developed in Italy (Marchetti et al. 2008). The seismic module (Figure 1a) is a cylindrical element placed above the DMT blade, equipped with two receivers spaced 0.50 m. The shear wave source, located at the ground surface, is a pendulum hammer (≈ 10 kg) which hits horizontally a steel rectangular plate pressed vertically against the soil (by the weight of the truck) and oriented with its long axis parallel to the axis of the receivers, so that they can offer the highest sensitivity to the generated shear wave. The shear wave generated at the surface reaches first the upper receiver, then, after a certain delay, the lower receiver (Figure 1b). The seismograms acquired by the two receivers, amplified and digitized at depth, are visualized on a PC in real time, and the time delay between the signals is determined immediately. V_S is obtained as the ratio between the difference in distance between the source and the two receivers ($S_2 - S_1$) and the time delay between the arrivals of the impulse at the two receivers (Δt). V_S measurements are typically taken every 0.50 m of depth (while the mechanical DMT readings are taken every 0.20 m). The *true-interval* test configuration, with the two receivers, avoids possible inaccuracy in the determination of the „zero time" at the hammer impact, which is necessary if the one-receiver configuration is utilized. Moreover, the couple of seismograms recorded by the two receivers at a given test depth corresponds to the same hammer blow (i.e. same generated waves) and not to different blows in sequence, which are not necessarily identical. Hence the repeatability of V_S measurements is considerably improved (observed V_S repeatability ≈ 1 %, i.e. a few m/s). The determination of the time delay from SDMT seismograms, normally obtained using a cross-correlation algorithm, is generally well conditioned, being based on the waveform analysis of the two seismograms rather than relying on the first arrival time or specific single points in the seismogram. An example of seismograms obtained by SDMT – as recorded and re-phased according to the calculated delay – is shown in Figure 2.

V_S measurements by SDMT have been validated by several comparisons with V_S measured by other in situ techniques at various research sites, as reported by Marchetti et al. (2008). Besides V_S, the seismic dilatometer provides the parameters obtained by the classical DMT interpretation (Marchetti 1980, TC16 Report – Marchetti et al. 2001), e.g. the undrained shear strength c_u (in clay) and the constrained modulus M_{DMT}.

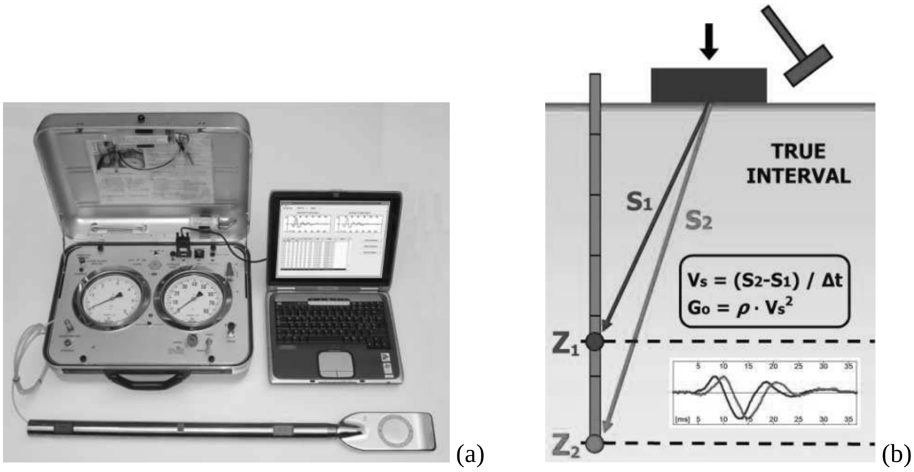

Fig. 1. Seismic dilatometer test. (a) DMT blade and seismic module. (b) Schematic test layout.

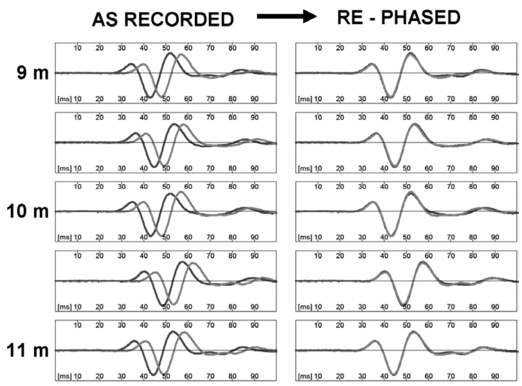

Fig. 2. Example of seismograms obtained by SDMT

Identification of ground types for the definition of the seismic action using v_S vs. c_u

Ground type classification according to the Eurocode 8

According to the Eurocode 8 – Part 1 (EN 1998-1:2004), the local ground conditions and their influence of the seismic action may be taken into account by identification of ground types (A, B, C, D, E) described by the stratigraphic profiles and the parameters given in § 3 Ground conditions and seismic action – Table 3.1. The site should be classified according to the value of an average (equivalent) shear wave velocity in the top 30 m $V_{S,30}$, if this is available. Otherwise the value of the SPT blow count N_{SPT} (in coarse-grained soils) or the undrained shear strength c_u (in fine-grained soils) should be used. The same criteria are adopted by the recent EC8-inspired Italian Technical Code for Constructions (NTC 2008), which explicitly reports formulations for evaluating the „equivalent" $N_{SPT,30}$ and $c_{u,30}$ (formulations similar to $V_{S,30}$), also for the case of alternating layers of coarse- and fine-grained soils.
In the Eurocode 8 – Part 5 (EN 1998-5: 2004), § 4.2.2 Determination of the ground type for the definition of the seismic action, it is prescribed that the profile of the shear wave velocity V_S in the ground shall be regarded as the most reliable predictor of the site-dependent characteristics of the seismic action at stable sites. It is also specified that in situ measurements of the V_S profile by in-hole geophysical methods should be used for important structures in high seismicity regions, especially in the presence of ground conditions of type D, S_1, or S_2. However for all other cases, when the natural vibration periods of the soil need to be determined, the V_S profile may be estimated by empirical correlations using the in situ penetration resistance or other geotechnical properties, allowing for the scatter of such correlations.

The above statement offers the way to some criticism. Though in the EC8 the shear wave velocity V_S is clearly recognized as the key parameter for quantifying the influence of the local ground conditions on the seismic action, on the other hand, in many practical cases, the designer is allowed to calculate the seismic action based on „secondary" parameters such as N_{SPT} or c_u as a subjective option.

Since several reliable and cost-effective routine in situ techniques for the direct measurement of V_S are available today, the possibility of seismic site classification based on N_{SPT} or c_u, rather than directly on V_S , appears somewhat outdated. Moreover experience has shown that in some cases the identification of ground types based N_{SPT} or c_u. V_S, as defined in the EC8, may lead to contradictory or ambiguous evaluations.

The issue of the identification of ground types based on the „surrogate" parameter c_u in fine-grained soils, in place of the „primary" parameter V_S, is discussed in this paper based on direct comparisons of $V_S - c_u$ profiles at various sites investigated by SDMT (§ 3.2), indirectly reinforced by recent research on the experimental interrelationship between small strain and working strain stiffness using SDMT (§ 4). Similar considerations based on direct comparisons $V_S - N_{SPT}$ in coarse-grained soils were illustrated by Monaco (2011).

Identification of ground types based on V_S vs. c_u from SDMT in silts and clays

This section presents a selection of results obtained by seismic dilatometer tests executed at various silty-clayey sites in Abruzzo, Italy. The typical graphical SDMT output (Figures 3 to 6) displays the profile of V_S as well as the profiles of four basic DMT parameters: the material index I_D (indicating soil type), the constrained modulus M, the undrained shear strength c_u (in clay) and the horizontal stress index K_D (related to OCR), calculated with usual DMT interpretation formulae (Marchetti 1980, Marchetti et al. 2001). The available experience (Marchetti et al. 2008) indicates that V_S measured by SDMT is generally accurate and highly reproducible, and that c_u obtained from DMT using the original Marchetti (1980) correlation is usually dependable for design (Marchetti et al. 2001). At most of the investigated sites, c_u values determined from laboratory tests on undisturbed samples were not available. To note, however, that the availability of continuous profiles of c_u obtained from DMT (or e.g. from CPT) proves generally advantageous for the identification of ground types, compared to the typically „discontinuous" laboratory c_u profiles. Therefore at each investigated site the pair of profiles of V_S (measured) and c_u (interpreted), used for the identification of ground types according to the EC8 (EN 1998-1:2004) – Table 3.1, were both obtained from SDMT. The results are summarized in Table 1.

Figure 3 shows SDMT results obtained at the site of Fucino – Telespazio. This site, a normally consolidated (NC), quite homogeneous lacustrine soft clay deposit, is well known to researchers, because at the end of the ‚80s it was selected as a national geotechnical research site and extensively investigated by several in situ and laboratory techniques (Burghignoli et al. 1991). At the site of Fucino – Telespazio the calculated value of $V_{S,30}$ = 106 m/s indicates „ground type D" ($V_{S,30}$ < 180 m/s, c_u < 70 kPa). according to the EC8 (EN 1998-1:2004) – Table 3.1. The same ground type identification is obtained using the equivalent c_u calculated over 30 m depth, in the form (similar to $V_{S,30}$) specified in the Italian building code (NTC 2008), which provides $c_{u,30}$ = 29 kPa, or even simply using an average value $c_{u,ave}$ = 37 kPa, accounting for the generic designation of „c_u" provided by the EC8 (EN 1998-1:2004) – Table 3.1. (Incidentally, at the site of Fucino – Telespazio c_u estimated from DMT was found in good agreement with c_u determined from laboratory or other in situ tests, as shown by Marchetti et al. 2001).

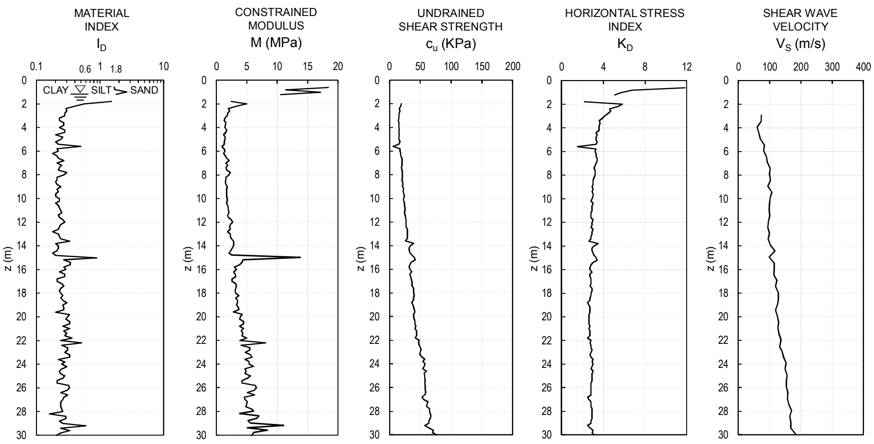

Fig. 3. SDMT results at the site of Fucino – Telespazio

Figures 4 to 6 show SDMT results obtained at various sites investigated by SDMT in the area surrounding the city of L'Aquila, in fine- to medium-grained soils. Some of these tests were carried out in the aftermath of the April 6, 2009 earthquake, as part of site investigations planned at a number of sites selected for the location of new temporary houses (C.A.S.E. Project) or for the seismic microzonation study promoted by the Italian Department of Civil Protection (MS-AQ Working Group 2010). A comprehensive review of SDMT results in the area of L'Aquila can be found in Monaco et al. (2013). The soils at the sites investigated by SDMT range from clay to silty sand, silt in the majority of the cases, and belong to the overconsolidated (OC) Pleistocene lacustrine deposits which fill the bottom of the L'Aquila basin (see Monaco et al. 2012). The SDMT results shown in Figures 4 and 5 were obtained at two C.A.S.E. sites (Cese di Preturo, Roio Piano), investigated also by other in situ techniques. At these sites the V_S profiles obtained by SDMT (diagrams on the right) were generally found in satisfactory agreement with the V_S profiles obtained by Down-Hole tests (DH) and multi-receiver surface waves tests (MASW) (Monaco et al. 2013, Santucci de Magistris et al. 2013). Figure 6 shows the SDMT results obtained at the site of Ponte Rasarolo, near the Aterno riverside. At this site liquefaction and lateral spreading phenomena, triggered by the April 6, 2009 earthquake, were observed (Aydan et al. 2009) in a shallow, 4-5 m thick loose sand layer, overlying very stiff clay. (Another case of liquefaction observed at Vittorito, about 45 km far from the epicentre, analyzed by use of SDMT results, was presented by Monaco et al. 2011).

As commonly found in the OC silty-clayey soils around L'Aquila, at all the above sites (Figures 4 to 6) the maximum investigated depth was limited to ≈ 17 to 23 m by the push capacity of the penetrometer rig. Therefore it was not possible to calculate the values of $V_{S,30}$ in the top 30 m according strictly to the EC8 formulation. However, since the purpose of this study was to compare ground type identifications provided by V_S and c_u in the same deposit/layer, an equivalent shear wave velocity V_S,test depth was then calculated over the investigated depth (< 30 m), by adapting the EC8 formulation for $V_{S,30}$ to the maximum test depth, instead of the conventional 30 m. A corresponding equivalent undrained shear strength c_u,test depth was calculated in a similar way.

At Cese di Preturo (Figure 4) and Roio Piano (Figure 5) the calculated values of V_S,test depth are in the range 233 to 257 m/s. According to the EC8 (EN 1998-1:2004) – Table 3.1 these values indicate „ground type C" ($V_{S,30}$ = 180-360 m/s, c_u = 70-250 kPa). The same ground type identification is obtained using the equivalent c_u calculated over the same test depth, cu,test depth = 123 to 168 kPa, or even using simply an average value of $c_{u,ave}$ = 148 to 241 kPa. However at the site of Ponte Rasarolo – Aterno River (Figure 6), in the clay layer between ≈ 5 and 17 m depth, the equivalent V_S,test depth = 259 m/s indicates „ground type C", while the equivalent cu,test depth = 374 kPa, or the average $c_{u,ave}$ ≈ 474 kPa, indicates „ground type B" ($V_{S,30}$ = 360-800 m/s, c_u > 250 kPa).

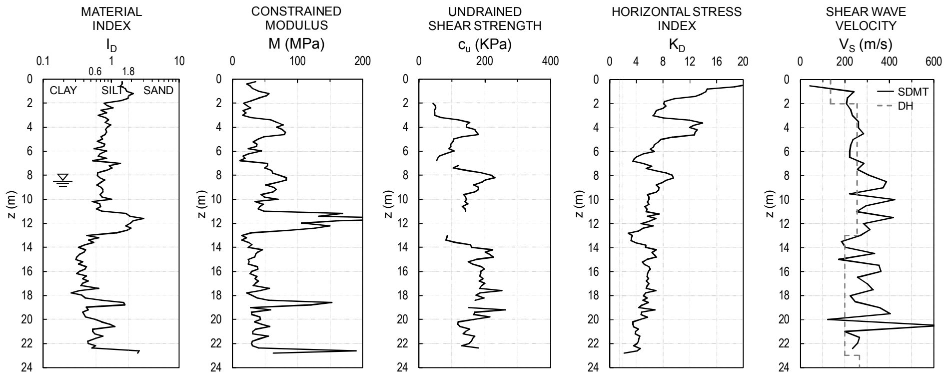

Fig. 4. SDMT results at the site of Cese di Preturo (C.A.S.E. Project), L'Aquila. *On the right*: Comparison of profiles of V_S from SDMT and Down-Hole (Polo Geologico, MS–AQ Working Group 2010).

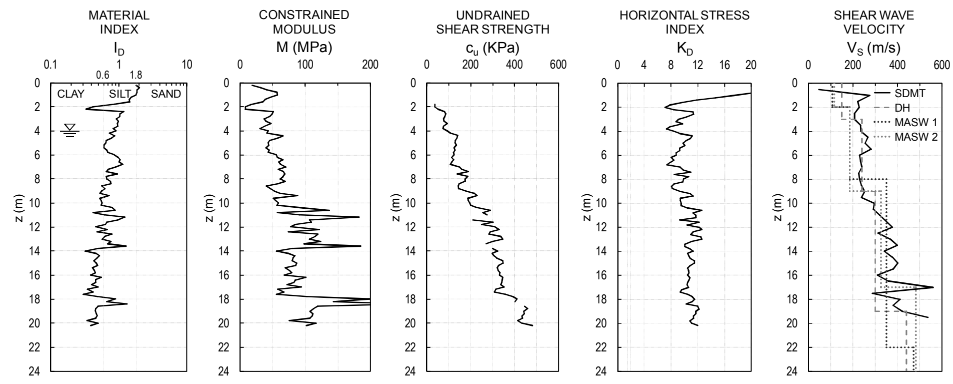

Fig. 5. SDMT results at the site of Roio Piano (C.A.S.E. Project), L'Aquila. *On the right:* Comparison of profiles of V_S from SDMT, Down-Hole (Polo Geologico) and MASW (Politecnico di Torino). DH and MASW data from MS–AQ Working Group (2010); see also Santucci de Magistris et al. (2013).

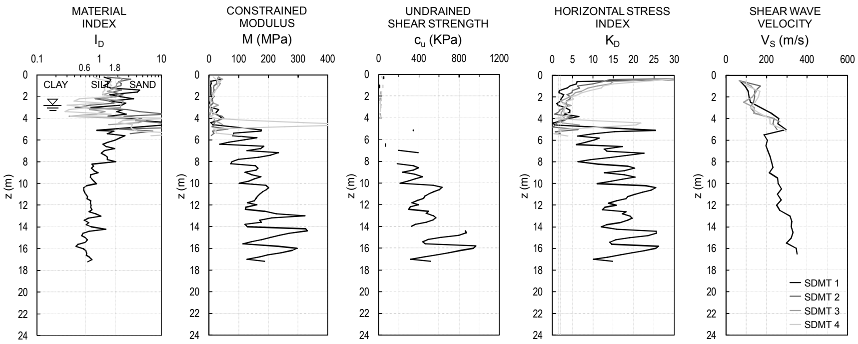

Fig. 6. SDMT results at the site of Ponte Rasarolo – Aterno River, L'Aquila

To note that the silty-clayey soils at the above three sites basically belong to the same geological formation (Pleistocene OC lacustrine deposits). However, the overconsolidation ratio OCR of these deposits is known to be rather variable across the L'Aquila basin, due to a very complex depositional/tectonic history. Compared to the other two examined sites, the clay deposit at Ponte Rasarolo exhibits much higher values of OCR (and c_u), as inferred from the DMT horizontal stress index K_D, but similar V_S. This suggests that the increase in c_u due to overconsolidation is much higher than the increase in V_S, which appears substantially unaffected by the OCR. Since the OCR seems to have a different influence on V_S and c_u, i.e. the two alternative parameters used for ground type identification according to the EC8, in highly OC clays the identification of ground type based on c_u rather than on V_S may lead to a contradictory evaluation, possibly resulting in an underestimate of the seismic action.

Tab.1. Identification of ground types according to the EC8 (EN 1998-1:2004) – Table 3.1 based on V_S vs. c_u in various silty-clayey sites in Abruzzo, Italy

Site	Soil description	OCR	$V_{S,30}$ or $V_{S,test\ depth}$ (m/s)	$c_{u,30}$ or $c_{u,test\ depth}$ (kPa)	$c_{u,ave}$ (kPa)	Ground type based on $V_{S,30}$	Ground type based on $c_{u,30}$
Fucino – Telespazio	clay	1.5-2	106	29	37	D	D
Cese di Preturo (L'Aquila)	silt – silty clay	4-6	233	123	148	C	C
Roio Piano (L'Aquila)	clayey silt – silty clay	8-12	257	168	241	C	C
Ponte Rasarolo (L'Aquila)	clayey silt – silty clay	20-30	259	374	474	C	B

Considerations on the experimental interrelationship between G_0 and M_{DMT}

The evidence emerging from the above direct comparisons is indirectly reinforced by recent research on the experimental interrelationship between *small strain* and *working strain* stiffness using SDMT results. Such interrelationship is investigated by considering that the SDMT provides routinely, at each test depth, both the small strain shear modulus G_0 (obtained as $G_0 = \rho V_S^2$) and the *working strain* constrained modulus M_{DMT}. The latter is obtained from the usual DMT interpretation. The effectiveness of the M_{DMT} estimation has been proved by the good agreement observed in a large number of well documented comparisons between measured and DMT-predicted settlements or moduli (Monaco et al. 2006).

The experimental relationship between G_0 and M_{DMT} is illustrated in the diagram in Figure 7, where the ratio G_0/M_{DMT} is plotted as a function of the DMT horizontal stress index K_D (related to OCR) for clay (having material index $I_D < 0.6$), silt ($0.6 < I_D < 1.8$) and sand ($I_D > 1.8$). Best fit equations are indicated for each soil type. This diagram was constructed using same-depth values of G_0 and M_{DMT} derived from SDMT results at 34 different sites, in a variety of soil types (Marchetti et al. 2008, Monaco et al. 2009).

Recognizable trends in Figure 7 are:

- The data points tend to group according to their I_D (soil type).
- The ratio G_0/M_{DMT} varies in a wide range (≈ 0.5 to 20 for all soils), hence it is far from being a constant, especially in clays and silts. Its value is strongly dependent on multiple information, e.g. soil type and stress history. (As a consequence, it appears next to impossible to estimate the „operative" modulus M by simply dividing G_0 by a constant, as suggested by various Authors).
- For all soils G_0/M_{DMT} decreases as K_D (OCR) increases.

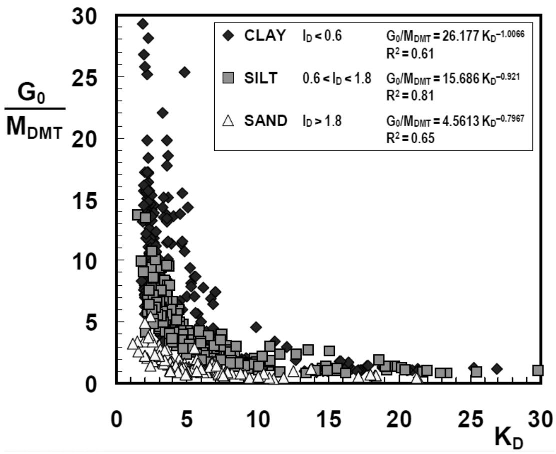

Fig. 7. Ratio G_0/M_{DMT} vs. K_D (OCR) for various soil types (Monaco et al. 2009)

The experimental diagram G_0/M_{DMT} vs. K_D in Figure 7 offers some elements of discussion on the feasibility of using c_u (or N_{SPT}) as a substitute for V_S – when V_S has not been measured – for ground type identification to define the seismic action, as allowed by the EC8. In fact, Figure 7 highlights the dominant influence of K_D (OCR) on the ratio G_0/M. In case of non availability of K_D, all the experimental data points would cluster on the vertical axis. In absence of K_D – which reflects the stress history – the selection of the ratio G_0/M would be hopelessly uncertain. Hence as many as three informations, i.e. I_D, K_D, M (though only two independent), are needed to formulate rough estimates of G_0 and V_S.

In view of the above consideration, the use of c_u (or N_{SPT}) alone, as a substitute of V_S (when not measured) for the seismic classification of a site, does not appear founded on a firm basis. In fact, if V_S is assumed to be the primary parameter for the classification of the site, then the possible substitute of V_S must be reasonably correlated to V_S. If three parameters (I_D, K_D, M) are barely sufficient to obtain rough estimates of V_S, then the possibility to estimate V_S from only one parameter appears remote.

Evaluation of V_S from mechanical DMT DATA

As a general rule it is by large preferable to measure V_S directly, as firmly recommended by the EC8. However Figure 7 might turn out helpful to obtain rough estimates of V_S (via G_0) at sites where V_S has not been measured and only mechanical DMT results from past investigations are available.

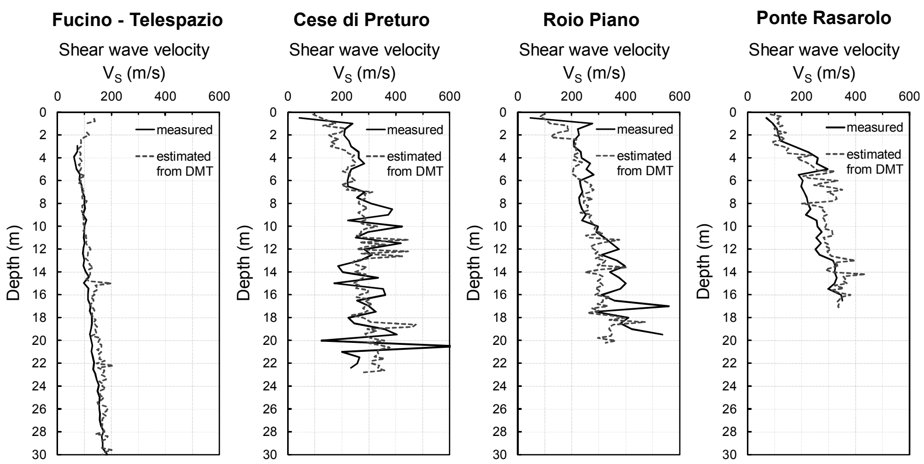

Fig. 8. Comparison of profiles of V_S measured by SDMT and estimated from DMT data at four silty-clayey sites in Abruzzo, Italy

This was just the case in the area of L'Aquila, where, in addition to the post-earthquake SDMT data, the results of several mechanical DMTs carried out in the past were collected for the seismic microzonation study. These results were then used to obtain rough estimates of V_S, at sites where it had not been measured, by use of the correlations in Figure 7 (Monaco et al. 2012, Santucci de Magistris et al. 2013).

The comparisons shown in Figure 8 indicate a good agreement between the profiles of V_S *measured* directly by SDMT (solid line) and V_S *estimated* from mechanical DMT data (dashed line) obtained in the same SDMT sounding, using the correlations in Figure 7, at the four silty--clayey sites previously examined. The relative error, calculated as $(V_{S\ measured} - V_{S\ estimated}) / V_{S\ measured}$, is on average about 20 % or less.

Tentative method for deriving in situ G-γ decay curves from SDMT

Marchetti et al. (2008) proposed the possible use of the SDMT for deriving „in situ" curves depicting elemental soil stiffness variations with strain level (G-γ curves or similar). Such curves could be tentatively constructed by fitting „reference typical-shape" laboratory G-γ curves through two points, both obtained by SDMT: (1) the initial *small strain* shear modulus G_0, obtained as $G_0 = \rho V_S^2$; (2) a *working strain* shear modulus G_{DMT} derived from the constrained modulus M_{DMT} provided by the usual DMT interpretation, as a first approximation by linear elasticity formulae.

To locate the second point on the G-γ curve it is necessary to know, at least approximately, the elemental shear strain corresponding to G_{DMT}(hereafter denoted as γ_{DMT}) along the G-γ curve. Preliminary qualitative indications have located the DMT moduli at an intermediate level of strain, i.e. $\gamma_{DMT} \approx 0.05$-$0.1$ % (Mayne 2001) or ≈ 0.01-1 % (Ishihara 2001). Quantitative indications were recently derived by Amoroso et al. (2012, 2014) by comparing SDMT data with „reference" stiffness decay curves (back-figured from the observed field behaviour under full-scale loading, or obtained by cyclic/dynamic laboratory tests, or reconstructed by the combined use of different in situ/laboratory testing techniques) in different soil types, according to the following procedure:

1) Using SDMT data obtained at the same depth of each available reference stiffness decay curve, a *working strain* modulus G_{DMT} (or E_{DMT}) was derived from M_{DMT} and normalized by its *small strain* value G_0 (or E_0) derived from V_S.
2) The G_{DMT}/G_0 (or E_{DMT}/E_0) horizontal ordinate line was superimposed to the same-depth experimental stiffness decay curve, in such a way that the data point ordinate was set to match the curve.
3) The „intersection" of the G_{DMT}/G_0 (or E_{DMT}/E_0) horizontal ordinate line with the stiffness decay curve provided a shear strain value, referred to as γ_{DMT}.

The application of the above „intersection" procedure at various sites permitted to obtain several G_{DMT}/G_0 - γ_{DMT} data points in different soil types. Such data points were superimposed on typical literature G/G_0 -g decay curves (e.g. Darendeli 2001), as shown in Figure 9. The shaded areas in Figure 9 represent the range of values of the *normalized working strain shear modulus* G_{DMT}/G_0 and the corresponding shear strain γ_{DMT} determined by the „intersection" procedure in different soil types. Based on the available information, „typical ranges" of the shear strain associated to the *working strain* moduli G_{DMT} were approximately assumed as: $\gamma_{DMT} \approx$ 0.01-0.45 % in sand, $\gamma_{DMT} \approx$ 0.1-1.9 % in silt and clay.

Amoroso et al. (2014) proposed a hyperbolic stress-strain formulation (Eq. 1) for estimating the G/G_0 -γ decay curve from SDMT data:

$$\frac{G}{G_0} = \frac{1}{1 + \left(\frac{G_0}{G_{DMT}} - 1\right)\frac{\gamma}{\gamma_{DMT}}} \tag{1}$$

At a given site, by introducing into Eq. (1) the ratio G_{DMT}/G_0 obtained from SDMT results and a „typical" shear strain γDMT estimated from Figure 9 for the given soil type, it is possible to plot the corresponding G/G_0 -γ hyperbolic curve. As an example, the comparison in Figure 10 shows that, at the site of Roio Piano (L'Aquila), the G/G_0 -γ curve derived from SDMT using Eq. (1) fits quite well the „measured" stiffness decay curve.

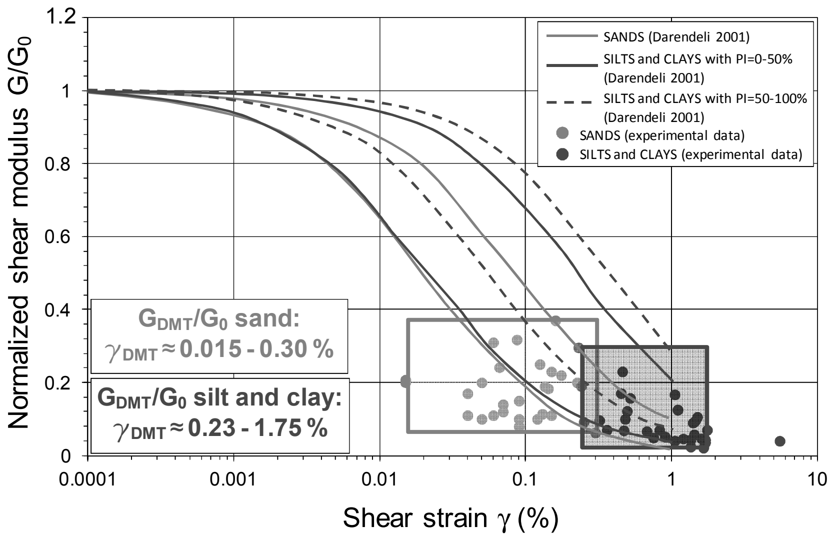

Fig. 9. Use of SDMT data for calibrating the selection of in situ G/G_0 -γ decay curves in various soil types (Amoroso et al. 2014)

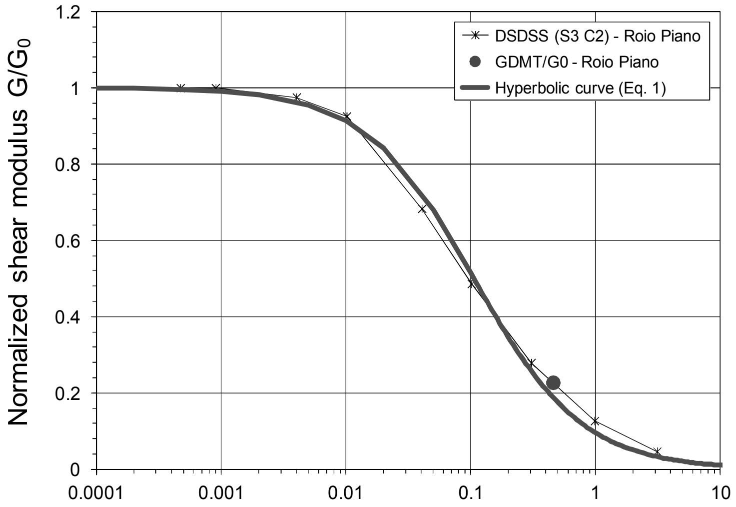

Fig. 10. Roio Piano, L'Aquila – Comparison between G/G_0 -γ decay curves estimated from SDMT by Eq. (1) and „measured" in the laboratory (DSDSS: double sample direct simple shear tests, University of Roma La Sapienza, Santucci de Magistris et al. 2013) (Amoroso et al. 2014)

Conclusions

The examples illustrated in the paper, based on the use of SDMT results obtained at various silty-clayey sites in the highly seismic Abruzzo region (central Italy), permit to express the following comments.

- The use of the „surrogate" parameter c_u, in place of the „primary" parameter V_S, for the identification of ground types required to define the seismic action on constructions according to the Eurocode 8, in some cases may prove to be inconsistent. In general, in NC to moderately OC clays the use of c_u provides the same broad identification of ground type as V_S. However in highly OC clays the identification of ground type based on c_u may possibly lead to a less conservative evaluation compared to V_S, resulting in an underestimate of the seismic action.
- The evidence emerging from such direct V_S - c_u comparisons is indirectly reinforced by recent research on the experimental interrelationship between *small strain* and *working strain* stiffness using SDMT, which is largely influenced by the stress history, especially in clays.
- Considering that several reliable and cost-effective routine in situ techniques are available today for the direct measurement of V_S, the possibility of identifying the ground type to determine the seismic action based on c_u (or N_{SPT}), rather than directly on V_S, appears outdated and possibly should be abandoned, or at least explicitly restricted to design of minor constructions (e.g. buildings of importance class I of the EC8) or low-risk projects.

- At sites where V_S has not been measured and only mechanical DMT results from past investigations are available, rough estimates of V_S (via G_0) can be obtained from mechanical DMT data.
- SDMT results could be used to assess the decay of in situ stiffness with strain level and to provide guidance in selecting elemental G-γ curves in various soil types (a basic input information in a site seismic response analysis). This potential stems from the ability of the SDMT to provide routinely, at each test depth, both a *small strain* stiffness (G_0 from V_S) and a *working strain* stiffness G_{DMT} (derived via standard DMT correlations). The use of the hyperbolic relationship (Eq. 1) proposed by Amoroso et al. (2014) for estimating G/G_0 -γ curves from SDMT, requiring the input of the ratio GDMT /G_0 together with an estimate of a „typical" shear strain γDMT for each soil type (Figure 9), can provide a useful first order estimate of a soil's G-γ degradation curve.

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The seismic dilatometer for in situ soil investigations

Diego Marchetti
Senior Eng., Studio Prof. Marchetti Srl – Italy

Abstract. In the last decades we have assisted at a massive migration from laboratory testing to in situ testing, to the point that, today, in situ testing is often the major part of a geotechnical investigation. The State of the Art at the last Geotechnical World Conference in 2009 indicates that direct-push in situ tests, such as the Cone Penetration Test (CPT) and the Flat Dilatometer Test (DMT), are fast and convenient in situ tests for routine site investigation. Scope of this paper is to describe the DMT, the obtained information and engineering applications. The paper also describes the recently developed Seismic Dilatometer (SDMT), which is a DMT with an add-on seismic module for measuring also the shear wave velocity V_S. DMT and SDMT have been found helpful in projects where soil stiffness and settlements predictions are critical to the design.

Keywords: dilatometer test, seismic dilatometer test, geotechnical applications, settlements, liquefaction.

Introduction

The Flat Dilatometer (DMT) is an in situ testing tool developed about 35 years ago (Marchetti 1980). The DMT is currently used in practically all industrialized countries. It is standardized in the ASTM (2001 & 2007) and the Eurocode 7 (1997 & 2007). The DMT has been object of a detailed monograph by the ISSMGE Technical Committee TC16 (2001).

Some key features of the DMT are:

- The DMT is a penetration test. As such, it has the advantage of not requiring a borehole.
- The DMT, being a load-displacement test, provides information on soil stiffness, an information unobtainable by penetration tests, that essentially measure "rupture" characteristics, i.e. strength. Moreover the insertion distortions caused by the DMT blade are substantially less than the distortions caused by conical probes.

- The DMT equipment is robust, easy to use and remarkably operator-independent and repeatable.
- The DMT provides information on Stress History, whose knowledge is of primary interest, because Stress History has a dominant influence on soil behavior. In particular stress history is necessary for estimating operative moduli and liquefaction resistance.

  As to the SDMT, the add-on module has added to the parameters measurable by DMT the shear wave velocity V_S. V_S is today increasingly measured because of:
- More frequent requirement of seismic analyses, for which V_S is a basic input parameter.
- The newly introduced Eurocode 8 seismic regulations prescribe the determination of V_S in the top 30 m at all construction sites located in the seismic zones.
- SDMT provides both the Go stiffness at small strains (the shear modulus G_0 = ρ V_S2) and the stiffness at operative strains (as represented by the constrained modulus M_{DMT}). Such two stiffnesses may offer guidance when selecting the G-γ curves, i.e. the decay of the shear modulus G with the shear strain γ.

Dilatometer test (DMT)

The flat dilatometer consists of a steel blade having a thin, expandable, circular steel membrane mounted on one face. When at rest, the membrane is flush with the surrounding flat surface of the blade. The blade is connected, by an electro-pneumatic tube running through the insertion rods, to a control unit on the surface (Fig. 1).

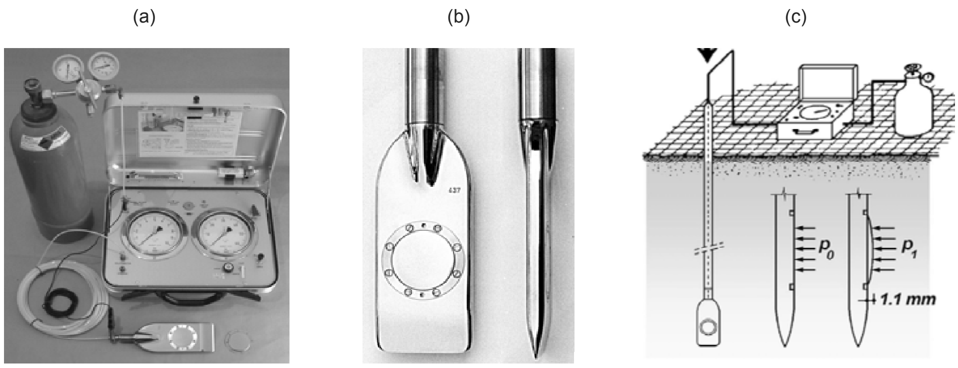


Fig. 1 Flat Dilatometer: (a) Equipment (b) Dilatometer Blade (c) Schematic layout of the seismic dilatometer test.

The control unit is equipped with pressure gauges, an audio-visual signal, a valve for regulating gas pressure (provided by a tank) and vent valves. The blade is advanced into the ground using common field equipment, i.e. penetrometers normally used for the cone penetration test (CPT) or drill rigs. The DMT can also be driven, e.g. using the SPT hammer and rods, but statical push is by far preferable. Pushing the blade with a 20 ton penetrometer truck is most effective (up to 80 m of profile per day). The test starts by inserting the dilatometer into the ground. When the blade has advanced to the desired test depth, the penetration is stopped. The operator inflates the membrane and takes, in about 30 sec, two readings: the *A* pressure, required to just begin to move the membrane ('lift-off' pressure), and the *B* pressure, required to expand the membrane center of 1.1 mm against the soil. A third reading *C* ('closing pressure') can also optionally be taken by slowly deflating the membrane soon after *B* is reached. The blade is then advanced to the next test depth, with a depth increment of typically 20 cm.

The interpretation proceeds as follows. First the field readings are converted into the DMT intermediate parameters I_D, K_D, E_D (Material index, Horizontal stress index, Dilatometer modulus). Then I_D, K_D, E_D are converted, by means of commonly used correlations, to: constrained modulus M, undrained shear strength *Cu*, *Ko*(clays), *OCR* (clays), friction angle *Ø*' (sands), bulk unit weight. Consolidation and permeability coefficients may be estimated by performing dissipation tests (TC16 2001). The C-reading, in sand, approximately equals the equilibrium pore pressure. An example of the profiles obtained by DMT is shown in Fig. 3, where:

- I_D is the material index, that gives information on soil type (sand, silt, clay)
- *M* is the vertical drained constrained modulus (at geostatic stress)
- *Cu* is the undrained shear strength
- K_D is the Horizontal Stress Index. The profile of K_D is similar in shape to the profile of the overconsolidation ratio *OCR*. $K_D \approx 2$ indicates in clays $OCR = 1$, $K_D > 2$ indicates overconsolidation. The K_D profile often provides, at first glance, an understanding of the Stress History of the deposit.

More detailed information on the DMT equipment, test procedure and all the interpretation formulae may be found in the comprehensive report by the ISSMGE Technical Committee TC16 2001.

Seismic dilatometer test (SDMT)

The SDMT is the combination of the flat dilatometer with an add-on seismic module for the measurement of the shear wave velocity (Monaco et al. 2005, Marchetti 2010). The seismic module (Fig. 2a) is a cylindrical element placed above the DMT blade, equipped with two receivers located at 0.5 m distance. When a shear wave is generated at surface, it reaches first the upper receiver, then, after a delay, the lower receiver. The seismograms acquired by the two receivers, amplified and digitized at depth, are transmitted to a PC at the surface, that determines the delay. V_S is obtained (Fig. 2b) as the ratio between the difference in distance between the source and the two receivers (S2 - S1) and the delay from the first to the second receiver (Δt). The true-interval test configuration with two receivers avoids possible inaccuracy in the determination of the "zero time" at the hammer impact, sometimes observed in the pseudo-interval one-receiver configuration. Moreover, the couple of seismograms recorded by the two receivers at a given test depth corresponds to the same hammer blow. The repeatability of the V_S measurements is remarkable (observed V_S repeatability ≈ 1 %, i.e. a few m/s).

(a) (b) (c)

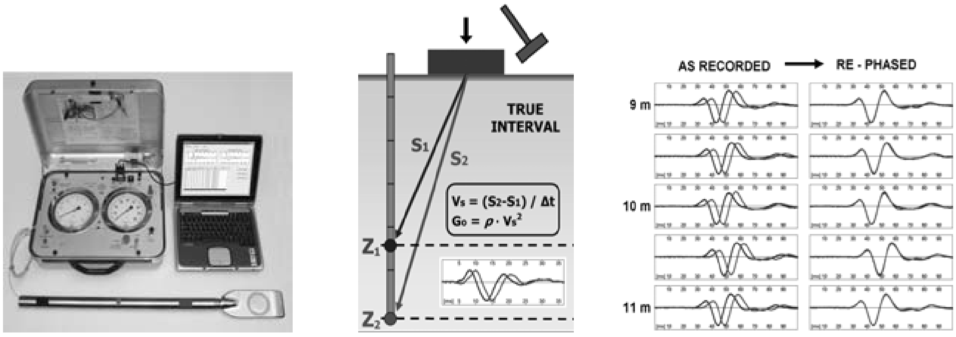


Fig. 2 Seismic Dilatometer: (a) DMT blade and seismic module (b) Schematic layout of the seismic dilatometer test. (c) Example of seismograms as recorded and rephased.

Fig. 2c shows an example of seismograms obtained by SDMT at various test depths at the site of Fucino. Fig. 3 shows an example of SDMT results. The fifth diagram is the V_S profile obtained by the seismic module.

Testable soils

The soils that can be investigated by DMT range from extremely soft soils to hard soils like soft rock. The DMT readings are accurate even in nearly liquid soils. On the other hand the blade is very robust and can penetrate even in soft rock. Clays can be tested from *Cu* = 2-4 kPa up to 1000 kPa (marls). The range of measurable moduli *M* is from 0.4 MPa up to 400 MPa.

The DMT blade can be inserted by a variety of penetration machines. Truck-mounted penetrometers are by far the fastest. A drill rig is also usable, with the Torpedo configuration (TC16 2001), though at a lower productivity. Penetration by percussion, e.g. using the SPT hammer, is also possible, although not recommended in soft soils.

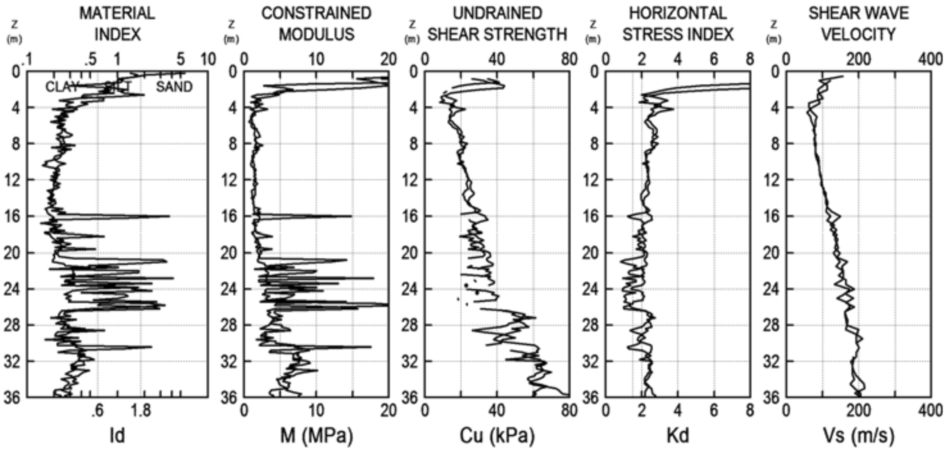


Fig. 3 Example of SDMT results (from two nearby SDMTs)

Applications to engineering problems

Design via Parameters

In most cases the DMT estimated parameters, in particular the undrained shear strength *Cu* and the constrained modulus *M*, are used with the common design methods of Geotechnical Engineering for evaluating bearing capacity, settlements etc. However, for a number of applications, additional specific comments require further discussion, which goes beyond the scope of this paper.

Settlements of Shallow Foundations.

Predicting settlements of shallow foundations is probably the No. 1 application of the DMT, especially in sands, where undisturbed samples cannot be retrieved. Settlements are generally calculated by means of the one-dimensional formula (Fig. 4a) :

$$S_{1-DMT} = \sum \frac{\Delta\sigma_v}{M_{DMT}} \Delta z \tag{1}$$

with $\Delta\sigma_v$ calculated according to Boussinesq and M_{DMT} constrained modulus estimated by DMT. The validity of the method has been confirmed by a large number of observed agreement between measured and DMT-predicted settlements. Fig. 4b compares the insertion distortions caused by probes of different shape.

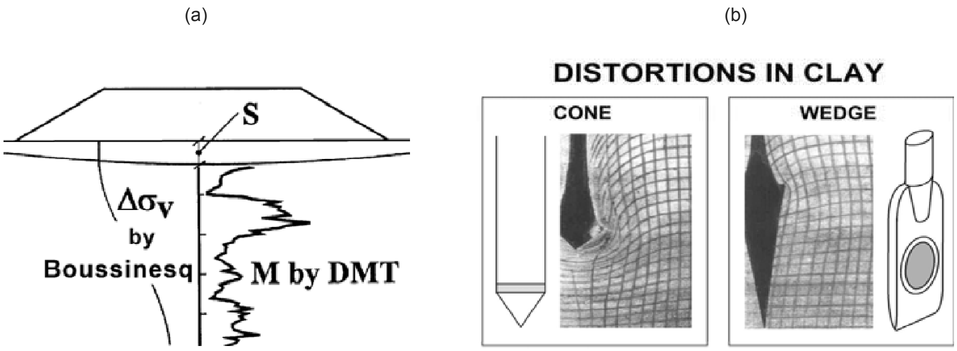


Fig. 4 (a) Settlement prediction by DMT (b) Soil distortions caused by tips of different shape (Baligh & Scott 1975).

Laterally Loaded Piles

Methods have been developed for deriving P-y curves from DMT results (Robertson et al. 1987, Marchetti et al. 1991). A number of independent validations (NGI, Georgia Tech and tests in Virginia sediments) have indicated that the two methods provide similar predictions, and that the predictions are in quite good agreement with the observed behavior. Note that all methods are for the case of first time monotonic loading.

Detecting Slip Surfaces in OC Clay

The $K_D \approx 2$ method (TC16 2001) permits to detect active or old slip surfaces in overconsolidated (OC) clay slopes, based on the inspection of the K_D profiles. In essence, the method consists in identifying zones of normally consolidated (NC) clay in a slope which, otherwise, exhibits an OC profile. The NC clay shear bands, remoulded by the sliding, then reconsolidated under the weight of the overlying soil, hence nearly NC, are recognized by using $K_D \approx 2$ as the identifier of the NC zones. Note that the method involves searching for a specific numerical value ($K_D = 2$) rather than for simply weak zones, which could be detected just as easily by other in situ tests. The $K_D = 2$ method permits to detect even quiescent surfaces, which could reactivate e.g. due to an excavation.

Monitoring Densification/ Stress Increase

Before-after DMTs have been frequently used for monitoring soil densification treatments. Compaction is generally reflected by a brisk increase of both K_D and M_{DMT}. Results by various authors indicate that the percentage increase in M_{DMT} is approximately *twice* the increase in q_t. In other words densification increases both q_t and M_{DMT}, but M_{DMT} increases at a faster rate. DMT appears therefore well suited to detect the benefits of the soil improvement.

It may be noted that, since densification is often aimed at reducing settlements, it would appear more direct to set the specifications in terms of minimum M rather than of minimum D_r - a not precisely measurable parameter.

The DMT is suitable for detecting small horizontal stress variation, e.g. in the relaxing soil behind diaphragm walls during the excavation.

Liquefiability Evaluation.
Fig. 5 and Fig. 6 show two diagrams for estimating the Cyclic Resistance Ratio (*CRR*) of a *clean* sand (FC<5%) by DMT/ SDMT, using K_D and V_S respectively. Note that DMT provides one estimate of *CRR* (based on K_D), while the seismic module of SDMT provides a second independent estimate of *CRR* (based on V_S). The curve for estimating *CRR* from V_S is by Andrus & Stokoe (2000). Details on the derivation and use of the K_D -CRR curves may be found in Monaco et al. 2005 & 2007. Recent research suggests that the most likely location of the K_D -CRR correlation is in the band comprised between the Monaco (2005) curve and the Robertson (2012) curve. The equation of the curve intermediate between the just mentioned two curves is :

$$\mathrm{CRR} = 0.0038\, K_D^{\,3} - 0.0176\, K_D^{\,2} + 0.0532\, K_D + 0.0264 \tag{2}$$

Eq. 2 is the preferred correlation, as today 2014, for estimating CRR from K_D. The use of K_D to evaluate *CRR* is increasing, due to the recognized sensitivity of K_D to a number of factors which are known to increase liquefaction resistance – such as stress history (Marchetti 2010), prestraining/aging (Monaco & Marchetti 2007), cementation, structure – but are difficult to sense by other tests, and to the K_D's relationship with the state parameter (Marchetti 2010).

Subgrade Compaction Control
DMT has been used for verifying the compaction of the natural ground surface (i.e. the subgrade) to support the road superstructure (Marchetti 1994). DMT has been used as an economical production tool for quality control of the compaction, with only occasional verifications by the originally specified methods.

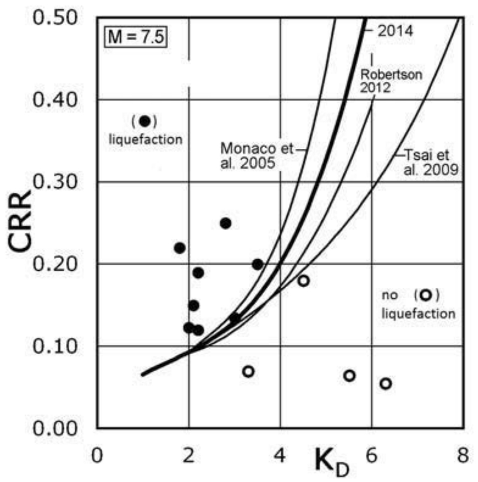
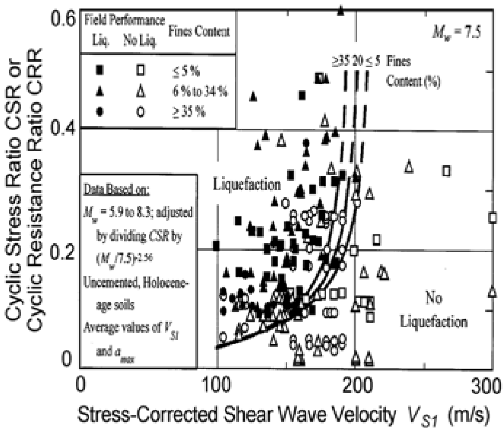


Fig. 5 Curves for evaluating CRR for *clean* sand from K_D



$$\mathrm{CRR} = \left[0.022 \left(\frac{K_{a1} V_{S1}}{100} \right)^2 + 2.8 \left(\frac{1}{V_{S1}^{*} - K_{a1} V_{s1}} - \frac{1}{V_{S1}^{*}} \right) \right] K_{a2}$$

Fig. 6 Curves for evaluating CRR from Vs (Andrus & Stokoe 2000) Conclusions

Conclusions

The Flat Dilatometer is a relatively recent in situ test. It provides estimates of a variety of design parameters. It is fast and simple to operate, and the measurements are reproducible and operator independent. The DMT most frequent application is to predict settlements. Other applications have been briefly described in the paper. The test is standardized in the ASTM and the Eurocode.

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Improved CPT sleeve friction sensitivity in soft soils

Andy Barwise, Roi Santos, Matt Alexander
Gardline Geosciences, Great Yarmouth, United Kingdom

Abstract. Current deep-water developments are based on the use of highly accurate CPTU-data for geotechnical design. The deep-water sediments consist mainly of soft to very soft clays and silts. It is known, from previous publications, there is limited sensitivity and repeatability of the sleeve friction measurement in these very soft soils. Investigations using high-resolution cone calibration techniques were recently conducted and the results show that resistance on the friction sleeve caused by O-rings and dirt seals negatively affect accuracy. On-going developments have recently led to the introduction of a new cone sealing system. A new quad seal and corrugated spring assembly incorporated in the cone provide a small preloading of the friction sleeve and improves both dirt sealing and the sensitivity of the sleeve friction measurement. CPTU-data acquired in the field confirm that cones with this feature are more sensitive in very soft soils.

Introduction

The electrical cone, as the most widely used tool for CPT, is the subject of on-going improvements. The reliability of sleeve friction resistance has been in question for several years, (Lunne et al. 1986, Bogges & Robertson, 2010). Gardline carried out internal research to identify the possible causes of the unreliability of the measurements. The review of the Gardline geotechnical database showed that poor sleeve friction response was generally associated with very soft soil conditions and tests performed in deep water. It was observed that the tests that showed poorer response on those tests where the cone had not been fully serviced before the start of the tests. It was also noted that the sleeve friction response is generally very poor in soils of less than 15 kPa in undrained shear strength. From these observations it is apparent that the cause of the reliability issues are mechanical and possibly related with the sealing components. A series of calibration tests were performed at Gardline calibration facilities in order to determine the influence of the seals during testing. This paper describes the developments to an improved sleeve friction measurement particularly for soft soils.

CPT measurement techniques

Measuring soil characteristics

When pushing a cone into the soil at a constant rate, the soil is reacting by giving a resistance on the cone surfaces. The loads acting on the cone tip and friction sleeve, are passed on to load cells where they are measured by means of strain gauges. Thus soil characteristics are converted into electrical measurement values, which can be further processed and analysed.

A load cell is a solid spring with a very high spring constant. This means that under load, the load cells will be shortened and the cone tip and friction sleeve will move relative to the fixed parts of the cone. In practice the loads on the cone tip and friction sleeve are subject to continuous changes during a CPT. So the cone design must provide sufficient movement, to allow an undisturbed measurement of cone resistance and sleeve friction. The requirement of sufficient movement has been solved by using gaps on either side of the friction sleeve. To avoid these gaps becoming clogged, causing bad measurements, elastic dirt seals are used to seal off the gaps and give the cone a smooth outer surface. Directly behind the dirt seals, inside the cone construction, O-ring seals are used to protect the cone electronics against damage from water ingress and also provide for guidance of the friction sleeve. These techniques of cone design allow for an efficient friction measurement, but also can affect the measuring process.

Influences on the measuring process

In general, the friction measurement is affected by a variety of influencing factors like method of use, ambient factors, mechanical design, and, electrical design. This paper focuses on the influences caused by the mechanical design, and in particular on those cone elements that exert a load on the friction sleeve. In Figure 1, a simplified structure of a common cone is shown. In this structure it can be seen that the friction sleeve is axially enclosed by two dirt seals and radially supported by two O-ring seals. These seals must keep out dirt and water up to 200 bar ambient pressure. It will be clear that the seals exert influence on the friction sleeve and affect the measurement results, in particular in the lower range, measuring very soft soils showing up to 15 kPa in undrained strength.

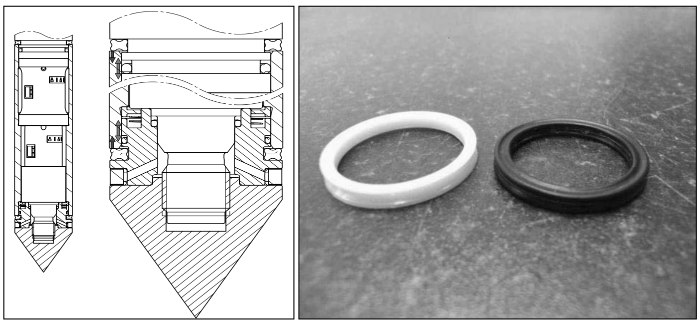

Fig. 1. Influences on the friction sleeve (left) drawing courtesy of AP Van Den Berg. Dirt seals: plastic ‘V’ shaped lip-seals an rubber quad-o-rings (right)

To ensure good sealing, the dirt seals must be sufficiently elastic to follow the 0.15 mm axial movements of the friction sleeve. At zero-load situation, both dirt seals exert an equal, but opposite load into the friction sleeve, as indicated by the blue arrows in Figure 1. During CPT, movement of the friction sleeve causes an increase of the upper seal load and a decrease of the lower seal load. This results in a one-directional, influencing effect on the sleeve, which is directly proportional to the friction sleeve load.

The O-ring seals are located in small grooves and are slightly pressed oval for best seal properties. By placement of the friction sleeve onto the cone, the O-rings are pressed to one side of the grooves, giving a reaction load into the sleeve due to elastic deformation. This influencing effect can be reduced by rotating the sleeve back and forth a few times (if the sleeve is designed to rotate). When the friction sleeve is moving due to a CPT-load, the O-ring seals exert a frictional load onto the inner surface of the sleeve, as indicated by the red arrows in Figure 1. This results in a influencing effect, which has a constant value and is opposite to the direction of movement of the sleeve. This effect may be greater when the cone is used offshore. In this case the O-rings are pushed inwards by the outside water pressure, causing a higher-pressure force and hence a greater friction into the sleeve.

The above influences on the measuring system, are in the order of magnitude of a few kPa and can only be observed in very soft soils. These effects will be reflected on the CPT results in the form of :

- A threshold value to start the friction sleeve moving. As long as the CPT-load is below the threshold value, the friction measurement gives a zero value.
- A measured friction value lower than expected, due to influencing effects of the seals.
- A slow response, because the very low soil friction loads are unable to overcome the influencing effects.

These are the criteria on which the improvement will be assessed. To keep the above factors within limits, CPT operators with experience in very soft soils, know the necessary measures to get accurate results, such as: Clean all cone parts thoroughly so free of dirt, accumulated dirt causes additional friction and prevents free movement of the sleeve; Replace all seals, used and damaged seals have different resistance values that do not match the ones used during calibration; Grease O-ring seals to reduce friction effects.

The effect of the pore pressure on the sleeve reading are also important influence on the accuracy of sleeve fiction measurements. When area ratios at upper and lower end of sleeve are not equal this has been shown to have significant effects in some cases (Lunne 2006). However even when the cone equal area ratio the pore pressures at the two ends of the sleeve can be different. In order to have representative sleeve friction results the acquired datasets should be corrected for the pore pressure effects. ASTM D5778_12 recognises that different end areas of the sleeve can cause unbalance end forces due to the water pressures and requires the usage of cones designed with equal end areas. Figures 2 and 3 shows the results a pressure chamber calibrations when the same pressure is applied to a cone of a equal sleeve area ratio, the results shows that the beta factor for the cone with the new assembly is very small (6.5 x 10-4) and 1000 times smaller than the alpha factor.

Sleeve friction measurment

The magnitude of the discussed influences on the friction sleeve, has been measured with highly sensitive calibration equipment. The results of these measurements and the effects of using a new corrugated spring assembly, providing a certain preload unto the friction sleeve, are discussed in this section.

Measuring of disturbing influences

A series of tests were carried out in the calibration facilities in order to quantify the magnitude of the resistances imposed by O-rings and seals. The calibration tests were conducted using 10 cm^2 cones with independent load cells (non subtraction). Measurements were recorded 40 times per second while the imposed force was increased continuously. The cones were loaded three times in order to check the repeatability of the measurements.

The initial test, shown in Figure 4, shows the results of sleeve load cell output, for a cone mounted with no O-rings or lip seals. The load cell response shows immediate linear response even when extremely low loads are applied, indicating good coupling between load cell and sleeve. A series of test were performed to establish the influences of the O-rings, the dirt seals and the combined effect of both. Results are presented in Figures 4, 5 and 6. When any of the sealing elements were mounted, the load cell showed delayed response and the output voltage increases only after certain load was applied (threshold resistance). The response was thereafter linear, indicating that once the threshold resistance is reached the load cell will keep producing accurate measurements. As expected, the resistance was larger for the dirt seals than for the internal O-rings (2.5 and 0.5 kPa, respectively).

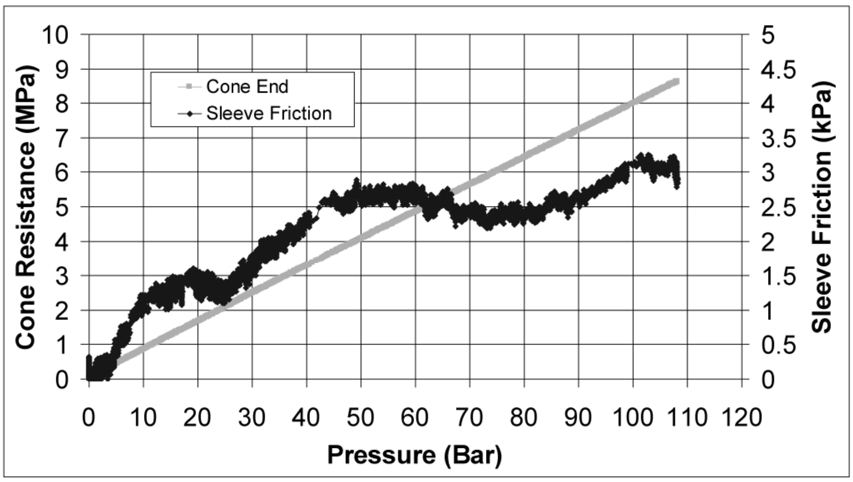

Fig. 2. Calibration of Beta factor (Sleeve vs chamber pressure)

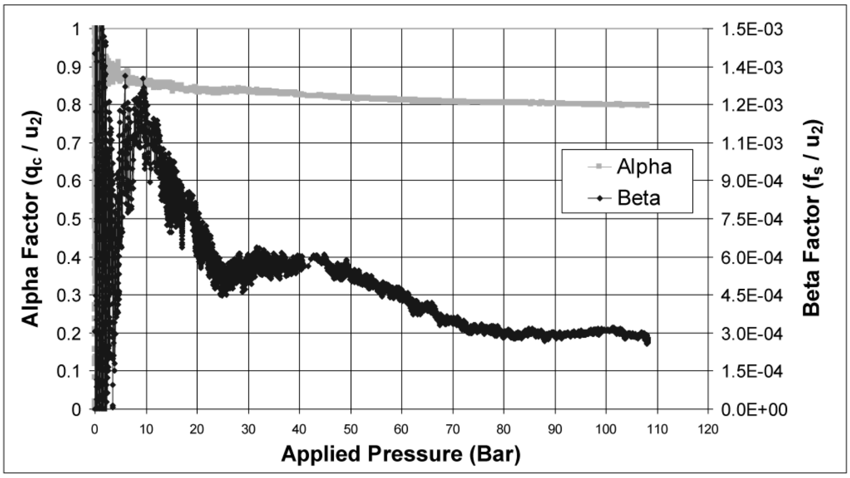

Fig. 3. Calibration of Beta factor (Beta vs chamber pressure)

Improvements

To improve the local friction measurements, in particular in very soft soils, a corrugated spring assembly a new dirt seal is added to the cone as shown in Figure 7. This spring assembly acts on the friction sleeve with a 3.5 kPa preload. This preload is sufficient to overcome the disturbing influences of the seals and the inertia of the friction sleeve. The dirt seals of the cone were changed from plastic 'V' shaped lip-seals to quad-o-rings (Figure 1). These rubber quad o rings prevent dirt getting trapped in the grooves at either end of the friction sleeve which allow it to move freely. This allows for a more responsive and repeatable measurement, with less chance of the friction sleeve becoming soiled and immobilised during testing.

The improved sleeve measuring system was also tested in the same calibration facility. Figures 8 and 9 show the results obtained from the classic design and the improved sleeve assembly respectively. The figures clearly show that the new assembly overcomes influence of the resistance caused by the seals, eliminating the threshold resistance effect, and reliable measurements are possible even at very low loads. Figure 10 and 11 shows that the behaviour of the load cell during an load –unload cycle is not affected by the introduction of the new spring assembly. The results from the calibration chamber showed not significant differences on hysteresis response and that the linearity improved due to the introduction of a small preload to overcome the resistance of the seals.

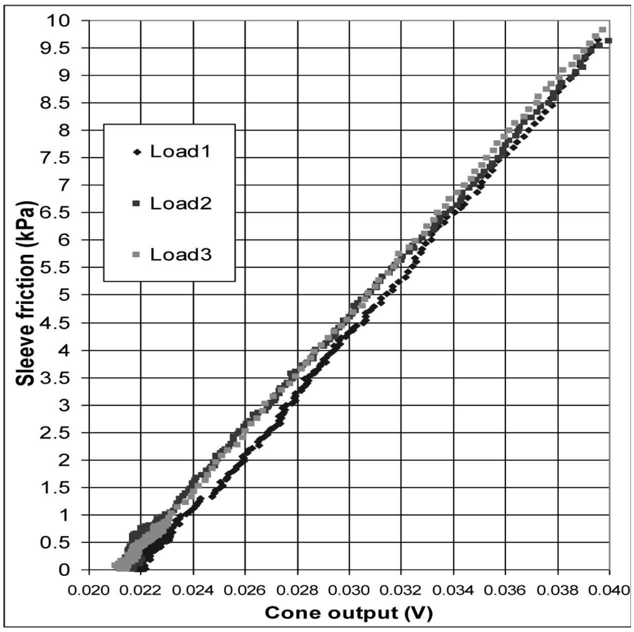

Fig. 4. Sleeve calibration with no seals mounted

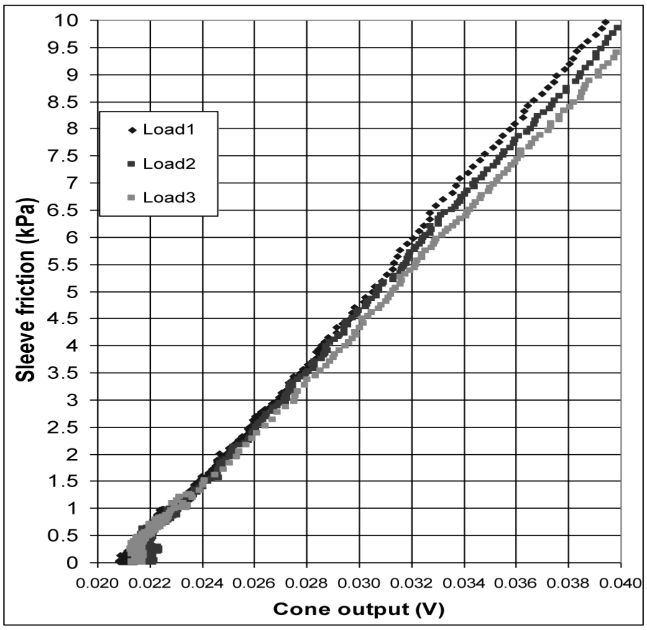

Fig. 5. Influence of O-rings on sleeve measurements

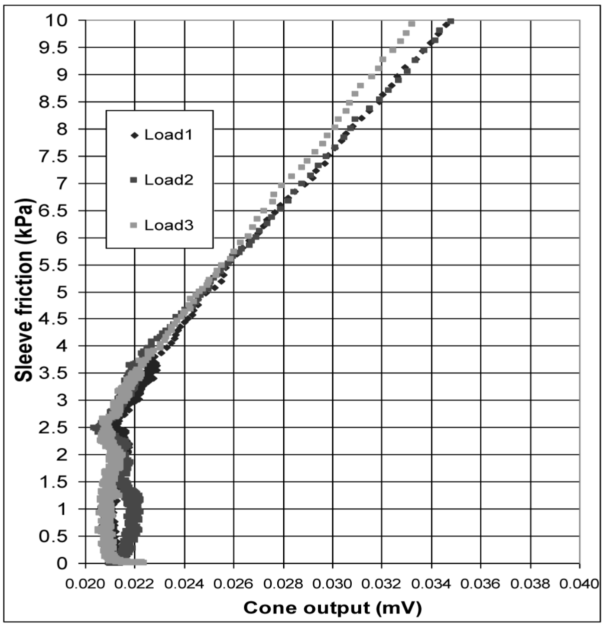

Fig. 6. Influence of dirt seals on sleeve measurements

Fig. 7. The corrugated spring assembly and dirt seal. (Courtesy of APVDBerg)

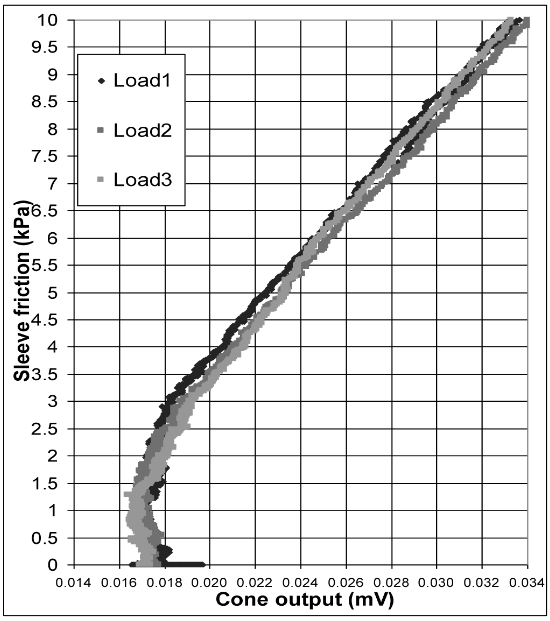

Fig. 8. Sleeve measurements on classic cone design

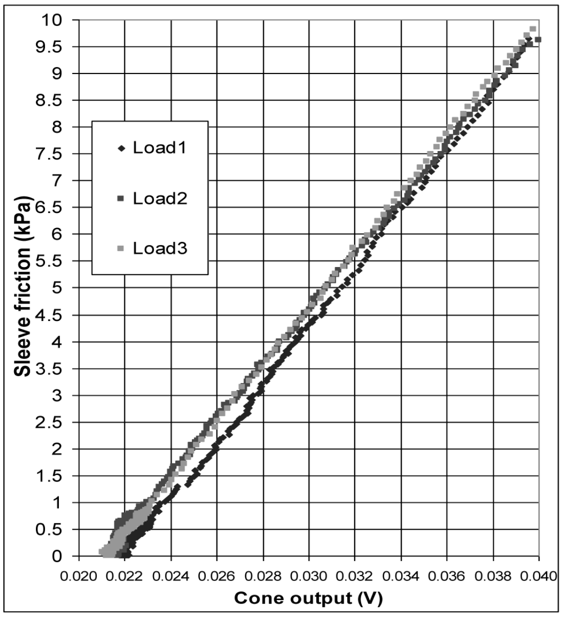

Fig. 9. Sleeve measurements with new spring assembly

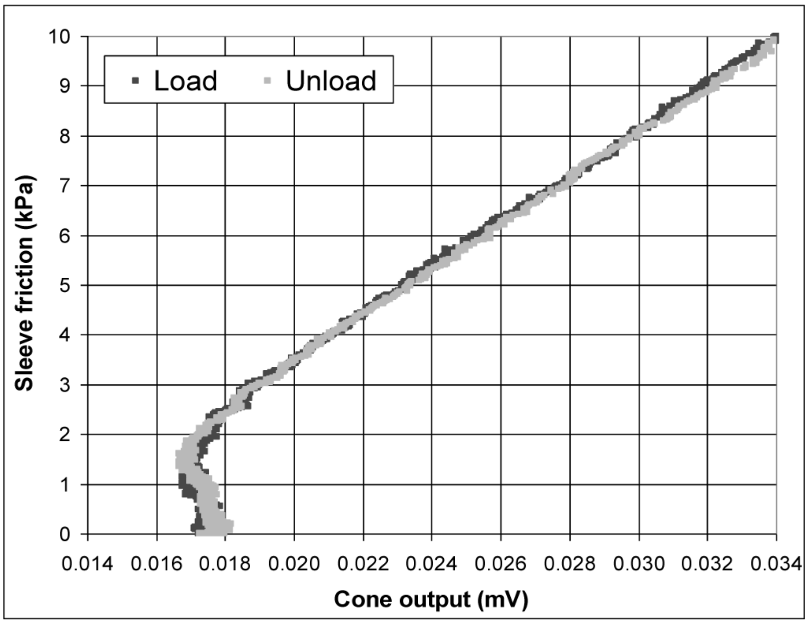

Fig. 10. Load – Unload on classic cone design

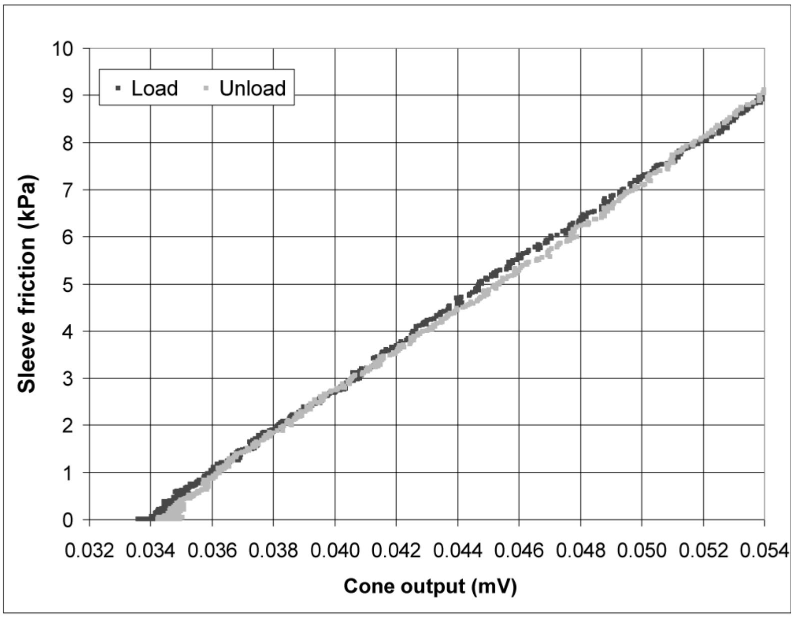

Fig.11. Load – Unload with new spring assembly

Overview of fieldwork

After the positive results obtained in the calibration facilities, the improved sleeve measuring system was tested in the field. Two sites were selected to test the benefit of the improved sleeve design, on land and offshore.

The offshore site is located in the Barents Sea in 300 m water depth. The site is composed of uniform normally consolidated soft glacio-marine clay with occasional gravel. The onshore site, Onsøy, is located in south-eastern Norway. This is a site characterized by very a thick deposit of remarkably uniform normally consolidated marine clay. This site is a very well known testing site for marine soft clays. The Onsøy site has been used by NGI as a soft clay test site for 40 years. This site is very suitable for testing the response of the cones and ideal for comparison of results with those published by Lunne et al (2003).

Method and implementation

The offshore tests were conducted with a seafloor mode CPT rig. At Onsøy the tests were conducted using discontinuous top push techniques, where the test is halted every time that a penetration rod is added to the system. The tests were carried in accordance with the ISO 22476-1 (2012). The accuracy and sensitivity of the cones and the acquisition system were within Application Class 1. The zero reading offsets were consistent before and after testing and do not indicate sensor drift effects. In addition, the tip resistance, pore pressure and sleeve friction measurements showed excellent responsiveness to layer changes and to the presence of small fragments of shell or gravel.

Results

Offshore site results

A total number of nine CPT tests were conducted on the site using the same cone. The distance between locations varies between 500m and 2000m. The results show relatively uniform profiles, considering the distances between locations. The vertical and horizontal variability in soil proprieties is however too high to be able to conduct a meaningful analysis of the repeatability of the measuring sensors.

The results are presented in terms of the corrected cone resistance, q_t, the measured pore pressure, u_2 and the measured sleeve friction, f_s (Figure 12). It can be seen that there is relatively more scatter in the measured sleeve friction values compared to the cone resistance and pore pressure values. The improved sleeve assembly shows significantly higher response, on average the values are 12 kPa higher. It is also observed that the response to changes in ground conditions is slower with the classical design. The pressure threshold for the sleeve measurement system is where the resistance offered by the seals is overcome by the frictional resistance imposed by the soil. The sleeve records obtained with the classical design are almost zero until the soil frictional resistance increases at 2 meters depth, where the resistance threshold is reached. The improved sleeve assembly results do not show any threshold effect and it reacted to very low loads, even in the first few centimetres of penetration. If we take the average values of sleeve friction recorded with the new sleeve set then the theoretical threshold is higher than measured in the calibration facilities. This is likely due to the influence of the ambient water pressure on the O-rings.

The outside of the cones were cleaned with sea water after every test. In the process the sleeve is turned and moved up in order to help the cleaning on the joints. However the sleeve was not disassembled to allow thorough cleaning of the dirt seals and gaps. The tests conducted using the classical cone design were done consecutively. It is observed that the responsiveness of the friction sleeve sensor decreased significantly after the first test. This indicates that dirt may have ingress between the dirt seals and sleeve. The tests that were conducted with the improved sleeve friction assembly did not suffer from these effects, confirming that the new design requires less maintenance and therefore the reliability of the results is less dependent on the operator.

Onsøy site results

Figure 13 shows the results obtained at Onsøy. The first two tests (CPT 1 and 2) were acquired using the classical design and subsequent tests were carried out using the improved sleeve assembly (CPT 3 to 8).It is clear that the improved sleeve assembly provides higher values and sharper response to lithological changes. The results obtained by the classical cone are offset by approximately 2.5 kPa. It is therefore apparent that the incorporation of the new seals and corrugated spring assembly increases the sensitivity of the cone in soft soils.

From an operational standpoint the results show that the improved sleeve assembly provided great advantages. The cones were cleaned with water after every test, and the sleeve was rotated and moved up and down in order to help remove any soil from the seals and gaps. This basic cleaning operation is sufficient for the new design; records show good repeatability and relatively low scatter (CPT 3-8).

CPT 1 and CPT 2 were carried out consecutively and the records show that the sleeve measuring system lost significant sensitivity and the load cell did not sense any load until 8 meters depth was reached. It is apparent that the dirt seals of the classic design allowed some dirt into the gaps which provided some extra resistance to the movement of the sleeve and the load cell did not contact the sleeve until the threshold pressure was reached.

The recently obtained results were compared with the CPT results published by Lunne et al (2008). The results compare cone types from six different manufacturers. The results can be seen in Figures 14 and 15. The cone resistance and pore water pressure results showed good agreement with the results obtained with other cone types. The sleeve friction records show significant more scatter, the total variance is 7 kPa, on average. It is interesting to observe that individual cone types results can be organized into two groups.

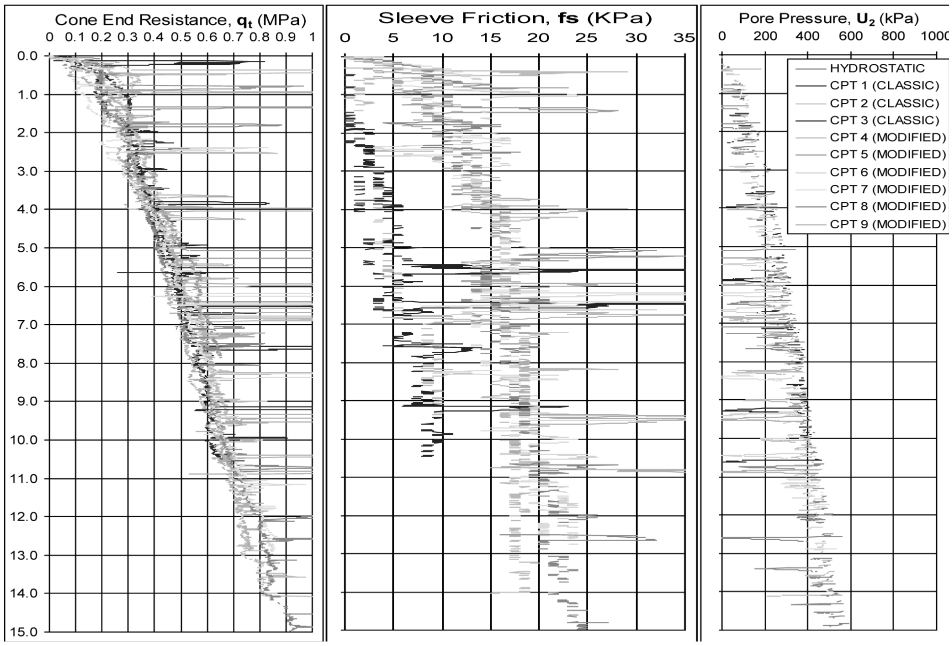

Fig. 12. Test results at offshore site Barents Sea

The datasets were corrected for the effects of unequal end areas and for differential pore pressure acting at sleeve ends (Figure 15). The pore water pressure at the u_3 position was not recorded and was assumed to be $0.75u_2$ based on previous experience at Onsoy (Lunne et al, 2003). The correction could only be performed to four of the cone types due to the limited information on the cone geometry on some of cone types. The correction reduced significantly the scatter and bring the two groups of results closer to each other.

It has been shown that small differences in cone design can result in significant changes to the measurements. The fact that the results could be grouped may be explained by differences/ similarities with cone design. As the true value of sleeve resistance in the ground is not known it is impossible to determine which one of the two groups of cones provides more accurate results. The reasons for these differences is unknown but several causes can be outlined:

- Different solution for the coupling between the sleeve friction and the measurement element to where the load cell is installed.
- Differences on sleeve friction roughness
- Difference of sleeve diameters. The current ISO standard allows differences of 0.35 mm which can result in a projected sleeve area 0.2 cm^2 greater than the cone tip.
- The determination of the sleeve end area ratio (beta factor) was done based on the dimensions on the sleeve provided by different manufacturers. The actual beta factor should be obtained in a calibration pressure chamber with the cone fully assembled with all its components including seals.

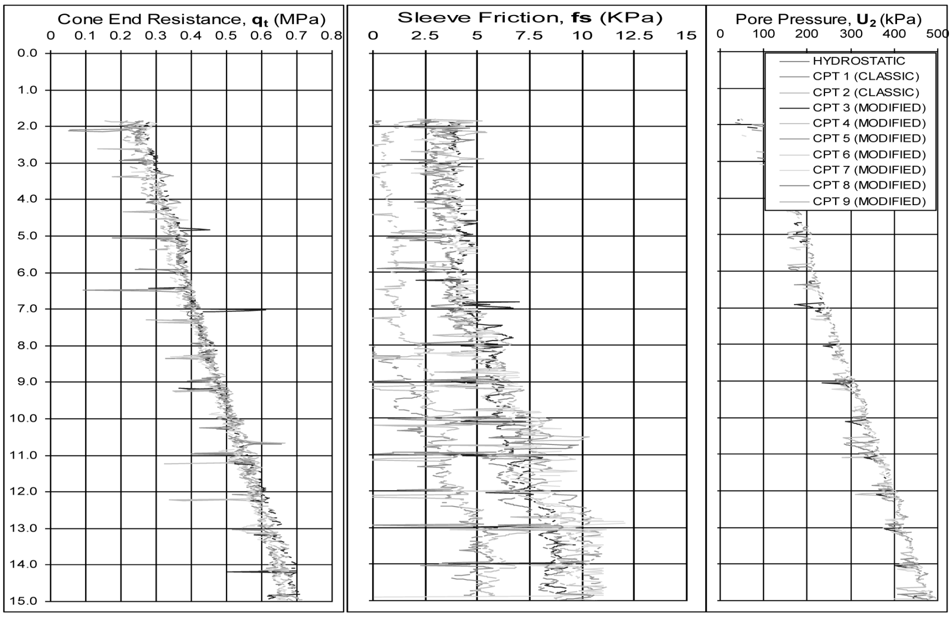

Fig. 13. Test results at Onsøy

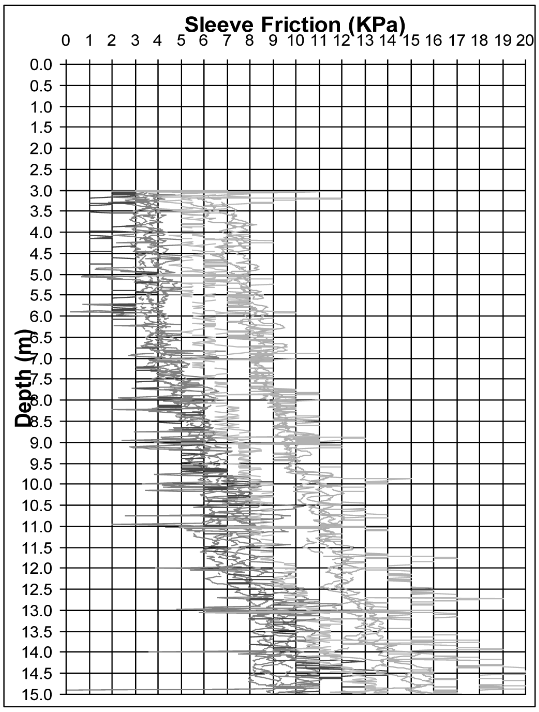

Fig. 14. Sleeve measurements comparison at Onsøy

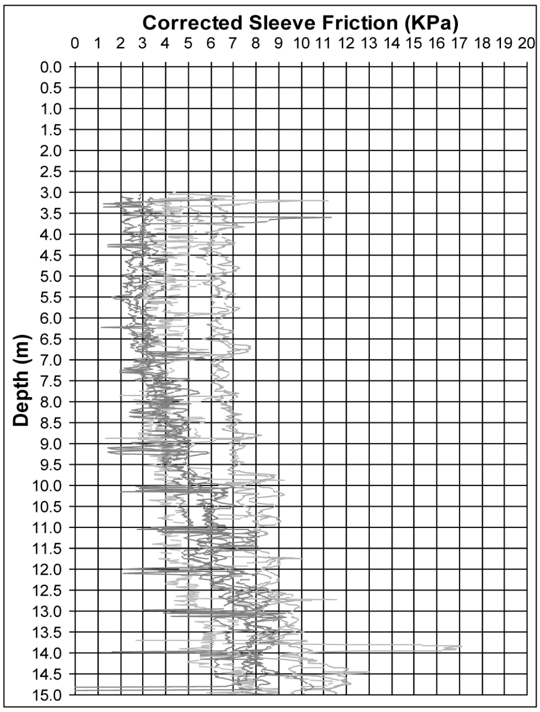

Fig. 15. Corrected sleeve measurements comparison

Conclusions

The review of the field and calibration database indicates that the mechanical improvements could increase the sensitivity of the sleeve measurement at low loads.

The incorporation of a corrugated spring eliminates any threshold resistance caused by the sleeve sealing system. The calibration and field results confirm that the load cell responds to extremely small load changes. The field tests show that the usage of the new sleeve assembly provides higher sleeve resistance readings and sharper response to soil changes due to the elimination of the frictional resistance of the seals. The elimination of the disturbances the seals impose to the measuring system contributes to the repeatability of the measurements.

The incorporation of new dirt seals prevents the entry of fine soil particles in the sleeve gaps more effectively. This is a very significant operational improvement as this allows consecutive tests without the need to conduct a full cone service between tests. In addition it makes the operation less prone to operator error.

The comparison with the other cone types from different manufacturers highlights that the requirements proposed by ISO 22476-1 (2012) for accuracy of sleeve measurement difficult to achieve given the current differences on cone sleeve friction designs between manufacturers.

The application of corrections for the sleeve friction measurements due to differential ambient pressures affecting the sleeve-ends can dramatically improve the repeatability of results obtained from different cone types. Given the importance of the cones geometry specially when measuring soft material the current standards should address stricter definition of cone dimensions and tolerances.

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Shear strength and deformation parameters of peat and gyttja from CPTU, SDMT and VT tests

Zbigniew Młynarek
Poznań University of Life Sciences, Poland
Jędrzej Wierzbicki
Adam Mickiewicz University in Poznań, Poland
Sławomir Gogolik
Poznań University of Life Sciences, Poland
Michał Bogucki
ILF Consulting Engineers, Poland

Abstract. The paper presents the results of the evaluation of shear strength and deformation parameters of organic subsoil by in situ tests. The results of the investigations confirmed the necessity to use different correction factors for estimation of constrained modulus from CPTU and DMT tests. The analyses of the relationship between undrained shear strength from vane test and CPTU proved that N_{kt} factor is strongly dependent on the type of organic soil and variables which define this soil. For the identification of organic subsoil the greatest consistency with laboratory assessment of the type of tested peat, gyttja and mud was obtained with the use of the DMT system by Marchetti and the CPT system by Schemrtmann.

Key words: organic soils, deformation and strength parameters, CPTU, DMT

Introduction

In the case of subsoil composed of organic soils we face two research problems. The first is to identify the zone and range of these soils in the subsoil, while the next problem is connected with the determination of their mechanical properties. The static penetration method (CPTU) is particularly suitable for the determination of zones of organic soils in subsoil, since characteristics recorded in this test present in a continuous manner changes in properties of the soil medium. When supplemented with dilatometric tests (DMT) and vane tests (VT) CPTU facilitates a comprehensive determination of mechanical parameters of organic subsoil at the geostatic state of stress.

Classification systems developed for CPTU e.g. by Robertson (1990) and for DMT (Marchetti, 1980) may be used to identify organic soils in the subsoil. Studies conducted by Młynarek (Młynarek et all. 2008) showed that these systems in the case of organic soils and gyttja have to be verified using laboratory analyses, since the diversity and high variability of properties of these soils have a different effect on recorded values of cone resistance in CPTU and measured values of pressure po and p_1 in DMT. The same problem is observed in the determination of mechanical parameters of peats and gyttja if we use empirical relationships established for mineral soils to determine undrained shear strength and constrained modulus. These problems are discussed in this paper. Material for analyses was provided by testing results of diverse organic subsoils by CPTU, DMT and VT.

Characteristics of the study area

Tests were conducted in three locations in Poland (Fig. 1). In all the locations organic soils originated from the Holocene and were found immediately under the surface. Soils of different types predominated in each of the test sites; in Poznań it was peats, in Stargard Szczeciński - gyttjas, while in the Żuławy area it was mainly organic mud (Fig. 2).

Fig. 1. Location of test sites

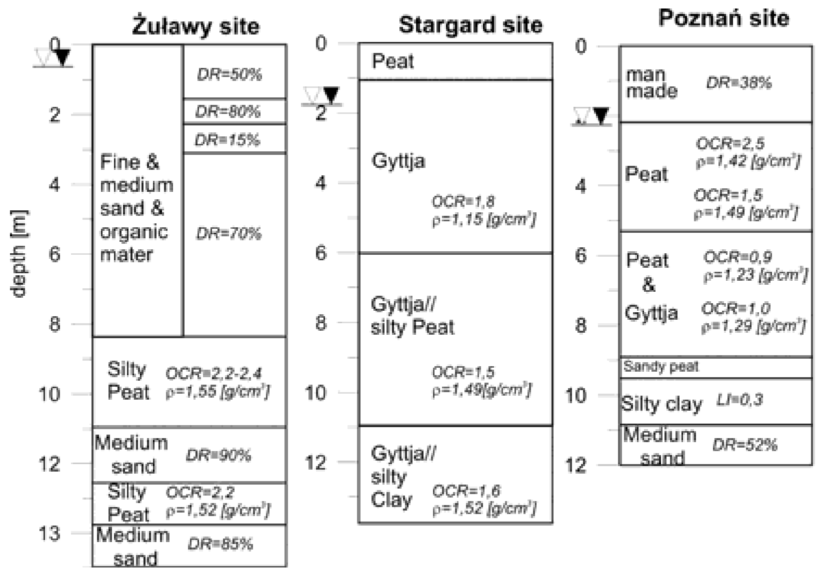

Fig. 2. Geotechnical profiles of the test sites.

Organic sediments in the Poznań location were deposited in the transition zone of the subglacial meltwater channel running from the north-west to the Warta (Młynarek et al. 2006). These deposits lie over Holocene fluvial sands, found within the channel and interbedded with typical glacilacustrine deposits. A characteristic feature of these soils is connected with the relatively high degree of consolidation in the subsurface zone, resulting from the fact that several decades ago a 2-meter embankment was constructed in this area. This is reflected in the position of these soils in the CPTU classification system (Chapter 4).

In the Stargard Szczeciński area organogenic soils constitute the upper section of Holocene lacustrine deposits of the systematically shallowing reservoir. The profile of organogenic soils is composed mainly of gyttjas, of up to around a dozen meters in thickness. Locally gyttjas are covered by a layer of peat.

In the Żuławy location the subsoil structure is determined by the accumulation and erosion processes typical of deltas. Characteristic features of organic layers in this case are their relatively small thickness (approx. 1 m) and their position at different depths of the profile. Soils in the test site also had increased values of the overconsolidation ratio, which may be explained by the effect of the quasi-preconsolidation processes connected with infiltration of waters in the region of the delta (Młynarek et al. 2008) and the effect of hydrodynamic pressure.

In each location CPTU, DMT and VT tests were performed and samples with an undisturbed structure were collected for laboratory analyses. Samples were collected using a MOSTAP apparatus by a. p. van den Berg as cores of 1 m in length and 65 mm in diameter. As an example the characteristics from CPTU and DMT for the Stargard location are presented in fig. 3.

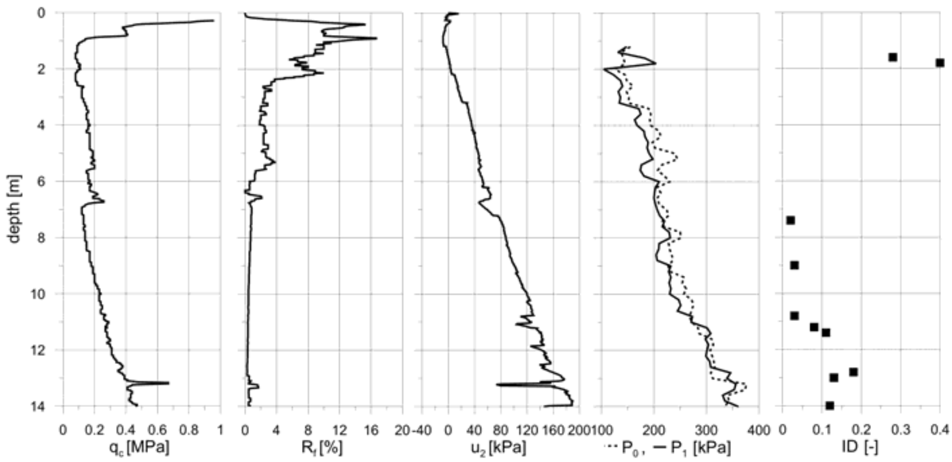

Fig. 3. Typical CPTU and DMT characteristics for Stargard Szczeciński test site.

The concept for the identification of organic deposits and gyttja by CPTU and DMT tests

Organic deposits found in land areas constitute a highly diverse group of soils. They are distinguished from the other soils by their genesis, as a result of which significant components of the soil skeleton include more or less decomposed plant residue and calcium carbonate. Thus a primary problem in the analysis of characteristics from in-situ testing in organic soils is to identify factors affecting parameters measured in static penetration process and DMT and VT tests. In order to establish factors affecting parameters measured in the course of these tests, which are identified by the characteristics from CPTU and DMT we may use functions, which describe these processes.

CPTU

The process of soil penetration during CPTU may generally be defined by the law describing the process of translocation of a material point in a medium showing friction (Banach 1950). For organic soils this process may then take the form of an equation (Młynarek 1978):

$$F(P, V_p, Q_1, Q_2) = 0 \tag{1}$$

where: P – measured parameters of the process, eg – q_c – cone resistance, f_s – sleeve friction, V_p – rate of penetration, Q_1 – characteristics of the soil medium, Q_2 – cone characteristics.
Sanglerat and Młynarek (1980) defined parameters Q_1 for organic subsoil in the form of a function:

$$Q_1 = f(X_1,\dots, X_{10}) \tag{2}$$

where: X_1 – effective unit weight of soil, X_2 – moisture content, X_3 – percentage of organic matter, X_4 – degree of decay, X_5 – context of $CaCO_3$, X_6 – percentages of clay, silt or sand fractions, X_7 – modulus of deformation of organic matter, X_8 – parameter describing shear strength, cohesion, angle of internal friction, X_9 – structure of organic soil, X_{10} – overburden stress (Młynarek et al. 2008).
In turn, parameter Q_2 is defined by the equation (Młynarek, 1978):

$$Q_2 = f(X^c{}_1, X^c{}_2, X^c{}_3) \tag{3}$$

where: $X^c{}_1$ – variable describing cone geometry (e.g. height, diameter), $X^c{}_2$ – coarseness of cone material, $X^c{}_3$ – deformation modulus of cone material.
It results from an analysis of equations 2 and 3 that function (1) is not dimensionally homogeneous and some of the variables are latent or discrete.

DMT

The form of the function, which describes the process of the dilatometric test in organic soils was proposed by Młynarek et al. (2008). The equation describing this process has the following form:

$$F_2\ (Pd, Vd, Q_1, Q_2) = 0 \tag{4}$$

where: Pd – measured process parameter, e.g. pressure p_0, p_1,Vd – velocity of pressure applied on the membrane of the dilatometer blade.
Parameter Q_1 is written as function (2), which is reduced by variable X8, as the essence of the test is to register small deformations before the ultimate limit state is reached in the subsoil.
Parameter Q_2 is described by the function

$$Q_2 = f(X^d{}_1, X^d{}_2, X^d{}_3) \tag{5}$$

where: $X^d{}_1$– a variable describing geometry of the membrane e.g. diameter, $X^d{}_2$ – membrane rigidity, $X^d{}_3$ – membrane roughness.
Function (4) is not homogeneous dimensionally. It results from the notation of equations (1) and (4) that from the formal point of view the effect of all variables on parameters measured in CPTU or DMT should be investigated in order to specify which of the variables may be

excluded when constructing empirical dependencies for the assessment of shear strength or constrained modulus of organic subsoil. Equations (1) and (4) make it possible to formulate two fundamental statements resulting from the fact that general solutions to equations (1) and (4) are not known to date. If CPTU, DMT, as well as VT (Młynarek, 2007) are performed using standard equipment, i.e. variables of function Q_2 are constant, the first statement says that the effect of variables from function Q_2 on parameters measured in CPTU and DMT may differ. Similarly as VT, also CPTU records parameters at large deformations (ultimate limit state), while in DMT it is at small deformations (Mayne, 2001). In the context of the first statement the second statement is constructed, which says that in the case of organic soils it will not be possible to construct general formulas for the determination of shear strength parameters or constrained modulus from CPTU and DMT irrespective of the type of organic soil. This statement was confirmed e.g. by studies conducted by Młynarek et all. (2008) and Lechowicz, Szymański (2002).

Identification of tested organic soils in subsoil based on CPTU and DMT classification systems

Robertson (2009) stated that the first step in the interpretation of subsoil properties based on in situ tests should be to use classification systems such as soil behavior charts in order to determine the direction of interpretation for CPTU and identify zones of the soils in subsoil. While in mineral soils such a procedure in most cases makes it possible to determine the general type of soil and its engineering properties (Ramsey 2010), as it is shown by experiences from Poland, in the case of organic soils not all testing methods provide satisfactory results (Młynarek et al. 2008) (Lechowicz, Szymański, 2002).

As a rule, a good identification of organic soils is ensured by the presentation of DMT results in the classification diagram proposed by Marchetti and Craps (1981) (Młynarek 2007, 2010). In the case of analyzed soils this observation was partly confirmed. Both tested peats and gyttjas are located in the lower limit zone of soils for mineral soils, in certain cases it was slightly above the limit defined by Marchetti for organic soils (Fig. 4). At the same time some of the results exceed the original scope of the diagram, indicating extremely low values of dilatometric parameters. Independently of the above statements, DMT testing makes it possible to verify the presence of organic soils in the analyzed profile and distinguish them from mineral soils. A very good and effective supplementation to this system is provided by the systems developed by Rabarijoelly (2013) and Larsson (1989). Those systems facilitate more precise separation of soils in the group into gyttjas, peats and organic mud. Such a possibility is also partly confirmed by this study, in which in this diagram gyttjas are relatively well separated from peats (Fig. 5). To construct these systems it is necessary to know corrected values of material index I_D (Larsson, 1989).

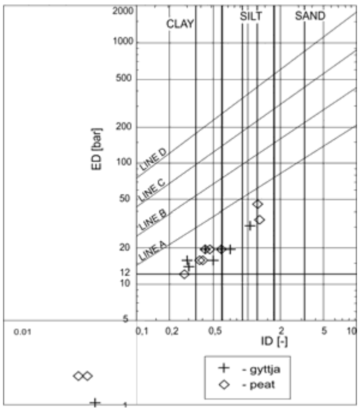

Fig. 4. Position of tested organic soils in the classification diagram by Marchetti and Craps (1981)

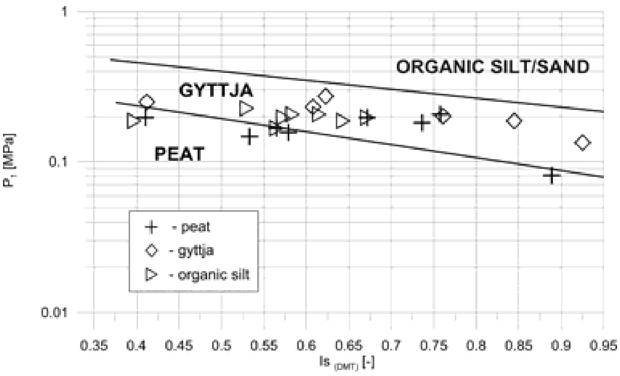

Fig. 5. Position of tested soils in the classification diagram by Rabarijoely (2013)

In contrast to DMT, in CPTU there are many classification charts available, created within the last several decades (Lunne et al. 1997). They are mostly systems using values of cone resistance and sleeve friction as well as their derivative parameters, such as friction ratio Rf or normalized cone resistance Q_t. One of the latest and at the same time the most frequently applied is the extended diagram by Robertson (2009), which original version dates back to 1990 (Robertson 2010). In the case of typical Holocene organic soils found in Poland, very often these soils are not correctly identified by that system (Młynarek et al. 2008). This observation is also confirmed by the results of the current study. Investigated soils should mostly be classified, in accordance with the Eurocode 7 definition, as low organic (< 20% Iom), but they are located away from the area marked for organic soils (Fig. 6)

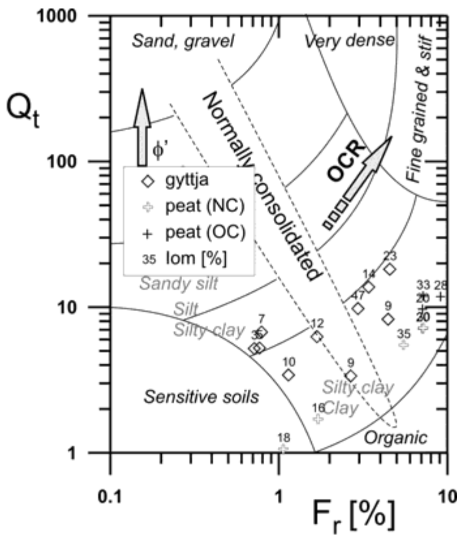

Fig. 6. Position of tested organic soils on the soil behavior chart by Robertson (1990) on the background of contents of organic matter (Iom)

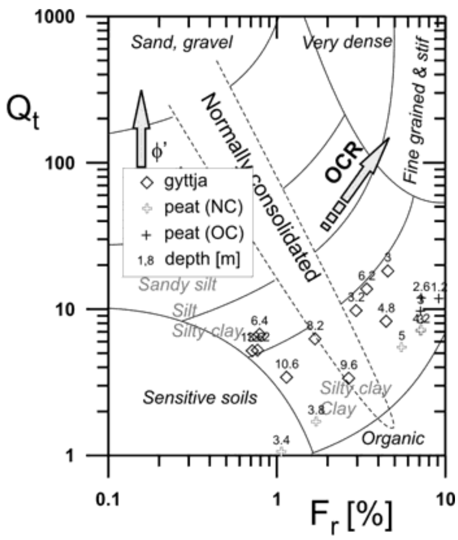

Fig. 7. Position of analyzed organic soils in the soil behavior chart by Robertson (1990) on the background of depth

The Robertson soil behavior chart indicates the preconsolidation effect in investigated soils. As it may be observed, this effect is mainly visible in the case of data from lesser depths, irrespective of the genesis, type of soil and its location (Fig. 7). This fact shows, as it was mentioned in Chapter 2, the occurrence of processes connected with cyclical changes in the level of soil waters and the effect of hydrodynamic pressure.

In order to determine factors affecting the position of soils in the soil behavior chart by Robertson, the location of these soils was verified in the chart proposed by Schemrtmann (1969). This system does not use normalized values of CPTU parameters, but their original values. In the case of analyzed organic soils the Schmertmann system is definitely more effective (Fig. 8). All soils are located in the zone of weak soils, while a trend may also be observed for the location of data to approach the zone of organic soils with an increase in the contents of organic fractions. The contents of organic fractions were reference data and they were determined in laboratory analyses.

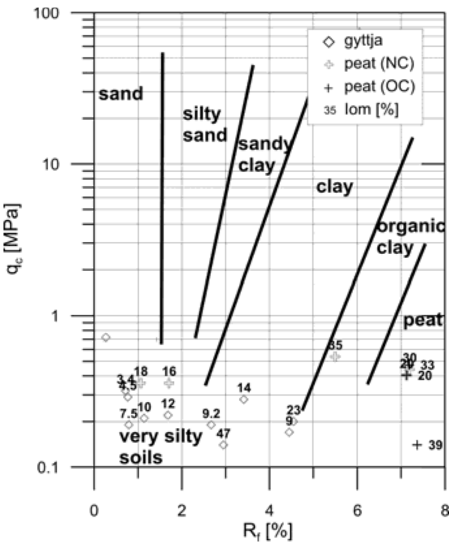

Fig. 8. Location of investigated soils in the classification diagram by Schmertmann (1969), numbers denote contents of organic fractions Iom.

The DMT classification systems and the system proposed by Schmertmann for CPTU make it possible to determine and explain causes for the low effectiveness of the systems using normalized and corrected values of cone resistance (Q_t) to identify suborganic soils in the subsoil. The causes may be assumed as follows:

- Młynarek et all. (2008) gave a partial function, which constitutes a partial solution to equation (1). Using multiple linear regression they established that the hierarchy of variables, which effect on the value of cone resistance q_t is the most significant, is as follows: the vertical component of geostatic stress v0, soil moisture content, dry density and OCR. If the content of organic fractions changes within a small range, the effect on changes in values of cone resistance is statistically non-significant. Determination of density and calculation of v0 values is burdened with high measurement uncertainty in organic soils and determination of the so-called relative values of dry density requires a large number of replications (Młynarek, 2010). In order to enhance the precision of volumetric density of investigated soils, in the profiles, in which no laboratory analyses were conducted, dry density was determined from the formula (Farkas & Kovacs, 1988):

$$\rho_d = (w/54{,}66)^{-0{,}787} \tag{6}$$

where: ρ_d – dry density of organic soil [g/cm^3], w – moisture [%].
This dependence showed a satisfactory consistency of the assessment of density with values determined in laboratory analyses.

- In the investigated soils relatively high values of q_c were recorded. This fact may also have a significant effect on the position of Q_t values in the classification system.
- In organic soils negative values of pressure u_2 are frequently recorded, which results in a reduced precision in the calculation of cone resistance Q_t. This problem is a separate issue, which obviously requires further studies.
- The classification system by Schmertmann is constructed directly based on measured values of cone resistance, similarly as the DMT (pressure p_1, p_0). These values are not normalized using stress v0 or corrected by pressure u_2. Thus this group of measurement uncertainty is not found in these systems.

Strength and deformation parameters of investigated organic soils from CPTU and DMT tests.

The CPTU method was applied to determine continuous changes in the subsoil the constrained modulus and shear strength modulus of analyzed soils. The following reference analyses were adopted in order to construct empirical dependencies or to verify existing dependencies between cone resistance q_c or q_t, and the constrained modulus, which corresponds to the oedometric modulus; for the determination of constrained modulus within the range of stress v0 + 100 kPa the moduli determined from DMT or laboratory tests. For the assessment of undrained shear strength su shear strength from the vane test (VT) was adopted as reference. Shear strength from this analysis was calculated from the formula:

$$\tau_{fv} = \frac{2M_{max}}{\pi D_v\left(H_v + \frac{D_v}{3}\right)} \tag{7}$$

where: M – maximum value of torque, H_v, D_v – geometry of the probe, i.e. height and diameter, respectively; v_s – shear velocity = const.
Values of shear strength τ_{fv} were corrected to values of shear strength from laboratory analysis using the index μ.

$$\tau_{fu} = \mu\tau_{fv} \tag{8}$$

Values of the index μ were adopted from a study by Młynarek et al. (1979) and Lechowicz & Szymański (2002), as mean μ !! 0.50 for peat and gyttja.

Constrained modulus of analyzed soils from CPTU and DMT

In the determination of constrained modulus of organic soils from CPTU the relationships developed for mineral soils are typically used. The most frequently applied formulas include:

$$M_0 = {}_{qc} \text{ Sanglerat (1972)} \tag{9}$$

$$M_0 = {}_1 (q_t - v0) \text{ Mayne (2001)} \tag{10}$$

where: q_t – corrected cone resistance, σv0 – overburden stress.
In the calculation of the constrained modulus of elasticity, which corresponds to the oedometric modulus, from the given formulas a problem appears in the determination of the index . It results from data in available literature that the index changes depending on the type of organic soils within a large range from 0.4 to 1.5 for peats and gyttjas and from 1 to approx. 8 for low organic soils.
Constrained modulus M0 from DMT is determined from the formula:

$$M_0^{DMT} = R_M \; E_D \tag{11}$$

where E_D – dilatometric modulus.

In the case of organic soils the coefficient R_M needs to be calibrated. Calibration using the oedometric test is the most advantageous. Calibrated formulas for certain organic soils from Poland are given by Lechowicz & Szymański (2002).
In the determination of calibrated values the modulus M_0^{DMT} of analyzed organic mud and peats oedometric tests were applied, which were conducted for organic soils from Poznań (Młynarek et al. 2006). Values of pressures p_0 and p_1 in the conducted in-situ tests exhibited large measurement uncertainties. This pertains particularly to gyttjas. In these soils frequently the values of pressures p_0 and p_1 did not differ (Fig.2). This fact prevented determination of values for modulus E_D. For this reason in the analysis of dependencies between moduli from CPTU and DMT a small number of observations was available.
In order to investigate a relationship between moduli M_0^{CPTU} and M_0^{DMT} a dependence was constructed between the modulus from DMT and net cone resistance q_n (Fig.9). This dependence makes it possible to formulate a general conclusion, which confirms a pre-formulated opinion that the index depends clearly on the type of organic soil. Estimated values of this index for peats amount to 10.2, while for organic mud they are 8.5. It also results from Fig. 9 that the investigated dependence has a relatively low statistical evaluation. This fact obviously results from two factors, i.e. measurement uncertainties connected with the determination of

q_c, p_0 and p_1 values in these soils, as well as low precision of assessment of values of moduli in oedometric tests due to the anisotropy of structure in the investigated soils.

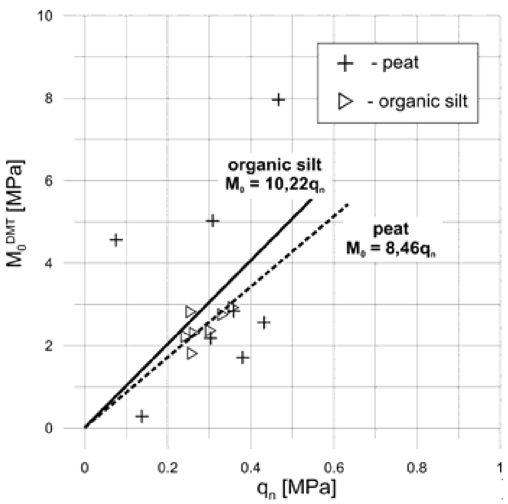

Fig. 9. Correlation between constrained moduli M_0 from DMT and q_n value from CPTU

Shear strength of analyzed organic soils from CPTU and VT

Lunne et al. (1997) formulated an opinion that no single undrained shear strength s_u exists. In the case of mineral soils the value of s_u depends on the mode of failure, soil anisotropy, strain role and stress history. In organic soils there are additional variables, which are found in equation (1). In order to determine s_u theoretical solutions are applied, which Lunne et al. (1997) grouped into five classes, while all theories result in relationships between cone resistance and s_u takes the form:

$$q_c = N_c \, s_u + \sigma_{v0} \tag{12}$$

where: q_c – cone resistance
The index Nc is transformed to the form of indexes N_k or N_{kt}

$$N_{kt} = (q_t - \sigma_{v0}) / s_u = q_n / s_u \tag{13}$$

Determination of index N_{kt} requires reference tests (Mayne 2006). In the group of these studies various schemes of laboratory analyses exist for triaxial test or vane test (VT). The values of N_{kt} depending on the reference test are greatly varied (e.g. De Groot & Luteneggar 2003, Mayne 2006). In turn, studies by Młynarek et all. (2008) showed that in organic soils there exists a significantly diverse hierarchy of the effect of variables from equation (1) on the value of the index N_{kt}.

In order to determine values of s_u the DMT method is also used. In contrast to CPTU, the basis for the establishment of s_u^{DMT} values is provided solely by empirical dependencies, constructed on measured values of pressures p_0 and p_1 (Marchetti 1981, Lechowicz & Szymański 2002).

It results from available literature that the value of N_{kt} for CPTU changes in the range from 12 to 21 (Long & Boylan 2012). Młynarek et al. (2006) showed that the value of this index amounting to 12 corresponds to the maximum shear strength τ_{fu}^{max} from VT, while $N_{kt} = 12$ for the value of residual strength from this test.

In the conducted analysis, as it was mentioned in 5.1., undrained shear strength τ_{fu}^{max} from VT was adopted as the reference test for the determination of undrained shear strength from CPTU. The analysis was conducted in two stages. In the first stage the value of net cone resistance q_n was compared with the value of τ_{fu}^{max} at respective levels of v0 in individual groups of soils (Fig. 10). Figure 10 documents a significant effect of the type of soils on undrained shear strength and a good correlation of this strength with net cone resistance. This relationship is curvilinear and in the investigated range of quantification for q_n the relationship between q_n and s_u may be described by equations:

$$\text{for organic mud: } s_u = 7.3e^{4,11q}{}_n + 7.0 \qquad (14)$$

$$\text{for gyttjas: } s_u = 6.5e^{4,11q}{}_n + 5.5 \qquad (15)$$

$$\text{for peats: } s_u = 11.8e^{4,11q}{}_n \qquad (16)$$

where q_n in MPa and s_u in kPa.

In the second stage the relationship was investigated between cone resistance q_n and undrained shear strength s_u from CPTU. This dependence actually determines the change in the index N_{kt} on the type of soil (Fig. 11).

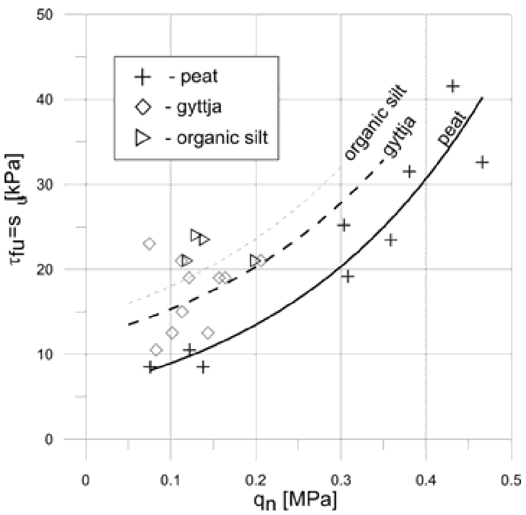

Fig. 10. Differences in correlation trends between maximal shear stress τ_{fu} from VT and net cone resistance q_n for different soil types

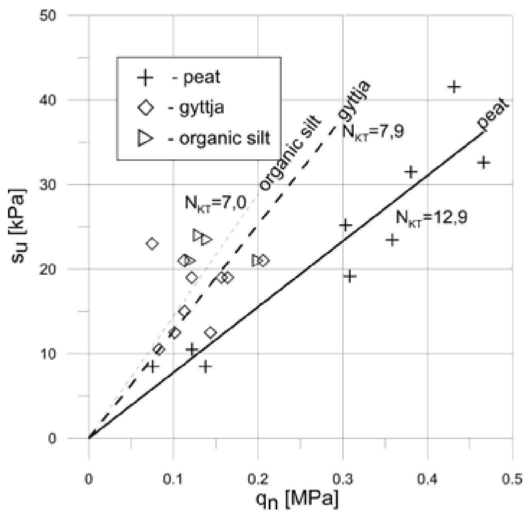

Fig. 11. Linear relationship between undrained shear strength s_u from VT and net cone resistance q_n, representing the N_{kt} values for different soil types

For practical reasons it is easier to use mean values of the index N_{kt}, which algebraically specify the slope of the straight line describing the relationship between strength s_u and cone resistance q_n (Fig. 11). Obtained values of the index N_{kt} perfectly confirm previously recorded values of N_{kt} for soils from Poland (Młynarek et al. 2006, Lechowicz & Szymański 2002).

A statistical assessment of quality of the dependence between undrained shear strength from VT and CPTU is presented in Fig. 12. An analysis was conducted jointly for 3 groups of tested soils. In order to determine values of s_u from CPTU a coefficient calculated from equations 10, 11 and 12 was introduced. The value of the correlation coefficient of 0.75 proves high statistical significance of this relationship.

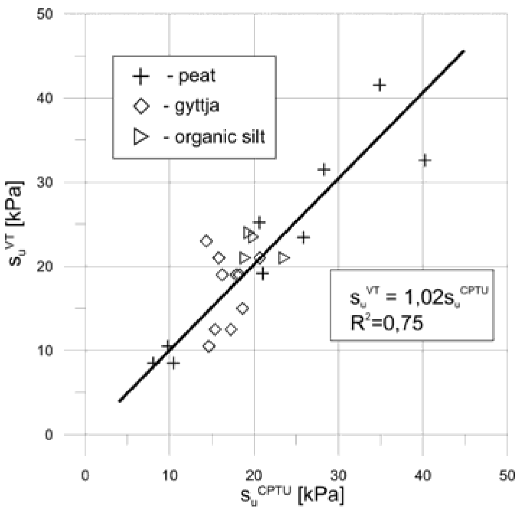

Fig. 12. Comparision of undrained shear values derived from vane test s_uVT and from piezocone test suCPTU , calculated on the basis of N_{kt} values given in Fig. 11.

Concluding remarks

Analyses of organic subsoil, which were conducted by CPTU and DMT, make it possible to formulate an opinion that identification of the zone and range of organic soils in the subsoil is feasible using classification systems from CPTU and DMT. The effectiveness of the systems varies due to the complex number of factor affecting parameters measured in CPTU (e.g. q_c, f_s) and DMT (p_0, p_1). The greatest consistency with the laboratory assessment of the type of tested peats, gyttjas and organic mud was obtained with the use of the DMT system by Marchetti and the CPTU system by Schmertmann.

Significant observations in the determination of the constrained moduli from CPTU and DMT show that the empirical coefficient (formulas 9,10) is dependent on the type of organic soil, which is described by several parameters such as contents of organic fractions, contents of carbonates and probably the mineral fractions. This conclusion may be formulated if the constrained moduli from DMT are adopted as reference in the determination of constrained modulus from CPTU. Very similar observations may be formulated for the determination of undrained shear strength from CPTU parameters if this strength is referred to strength from the vane test VT τ_{fu}^{max}. The index N_{kt} proved to be strongly dependent on the type of organic soil and the variables, which define this soil.

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Practical aspects of foundations on soft soils

Undrained shear strength for deep water field development in the Norwegian Sea

Tom Lunne, Knut H. Andersen, Shaoli L. Yang
Norwegian Geotechnical Institute, Oslo, Norway

Abstract. The Norwegian oil company Statoil is developing plans for the field development at the Luva site in the Norwegian Sea at about 1300 m water depth. A soil investigation was carried out with the aim of confirming the shallow geology at the site and to establish soil parameters for feasibility studies. Laboratory tests on samples obtained in the soil investigation revealed that the samples were disturbed due to stress relief and other factors and that the measurements gave soil parameters that may not be representative for the in situ conditions. A comprehensive study of the geology and the soil classification parameters at the site justified the correction of the laboratory measurements of undrained shear strength. The correction procedures applied were based on detailed research into the effects of sample disturbance on onshore clays

Clay; undrained shear strength; SHANSEP, sample quality

Introduction

LUVA gas field is located in the Norwegian sea at about 1300 m water depth (Figure 1). The site has not previously been mechanically overconsolidated. There is a large field of diapirs that partly penetrate to the seafloor and rise more than 100 m above seafloor 20 km west of Luva. To the east of LUVA, the slope is covered by furrows indicating downslope flow formed during the last glacial period. The seismic data recovered during the Luva soil investigation showed that the region has a uniform sediment in the uppermost 15 m, and no evidence for slope instability at the LUVA site or upslope has been observed (Tjelta & Yetginer 2010). However, evidence of mass movement is found 27 km north of LUVA. The main aim and objective of the LUVA site investigation was to develop a thorough understanding of the local geology and seafloor conditions at LUVA to assist with the field development. The detailed field program was aimed at resolving all early requirements for regional and local seafloor data for both geohazards and foundation studies. The geotechnical information was primarily to be used for the design of template foundations and anchors.

Soil investigation methods

After a thorough evaluation of available options Statoil decided to use the seabed founded drill rig PROD (Tjelta & Yetginer 2010). After the PROD is landed on the seafloor it is operated remotely from the support vessel (Kelleher & Hull 2008). For the Luva investigation the PROD was used to carry out the following field programme:
Four continuous CPTU profiles to 40 m below seafloor
Four boreholes with continuous sampling to 40 m
One continuous ball penetrometer profile to 20 m
Piezometer installations between 20 and 100 m below seafloor
The sampling was done with a piston sampler with liner and an inside diameter of 44 mm. Each sample was 2.7 m long and once on deck the liner was taken out and cut in 2 one m lengths and one 0.7 m length. To minimize soil disturbance during sampling the cutting edge and core catcher were modified on board as described by Tjelta & Yetginer (2010).

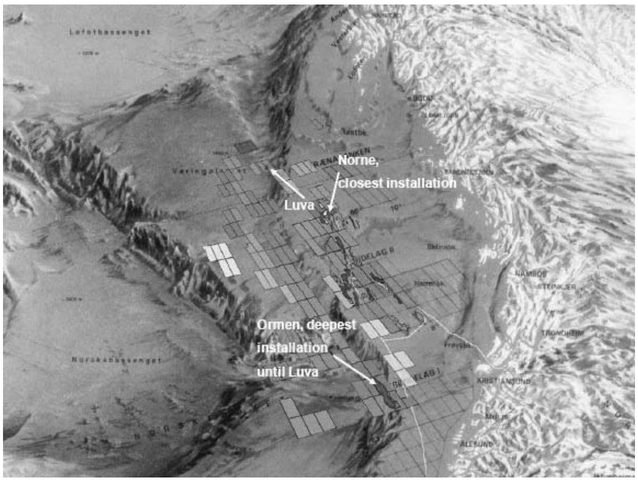

Fig. 1. LUVA location map

A comprehensive laboratory testing programme was carried out with the purpose to evaluate sample quality and to obtain data so that soil parameters for foundation design could be determined.

In addition to classification tests a large number of constant rate of strain (CRS) oedometer tests and anisotropically consolidated triaxial tests sheared undrained in compression (CAUC) were carried out.

Overview of soil conditions

Three soil units are identified in LUVA area based on the 4 CPTU profiles and sampling boreholes in the upper 40 m (Table 1). A soil profile (0-20 m) is shown in Fig. 2 as an example. Soil conditions are very uniform across the site from CPTU and ball penetrometer profiles.

Tab. 1. Soil units identified at LUVA area

Unit	Depth below seafloor, m	Soil description
I	0 – 12.5	CLAY, very soft, soft to medium
II	12.5 – 32	CLAY, medium to stiff
III	32-40 1)	CLAY, stiff

[1)] End of sampling

The overconsolidation ratio (OCR) was determined from oedometer tests using the Casagrande approach for interpretation of the preconsolidation pressure (Figure 3). OCR values interpreted from an empirical correlation with normalized CPTU cone penetration resistance are also included. The empirical correlation OCR = $k(q_{net}/_{VO}')$ has been used with k = 0.3 (Powell & Lunne 2003). $_{VO}'$ is the effective vertical stress.

Some years ago NGI developed a set of criteria for quantifying sample disturbance based on the change in void ratio relative to the initial void ratio, e/e_0, as measured in oedometer and CAU triaxial tests when consolidating a sample to the best estimate of in situ stresses (Lunne et al. (1997)).

Figure 4 shows e/e_0 values from oedometer and triaxial tests from the LUVA investigation. For most samples the quality is in the categories “good to fair” and “poor” with the latter being dominant below about 10 m. It turned out to be difficult to determine how much of the sample disturbance that was caused by tube sampling strains and how much that originated from stress relief.

Cracking of the samples due to gas exsolution was observed in one of the four boreholes, however the results of CAU and CRSC did not appear to indicate larger disturbance compared to the tests on samples from the other boreholes. The reason may be due to the fact that the laboratory always tried to use the best quality part of the sample in between the cracks. The formation of the cracking has been studied, and it was concluded that methane flux at the cracking site is higher than the other sites (Tjelta & Yetginer 2010). Regardless of the cause, the e/e_0 values shown in Fig. 4 show that especially below 10 – 15 m most of the samples are of poor quality. This means that the measured undrained shear strength, will not be representative for the in-situ conditions. In order to arrive at a more realistic and reliable undrained shear strength profile, a detailed evaluation of the available data was carried out with a basis in recent research on soil behavior for marine clays that have not been mechanically over consolidated.

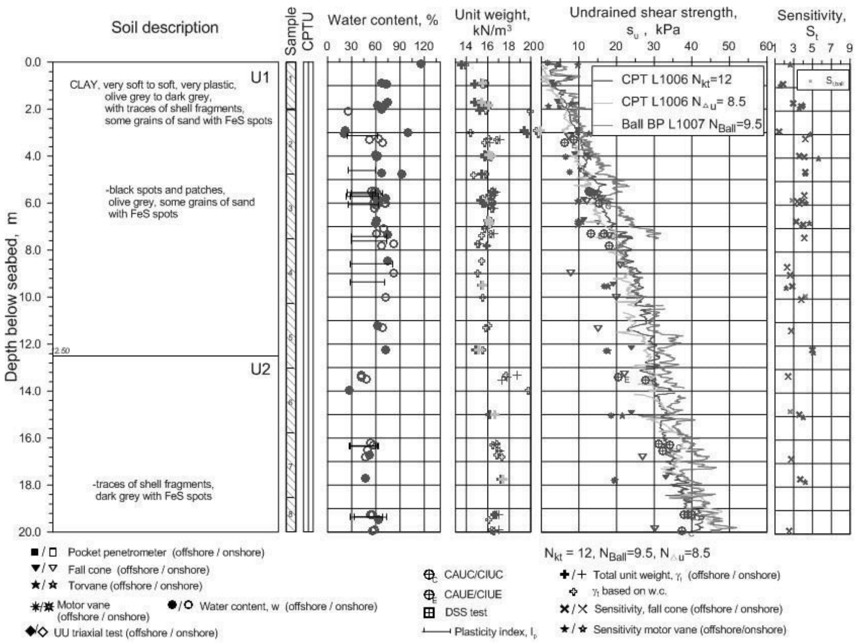

Fig. 2. Soil profile between 0-20 m for one borehole at the LUVA location

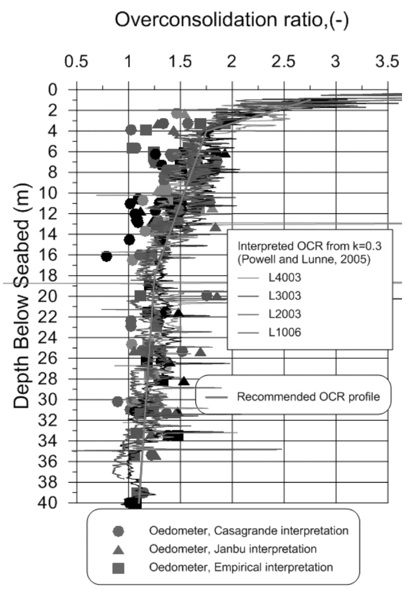

Fig. 3. OCR profile

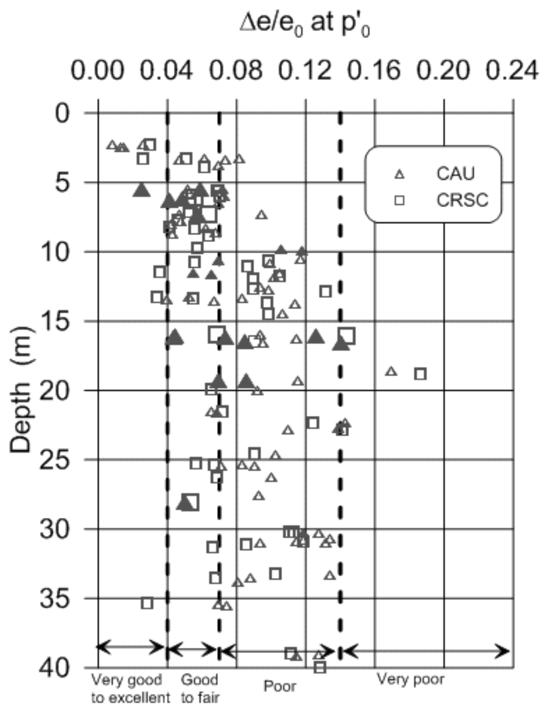

Fig. 4. Sample quality evaluation

Evaluation of undrained shear strength profile

Comparison with NGI's onshore test site at Onsøy

For 40 years NGI has had a soft clay test site at Onsøy about 100 km south of Oslo (Lunne et al. 2003). The geology at Onsøy and Luva is very similar. Both are clays deposited under marine conditions, have not been mechanically over consolidated, and have an apparent OCR due to aging effects. Figure 5 shows that the measured cone resistance and penetration pore pressure (u_2) are very similar. Further, classification parameters are very similar as evidenced by the plasticity index plot shown in Fig. 6.

NGI has used the Onsøy test site to develop empirical correlations between undrained shear strength determined by CAU tests on high quality block samples and samples of less good quality (Berre et al. 2007). R&D on interpretation of CPTU and full flow penetrometers in terms of undrained shear strength have led to recommended N factors for soft marine clays (e.g. Low et al. 2011).

Due to the proven similarity between the Onsøy and Luva clays, NGI and Statoil came to the conclusion that it would be justified to use correlations developed based on Onsøy data on the Luva site. However, it must be stressed that it is only because of this similarity that the corrections are justified.

It was decided to use four different approaches as considered in the following:

- Correction of undrained shear strength for sample disturbance based on the method of Berre et al. (2007)
- Correction of undrained shear strength for sample disturbance based on plots of s_u vs $\Delta e/e_0$.
- Use of SHANSEP parameters to obtain undrained shear strength
- Interpretation of undrained shear strength based on in-situ tests.

Correction of CAU tests based on Berre et al. (2007)

In a previous joint industry project (JIP) at NGI, it was found that sample disturbance has less influence on DSS and especially on CAUE tests compared to CAUC tests. The JIP concluded with a proposed procedure for correcting the undrained shear strength of soft clay at small strains from CAUC, CAUE and DSS tests (Berre et al. 2007).

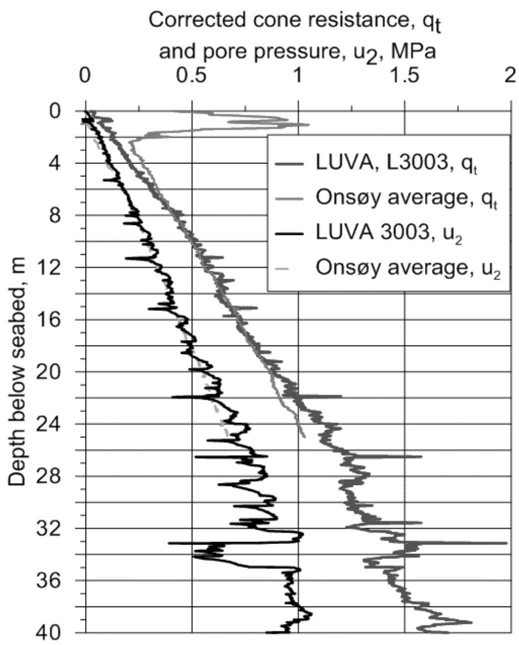

Fig. 5. Corrected cone resistance and pore pressure at LUVA and Onsøy areas

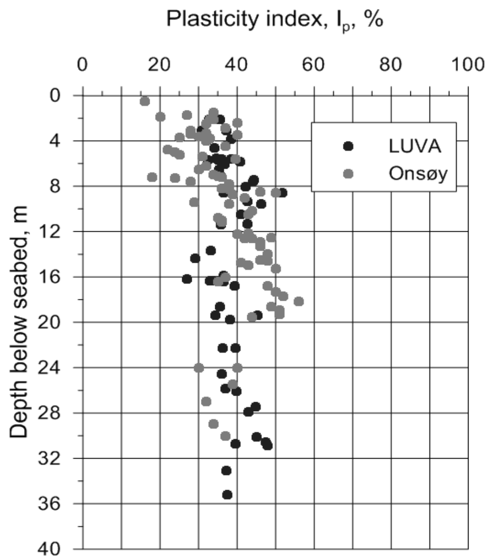

Fig. 6. Plasticity index for LUVA and Onsøy areas

The correction for a CAUC test on normally to lightly overconsolidated clay can be done by studying the stress path plot. The correction procedure consists of drawing one line from the origin through the peak shear stress point (point B) and one line vertically through the point at the start of shearing (point A) (see Figure 7). The shear stress at the intersection between these two lines (point D) is in most cases found to be close to the undrained shear strength for the high quality block samples. This was recommended by Berre et al. (2007) to be the corrected shear strength that one could potentially obtain on a sample of good quality. For other types of stress paths, please see the details for correction procedures in Berre et al. (2007).

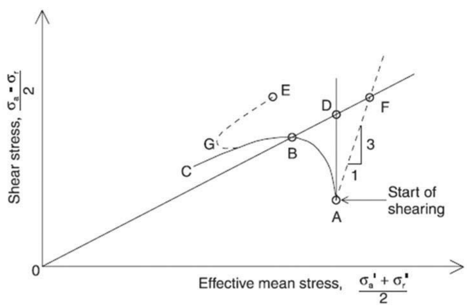

Fig. 7. Correction principle for normally to consolidated to lightly overconsolidated clay from Berre et al. (2007), small strain correction

Figure 8 shows both measured and corrected values of undrained shear strength as measured in CAUC. A best estimate profile for corrected s_u^C using this approach has been suggested in the figure.

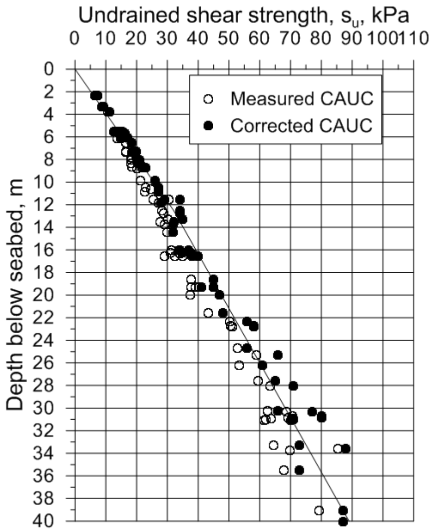

Fig. 8. Measured and corrected undrained shear strength from CAUC triaxial tests

Correction based on $\Delta e/e_0$

The change in normalized void ratio, $\Delta e/e_0$, has as mentioned above, been used to evaluate the sample quality (Lunne et al. 2006). In the following the normalized CAUC and CAUE shear strengths, s_u^C/σ_{vc}' and s_u^E/σ_{vc}', are plotted against $\Delta e/e_0$. Based on this relationship between s_u/σ_{vc}' and $\Delta e/e_0$, one can extrapolate to the shear strength at a low value of $\Delta e/e_0$. This value should represent the shear strength of a sample with minimal sample disturbance.
The normalized compression shear strength, s_u^C/σ_{vc}', from borings at the LUVA area is plotted in Figure 9. The results are sorted with respect to depth. There is a considerable scatter, but the general trend is that s_u^C/σ_{vc}' increases with decreasing $\Delta e/e_0$. Inspection of the data shows that the scatter is largely related to variation with depth within the data above and below 12.5 m.
Four curves are drawn based on the data. These curves are for depths of 2 m, 7 m, 12.5-32 m and >32 m, respectively. The data points and the curves indicate that s_u^C/σ_{vc}' increases as the depth decreases. This is reasonable, as the overconsolidation ratio will increase as the depth decreases and s_u^C/σ_{vc}' increases with increasing overconsolidation ratio.

Data from plastic Drammen clay and Onsøy clay are superimposed on the Luva data in Figure 9. These data are from both 54 mm samples and high quality block samples (Lunne et al. 2006, and internal NGI data). The Drammen clay data are for one given depth (~7 – 8 m) and eliminates the uncertainty related to the effect of depth and overconsolidation ratio. The same is true for some of the Onsøy data. The Drammen and Onsøy data support the trends interpreted from the Luva data, possibly with somewhat higher increase in strength at low $\Delta e/e_0$.

The strengths for OCR = 1, 1.5 and 2 are shown from the SHANSEP tests at the LUVA area in Figures 10 and 11 for depth of z>12.5 m and z<12.5 m, respectivley. Extrapolation of the curves copied from Figure 9 to low values of $\Delta e/e_0$ gives a shear strength approaching the SHANSEP shear strength of about OCR = 2.1 at 2 m, OCR = 1.7 at 7 m, OCR = 1.35 for depths between 12.5 and 32 m, and OCR = 1.13 for depths below 32m. This is in good agreement with the OCR-variation with depth shown in Figure 3.

The shear strengths corrected according to the procedure proposed by Berre et al. 2007 are also shown in the two figures (Figures 10 and 11). There is scatter in the corrected values, but in average they agree reasonably well with the strengths extrapolated from the curves at low $\Delta e/e_0$ with the extrapolated strength being somewhat higher.

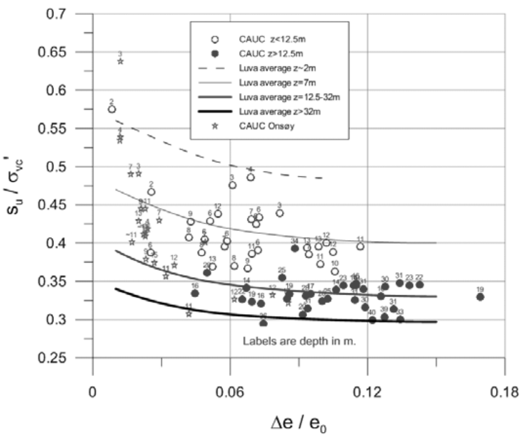

Fig. 9. Normalized compression shear strength as function of $\Delta e/e_0$

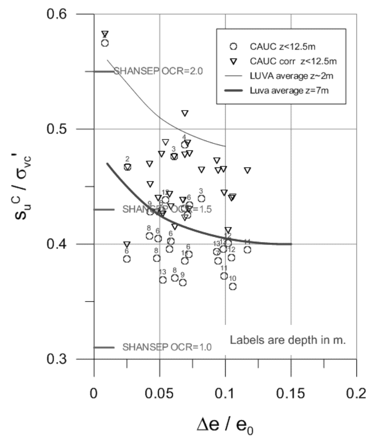

Fig. 10. Comparison of triaxial compression strength determination, z<12.5m

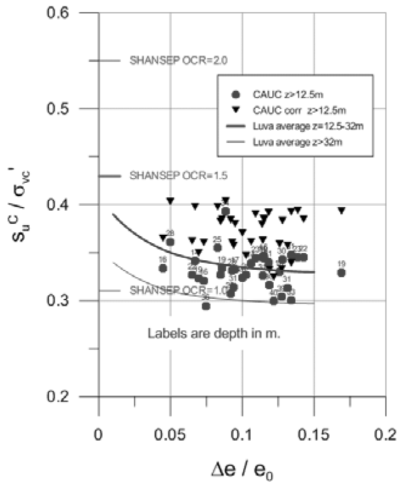

Fig. 11. Comparison of triaxial compression strength determination, z>12.5 m

Interpretation of undrained shear strength using SHANSEP parameters and OCR from CPTU

SHANSEP parameters (Ladd, 1991) were derived for the following relationship by performing a set of CAUC triaxial tests. The results were used to derive parameters for the SHANSEP equation:

$$\left(\frac{s_u}{\sigma_v'}\right)_{OC} = \left(\frac{s_u}{\sigma_v'}\right)_{NC} OCR^m \quad (1)$$

for CAUC tests $(s_u/\sigma_v')_{NC} = 0.31$ and m = 0.79.

Figure 12 shows suC found by using these SHANSEP parameters and the OCR profile recommended in Fig. 3. A best estimate profile for s_u^C using this approach is included in the plot.

Interpretation of undrained shear strength based on in situ tests

The four CPTU profiles to 35 – 40 m showed quite similar conditions across the Luva area (Figure 3). s_u^C was interpreted from the in situ test results using the following formula based on the CPTU and ball test results:

$s_u^C = (q_t - \sigma_{v0})/ N_{kt}$
$s_u^C = (u_2 - u_o)/ N_{\Delta u}$
$s_u^C = (q_{ball,corr})/ N_{ball}$

Luva clay is reasonably comparable to Onsøy and Troll clays as regards to geotechnical parameters. It is therefore a potential to evaluate N-factors based on high quality in situ and laboratory tests at Onsøy and Troll.

Tab. 2. Evaluation of N-values with Luva recommendations

Clay	N_{kt} Range/ (average)	N_u Range/ (average)	NT-$_{bar}$ Range/ (average)	N_{ball} Range/ (average)	Reference
Onsøy	10-13 / (12)	5–7 / (6)	8–9 / (8.5)	7.5–9 / (8.5)	NGI data
General soft clays- JIP	10–14 / (12)	4–9 / (6)	8.5–12.5 / (10.5)	8.5–12.5 / (10.5)	Low et al. 2010
Recommen-ded for Luva	12	8.5[Note]	9.5	9.5	

Note: By definition $N_u = N_{kt}$ * Bq. For Luva B_q varies between 0.6 and 0.8. If Nkt is taken as 12 as in Table 2 above, using B_q = 0.7, this means N_u = 8.5.

Figures 13 to 15 show suC computed from CPTU and ball data using the N-factors as recommended in Table 2. Each figure also includes a best estimate profile for suC.

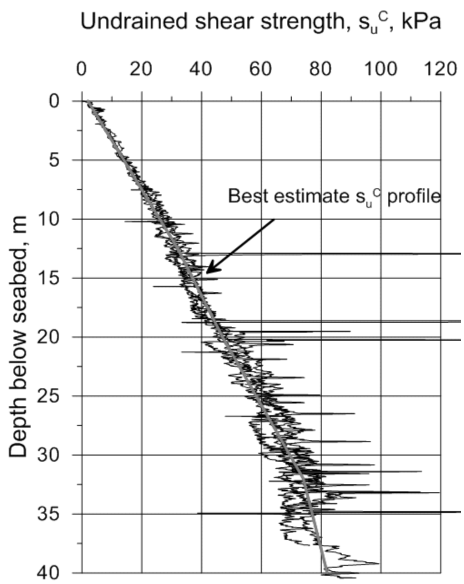

Fig. 12. Undrained shear strength interpreted based on SHANSEP parameters and OCR from CPTU

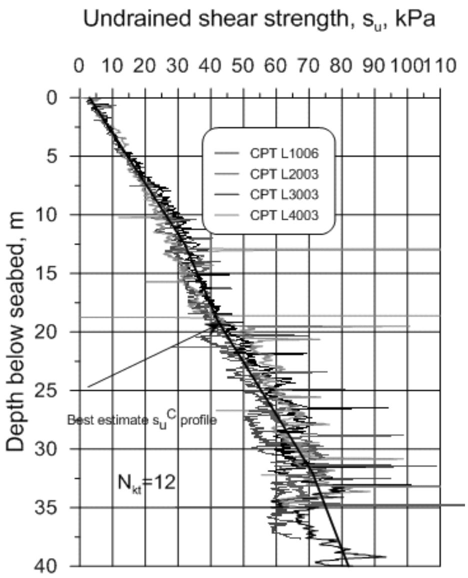

Fig. 13. Undrained shear strength from CPTU, N_{kt}

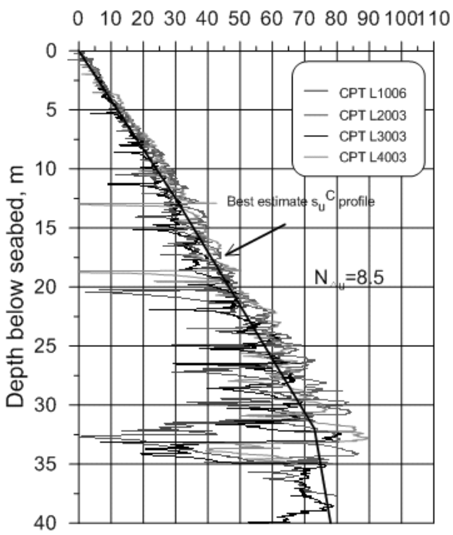

Fig. 14. Undrained shear strength from CPTU, $N_{\Delta u}$

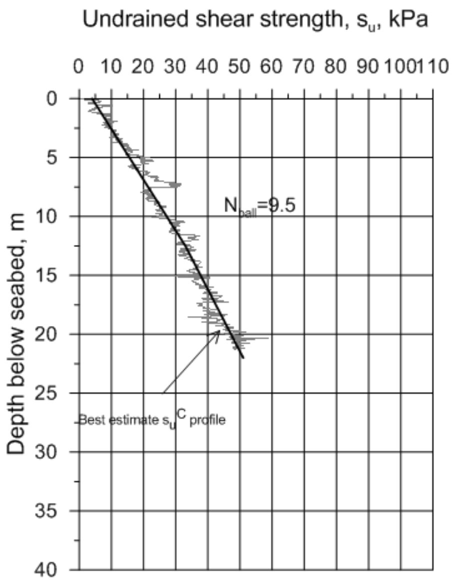

Fig. 15. Undrained shear strength from ball

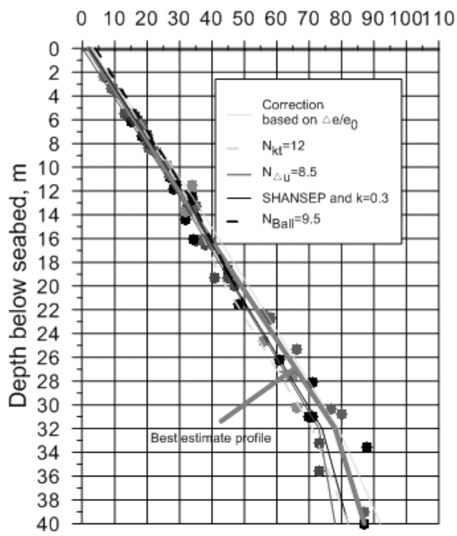

Fig. 16. Undrained shear strength from the four methods

Recommending undrained shear strength and anisotropy factors

Figure 16 summarises the undrained shear strength interpreted from the above four approaches. The corrected s_u^C values are given as points for each CAUC test, while for the other approaches best estimate lines are given.

Figure 16 includes an overall best estimate profile based on the four approaches discussed above.

The shear strengths and the corresponding shear strength anisotropy ratios are presented in Table 3 based on the e/e_0 approach for the evaluation of the undrained shear strength.

The strength anisotropy ratio is close to $s_u^E/s_u^C = 0.5$, which is a reasonable value for high quality samples, like Onsøy block samples and SHANSEP consolidated plastic Drammen clay at various overconsolidation ratios (Lunne & Andersen 2007; Andersen et al. 1988).

Tab. 3.Strengths and strength anisotropy ratios at low De/e_0-values

Depth (m)	s_u^C/svc’	s_u^E/svc’	s_u^E/s_u^C
2	0.56	0.285	0.51
7	0.47	0.235	0.50
12.5-32	0.39	0.20	0.51
>32	0.34	0.17	0.50

Taking into account that Onsøy data in the NGI data base on block samples gives s_u^E/s_u^C ratios varying from 0.48 to 0.51 it is believed that the values based on the $\Delta e/e_0$ correction is realistic. It is recommended to use an average of 0.51 over the depth interval of 0 – 40 m.

DSS tests were not performed in connection with the Luva soil investigation. Based on evaluation of laboratory tests on Onsøy block samples and other NGI in-house data an anisotropy ratio s_u^{DSS} / s_u^C of 0.67 is recommended.

Summary and conclusions

An advanced soil investigation was carried out at the Luva site in the Norwegian Sea in 1300 m water depth using a seafloor drilling rig. The laboratory tests showed that the samples were disturbed so that the measured strength and deformation parameters were not representative for in situ conditions. The sample disturbance is thought to be caused by a combination of tube sampling strains and large stress relief causing gas in solution in situ coming out of solution and disturbing the samples.

The clay at the Luva site is deposited under marine conditions and has never been mechanically over consolidated. However, it has an apparent over consolidation caused by aging. NGI's onshore soft clay test site at Onsøy has a very similar geological history as Luva. Measured soil properties are also very similar as evidenced by the measured CPTU cone resistance and pore pressure and also water content and liquid and plastic limits. Over several decades NGI has done research into soil behaviour on the Onsøy and other Norwegian soft marine clays. This research has resulted in reliable correlations between undrained shear strength measured on high quality block samples and less good quality tube samples. Correlations between undrained shear strength and results of CPTU and full flow penetrometers have also been developed. Due to the proven similarity between the Onsøy and the Luva site, Statoil and NGI were of the opinion that it was justified to use correlation developed for Norwegian onshore soft clay on the Luva clay measurements.

Based on the correlations developed for the onshore Marine clays four approaches were used for establishing an in situ undrained shear strength profile:

1. Correction based on correlation between block sample lab strength and somewhat disturbed tube samples
2. Correction based on measurements of the sample quality parameter De/e_0
3. Use of SHANSEP parameters
4. Interpretation of CPTU and ball penetrometer parameters

Undrained shear strength profiles using these four approaches confirmed each other and gave confidence to the recommended strength profile.

It is important to bear in mind that the corrections described in this paper could be applied only because of the comprehensive comparison of the Luva and Onsøy sites showing their similarity.

Acknowledgement

All members of the soil laboratory at Norwegian Geotechnical Institute were involved in the tests in this study, and their input was greatly appreciated. Statoil is greatly acknowledged for giving permission to publish this material.

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Additional parameters measured in a single CPT

Click-on modules for the digital cone

Mark Woollard, Onne Storteboom

A.P. van den Berg, Heerenveen, The Netherlands

Mauricio Coto Loria

MYV Soluciones Geotécnicas, San José, Costa Rica.

Abstract. The demand to build a comprehensive and accurate picture of the subsoil by using additional parameters from in-situ soil investigation is increasing. For example it may be required to derive the in-situ properties of both soil stratigraphy and soil elasticity to design a foundation that is subject to vibration; or both the soil density and soil electrical conductivity to allocate contaminated layers and predict future distribution. In general these parameters can only be acquired by separate systems (seismic, conductivity, magneto, etc.) and in subsequent tests. Apart from being time consuming, this process may also negatively affect the accuracy of the information obtained.

A data acquisition system which eliminates these drawbacks has been developed by the engineers from A.P. van den Berg. It consists of a digital data logger "Icontrol" and a digital cone "Icone", measuring the traditional CPT parameters. The Icone is easily extendable by click-on modules to measure additional parameters and any module is automatically recognized by the Icontrol, thus creating a true plug & play system.

This paper describes how, by moving to smart digital communication, sufficient bandwidth over a thin flexible measuring cable was created to accommodate additional parameters, without the need for changing cones, cables or data loggers. The following modules are available: seismic, conductivity, magneto and vane. Feedback from fieldwork with the Icone and a selection of click-on modules highlights the user experience with this new approach.

Keyword: CPT, digital cone, digital data transfer, click-on modules, seismic, conductivity, magneto, vane.

Introduction

Due to its benefits, digital technology is used in many applications and is now also available to support efficient soil investigation. The possibilities of this technology have led to the development of the digital cone, the digital data logger and digital click-on modules. These new developments are described extensively after a short historical review of CPT-techniques.

CPT investigation in historical perspective

Since Cone Penetration Testing (CPT) is used for soil investigation, a tremendous development has occurred in the techniques to measure soil parameters. After CPT with the mechanical cone had proven to be very useful, the development of the electrical cone brought a big step forward in ease of use and accuracy. Nowadays the advantages of digital technology are available for further improvements.

Mechanical cone

The first mechanical cone was developed in the 1930s and after two decades it was improved to a design that is still used today. The mechanical jacket cone allows to measure cone resistance and total resistance. The friction jacket cone also measures the local friction. The reading of these parameters is carried out by means of an electric measuring body. This device provides a continuous data stream, which is then stored and processed in real time by a computer system.

Fig. 1. Mechanical jacket cone and friction jacket cone

Mechanical cones are still widely used because of their low cost, simplicity and robustness. They are particularly useful in soil conditions with a high risk of cone breakage.

Because of its design and method of use the mechanical cone is limited in measuring soil parameters. For a jacket cone only cone resistance and total resistance can be measured, given a moderate accuracy due to friction between the inner rods and outer tubes. Furthermore, the mechanical cone is not suitable for investigating highly stratified soil, because a qualitative interpretation is impossible due to the coarse measuring data.

Electrical cone

The electrical cone has been developed during the 1950s and used commercially since the 1960s. Besides cone resistance and local friction, also the inclination was measured. In the 1970s the measurement of the pore water pressure was added which made soil investigation even more accurate and reliable.

The electrical cone is available with a 5, 10 and 15 cm^2 cone tip area, but the 10 cm^2 cone is most commonly used. The electrical cone is controlled by a data logger at surface level. Also the depth signal is recorded by the data logger and synchronized with the cone parameters

In comparison to the mechanical cone, the greatest advantage is that more soil parameters can be measured with a higher accuracy. A CPT performed with a pore water pressure measurement (U) enables a more reliable determination of stratification and soil type than a standard CPT. In addition, CPTU provides a better basis for interpreting the results in terms of mechanical soil properties.

The electrical cone is pushed into the soil with a constant rate and is therefore a more straightforward and faster way of soil investigation in comparison with the mechanical cone. Furthermore the real time processing and visualizing of the data obtained, facilitates a better control over the CPT-process.

The analog cone signals are subject to various disturbances especially during transfer via the cable and at connector transitions. The accuracy of an analog data acquisition system is therefore determined by the combined accuracy of individual components such as the cone, cables, connectors and the data logger. The calibration settings are essential for a good data processing and must be read from a USB-memory stick into the data processing computer system. Accidentally changing of calibration settings and getting rid of the USB-stick may lead to errors. Finally the electrical cone is more expensive compared to the mechanical cone, so financial losses are higher in case a cone breaks off.

Fig. 2. Electrical cone 10 cm^2 and 15 cm^2 area with analog data logger.

Digital cone

The digital cone or Icone has been available since 2006. The integration of intelligent electronics provides a range of possibilities in order to make further improvements to the electrical cone and to simplify its use. In addition, the Icone is stronger due to mechanical adjustments.

The Icone basically uses the same measuring sensors as applied in the analog cone. The difference however is that the analog signals are being digitized and multiplexed already inside the cone.

Digitizing means that the analog signals are being sampled with a certain frequency and converted into a digital data stream. This digital data stream is more robust, and therefore less sensitive to distortion and loss of accuracy in comparison with the analog signals. Another advantage of digital data transfer is the checking of signals on entry according to an established protocol. Missed or distorted data can be requested again.

By multiplexing, the various digital data streams are combined to one signal. This offers the great possibility that an almost unlimited amount of sensor signals can be combined and transmitted through a simple 4-wired cable. Also, specific sensors incorporated in customized modules can easily be added without changing cables and data loggers.

A built-in memory capacity gives the Icone several opportunities to increase the user friendliness. For example:

- The Icone number and calibration data are stored inside and are exchanged automatically when the Icone is connected to the Icontrol data logger.
- Extreme sensor values are stored in memory and can be read during service to explain possible calibration drift or damage to the Icone.
- The inside memory capacity allows the data storage of a full working day.

The Icontrol data logger provides power to the Icone and synchronizes the Icone signals with the depth signal, recorded from the pushing device. The Icontrol transmits the signals to a computer system, where the CPT-parameters are shown on real time graphs.

The use of smart electronics inside the Icone has provided the following benefits:

- The accuracy of the total data acquisition system is determined only by the accuracy of Icone calibration.
- Interchangeable click-on modules with specific sensors can be easily added to the Icone without the need of changing cables and data loggers.
- Specific sensors, added to the data acquisition system, are automatically recognized by the Icontrol and the corresponding display is automatically shown on the screen.
- Calibration data and Icone or module numbers are automatically transferred and are therefore no longer cause for errors.
- The Icone is able to monitor extreme measuring situations and overrule system control if needed.
- Several mechanical improvements have led to a stronger design.
- A pressure compensated Icone is available for water depths up to 4000 m.

Fig. 3. Icone 10 cm^2 and 15 cm^2 area with Icontrol data logger.

Icone and click-on modules

In the past five years several click-on modules for the Icone were developed. In this chapter the following three are described extensively: the seismic module, the conductivity module and the magneto module. All modules can be used with a 10 cm^2 and a 15 cm^2 Icone. When CPT-data are not required, the click-on modules can also be used with a dummy tip instead.

Seismic module

Seismic tests are performed to investigate the elastic properties of the soil. For this purpose a shear wave (S) or a compression wave (P) is guided into the soil by striking a hammer on a solid beam. Elastic soil properties are essential input for prediction of ground-surface motions related to earthquake excitation and for assessment of:

- Foundation design for vibrating equipment.
- Offshore structure behavior during wave loading.
- Deformations around excavations.

Principles

Elastic soil parameters are determined by measuring the propagation speed of an applied sound wave between two known depths. Mostly this is done by pushing the seismic module into the soil and stopping at 1 meter intervals. During the pause in penetration, a shear or compression wave is generated at surface level and the time required for the wave to reach the seismic sensors is recorded. The time difference between two consecutive seismic tests performed is a measure of the elastic properties of the soil. An even faster and more accurate way is to use two seismic modules which are mounted at a fixed distance of exactly 1 meter.

Since the time difference between two con-secutive measurements is approximately 2 ms, a very consistent measurement of the trigger signal is required. This requirement is met by using the same high sensitive sensors for the trigger module and by placing this module in the immediate vicinity of the hammer.

Fig. 4. Seismic module with 10 cm^2 Icone.

Technical specifications

The technical specifications of the seismic module are shown in table 1.

Tab. 1. Seismic module technical specifications

Item	Specification
Length	500 mm without Icone
Diameter	44 mm
Weight	4.8 kg without Icone
Sensors	Accelerometers X, Y and Z direction: - g-range: 2g – 50g - accuracy: 0.5% (FRO)
Data transfer	4-wire Icone cable inside CPT-rods
Connectors	Quadrax swivel connector to Icone Lemo 4-pins connector to Icontrol
Operating temperature	0° to 60°C

Data processing and visualizing

The output signals from the seismic sensors are being digitized inside the seismic module and from here transferred to the Icontrol data logger at surface level. From the Icontrol the signals are sent to a computer system where they are processed and recorded by the processing software GOnsite. After all tests have been performed, the data obtained are then analyzed off line by processing software, determining the propagation speed and corresponding elastic soil parameters for all investigated depth ranges. An example of how such processing can be visualized, is given in figure 5.

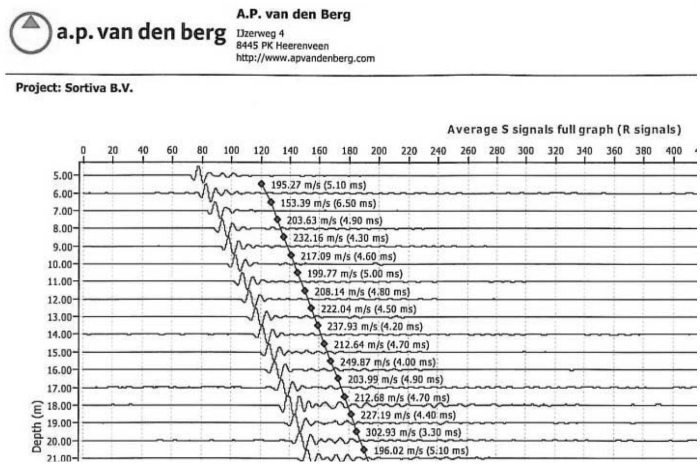

Fig. 5. Results of a series of seismic tests.

Conductivity module

The measurement of conductivity in the subsoil, facilitates separation of zones with differentiated water content, including determining the water table depth and the thickness of the zone of capillary ascent. Measurement of soil electrical conductivity is a function of both the conductivity of the pore fluid and the soil particles and how they are arranged. The dominant factor by far is electrical conductivity through the pore water. However, the most important application of the conductivity module is related to evaluating the degree of contamination of a soil medium containing electrolytes.

Determination of soil conductivity is not an absolute measurement, but indicates a change in soil condition. To what extent electrolytes are dissolved, is demonstrated without distinguishing these electrolytes. So only changes of the concentration of electrolytes dissolved in pore water are determined with the conductivity module.

Principles

Electrical conductivity of soils is not measured directly, but is derived from the measured voltage (V) across an electrode pair at a constant supplied current (I). According to Ohm's law, soil conductance (G) can be calculated as:

$$G = I / V \tag{3.1}$$

The electrical conductivity kappa (K) can be calculated with next formula:

$$K = C \cdot I / V \tag{3.2}$$

K is measured in milliSiemens per meter (mS/m) and C is a calibration factor which is found from direct calibration of the measurement module, whilst totally submerged in a solution of known conductivity.

Fig. 6. Conductivity module with 10 cm^2 Icone.

The conductivity module as shown in Figure 6 is equipped with four electrode rings which are isolated from each other by ceramic insulators. With a controlled voltage source inside the module, a known current (I) is induced through the soil between the outer electrodes. This current causes a voltage difference (V) across the inner electrodes, which voltage is held on a constant value of 50 mV by continuously controlling the current. When pushing the conductivity module into the soil with a constant rate, the conductivity is measured at depth using equation (3.2). To prevent polarization of the soil and precipitation of electrolytes on the electrodes, the voltage source operates with an alternating current at a frequency of 650 Hz.

The soil temperature is measured simultaneously with conductivity, because the solubility of an electrolyte is to a large extent dependent on temperature, and conductivity is mainly determined by the concentration of a dissolved electrolyte.

Technical specifications

The technical specifications of the conductivity module are shown in table 2 below.

Tab. 2. Conductivity module technical specifications

Item	Specification
Length	550 mm without Icone
Diameter	44 mm
Weight	3.7 kg without Icone
Sensors	Conductivity: - measuring range 50 - 1500 mS/m - accuracy: 0.5% (FRO) Temperature: - measuring range 0° - 50°C - accuracy: 2% (FRO)
Data transfer	4-wire Icone cable inside CPT-rods Wireless Optical data transfer
Connector	Quadrax swivel connector to Icone Lemo 4-pins connector to Icontrol
Operating temperature	0° to 60°C

Data processing and visualizing

In Figure 7 an example is shown of a conductivity module, measuring the salt water ingresses into the main land. The graph shows clearly that at the coordinates of measurement the transition of fresh to salt water is located at a depth of 15 meter.

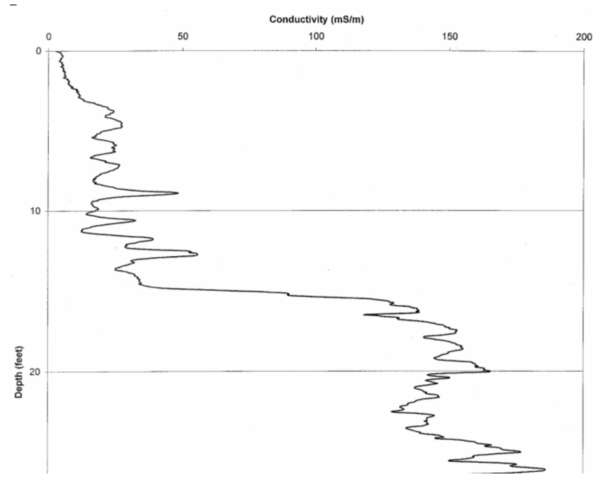

Fig. 7. Results of a conductivity test.

Magneto module

Unknown structures, obstacles like unexploded ordnance (UXO), and high voltage cables are a risk factor in the execution of earthworks. To avoid risks of damage and interruptions of work, these underground elements must be identified and mapped. Most underground structures are built out of metal such as sheet-piles, ground anchors and pipe lines or a combination of metal and concrete, such as reinforced foundation piles. Power supply cables and above structures have in common that they affect the earth's magnetic field.

Using the magneto module, metal objects in the underground can be detected by interpreting anomalies of the earth's magnetic field. In addition, the standard CPT-parameters can also be measured if the Icone is mounted in front of the magneto module.

Principles

The earth's magnetic field consists of power lines that run from North to South. Ferro metallic objects have the property to be influenced by the earth's magnetic field, causing them to act as a magnet themselves. This local magnetic field disturbs the earth's magnetic field in such a way, that the object can be detected and localized with a magnetometer.

The magnetometer sensor used is able to measure magnetic field anomalies in three orthogonal directions with an accuracy of 0.005 μT. Anomalies can be detected at a distance of 2 meter depending on the size of the object and the position relative to the natural North-South field lines. In practice it is not interesting to know the exact value of the magnetic field, but rather the difference in value at a particular location.

Fig. 8. Magneto module with 10 cm² Icone.

When the magneto module is used without the CPT-functionality of the Icone, the pushing rate can be increased from 2 cm/s to 20 cm/s. To accurately respond to changes in the measured value, in particular when detecting UXO's, also the gradients of the orthogonal measured anomalies are determined. With the GOnsite! processing software, alarm values can be set to stop pushing when one of these gradients is exceeded.

Technical specifications

The technical specifications of the magneto module are shown in table 3.

Tab. 3. Magneto module technical specifications.

Item	Specification
Length	600 mm without Icone
Diameter	44 mm
Weight	4.8 kg without Icone
Sensors	Magneto: - measuring range 0 – 100 μT - accuracy: 0.005 μT Inclination: - measuring range 0° - 20° - accuracy: 0.5° (FRO)
Data transfer	- 4-wire Icone cable inside CPT-rods - Wireless Optical data transfer
Connector	Quadrax swivel connector to Icone Lemo 4-pins connector to Icontrol
Operating temperature	0° to 60°C

Data processing and visualizing

The parameters measured by the magneto module are the anomaly of the earth's magnetic field in three orthogonal directions and the inclination relative to the vertical Z-axis. The gradients of the anomalies are determined for analysis and assessment purposes during measurement. The position of the magneto module in the Z-plane at the actual depth is calculated in order to know more precisely the position of the measured object.

The above mentioned parameters and gradients are shown in real time graphs. An example of these graphs is shown in figure 9.

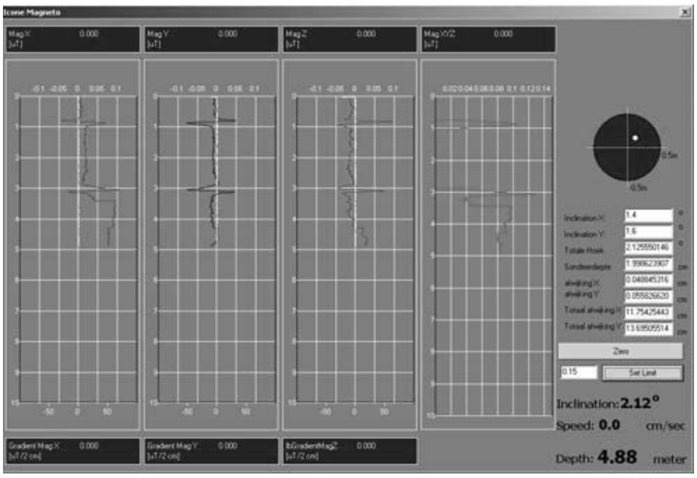

Fig. 9. Results of a magneto test.

Icone vane

The vane test is primarily used to determine the undrained shear strength s_u of saturated clay layers. The test can also be used in fine-grained soils such as silts, organic peat, tailings and other geomaterials where a prediction of the undrained shear strength is required.

Principles

The field vane test consists of four rectangular blades fixed at 90° angels to each other, that are pushed into the ground to the desired depth. Followed by the measurement of the torque required to produce rotation of the blades and hence the shearing of the soil. The chosen blade size depends on the stiffness of the soil in order to perform an accurate measurement; the stiffer the soil, the smaller the blades of the vane.

Fig. 10. Icone vane (without protection tube).

The new Icone vane has many features that facilitate an accurate vane test. The actuator is integrated in the same compact housing, enabling easier, faster and more accurate operation. The vane is pushed out of its protection tube and retracted again after the test. This advantage allows more vane tests at different depths without the need of retrieving the tool to surface level. The vane rotation speed is adjustable from 0.1 °/s for performing very accurate shear tests, up to 12 °/s for fast remoulding.

The vane tool is pushed into the soil by means of standard casing tubes and CPT-rods. Depth is measured on the pushing device and added to the field data by the Icontrol data logger. The vane field data is digitized and multiplexed using the same protocol of all other Icone applications, so no changes of data logger and cables are needed. The Icone vane tester can be used for onshore as well as offshore applications.

Technical specification

The technical specifications of the Icone vane tester are shown in table 4.

Tab. 4. Icone vane technical specifications.

Item	Specification
Length	1500 mm (incl. protection tube)
Diameter	90 mm (incl. protection tube)
Weight	14.6 kg (complete tool)
Sensors	Torque: - measuring range 0 – 100 Nm - accuracy: 0.5% FRO Inclination: - measuring range 0° - 20° - accuracy: 0.5°
Data transfer	4-wire Icone cable inside CPT-rods
Connector	Lemo 4-pins connector to Icontrol
Operating temperature	0° to 60°C

Data processing and visualizing

During a vane test, the vane is being rotated with a very low constant speed, while the required torque is measured with respect to the angel of rotation. This measured torque is analytically converted to the shearing resistance of the cylindrical failure surface of the vane used, and expressed in kPa. A typical shear curve is shown in figure 11.

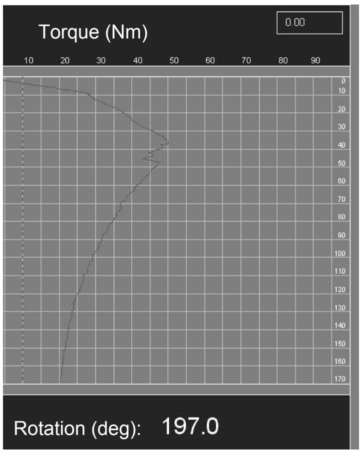

Fig. 11. Results of an Icone vane test.

The highest value of this curve is a measure for the undrained shear strength of the soil material that is being investigated. A repetition of this test, after thorough remoulding of the soil, provides a uniform curve of which the highest value is a measure of the remoulded shear strength.

Practical experiences

Seismic module

MYV Soluciones Geotécnicas recently acquired the latest version of the seismic module. The company in Costa Rica has used this module successfully in two different projects. The first exploration was conducted for the geotechnical investigation, required for the design of a rural bridge located in Cartago, Costa Rica. The second exploration was performed for the design of a steel storage tank foundation with a diameter of 20 meters, located in Puerto Corinto, Nicaragua.

Purpose investigation rural bridge

The main purposes in using the seismic cone at this site, was to:

- Determine the soil stratification.
- Estimate the bearing capacity of shallow foundations.

- Estimate the capacity of driven piles and select a foundation alternative.
- Classify the site for seismic design using V_{S30} methodology.

V_{S30} is a methodology for mapping and classification for seismic site effect evaluation.

Equipment used

For this investigation, a 23 ton CPT truck was used. As exploration tool, a 15 cm^2 piezocone was used along with the seismic module. To generate the shear wave, a Campanella pendulum hammer, like the one shown in the picture of Figure 12, worked as the dynamic source.

Fig. 12. 23 ton CPT truck.

Test results

Refusal was found at 18 meters depth due to the presence of boulders. In this respect, a complementary SASW-profile (Spectral Analysis of Surface Waves) was carried out to acquire V_S data from 18 meters down to 30 meters.

Figure 13 shows shear wave velocity profiles obtained with different methods at the project site.

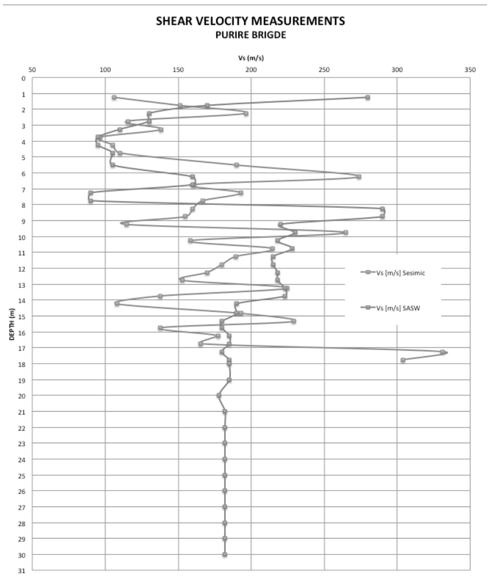

Fig. 13. Comparison between V_S from SCPTu and V_S from SASW.

The V_{S30} value obtained was 169 m/s, and based upon the Costa Rican Seismic Code, the soils classified as Site Class S4 (V_{S30} less than 180 m/s).

As observed in Figure 13, from the surface down to 8 meters depth, there is a good agreement between the V_s profiles obtained by both tests (SASW and SCPTu). From 8 meters down, V_s from SASW tends to be higher in average than the V_s from SCPTu. For the estimation of V_{S30}, the results from direct measurements (SCPTu) were considered more reliable.

As a result of the soil investigation, the foundation for the abutments of the bridge was designed using driven pile foundations.

Purpose investigation steel storage tank

The main purpose of the geotechnical investigation in Puerto Corinto, Nicaragua, was to obtain good quality soil data to design the foundation of the storage tank. Previous soil explorations with SPT classified the soil deposit as prone to liquefaction from the surface down to 19 meters depth. A second opinion and reassessment of the liquefaction potential was required by the owners to optimize the foundations.

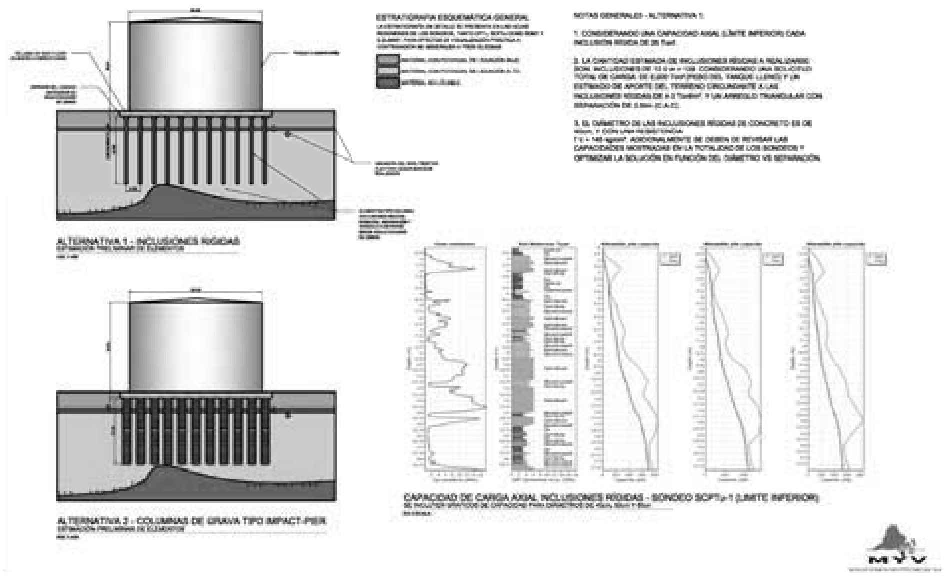

Fig. 14. Proposed foundation for the tank.

Equipment used

For this site, the geotechnical campaign included SDMT soundings as well as SCPTu soundings. As part of the scope, CPT disturbed samples were retrieved with MOSTAP Samplers. The heavy CPT truck was also used for this site.

Test results

According to the CPT tests, from the surface down to 4 meters depth, an old manually built fill layer was encountered. Based on some accounts, this fill was placed by the Nicaraguan government during the construction of Corinto Port. From 4 meters down to 19 meters depth, a medium dense sand deposit was found. Refusal was encountered at 19 meters depth which, according to previous soil reports, corresponds to the bedrock.

The next figure shows the geotechnical profile defined in each sounding.

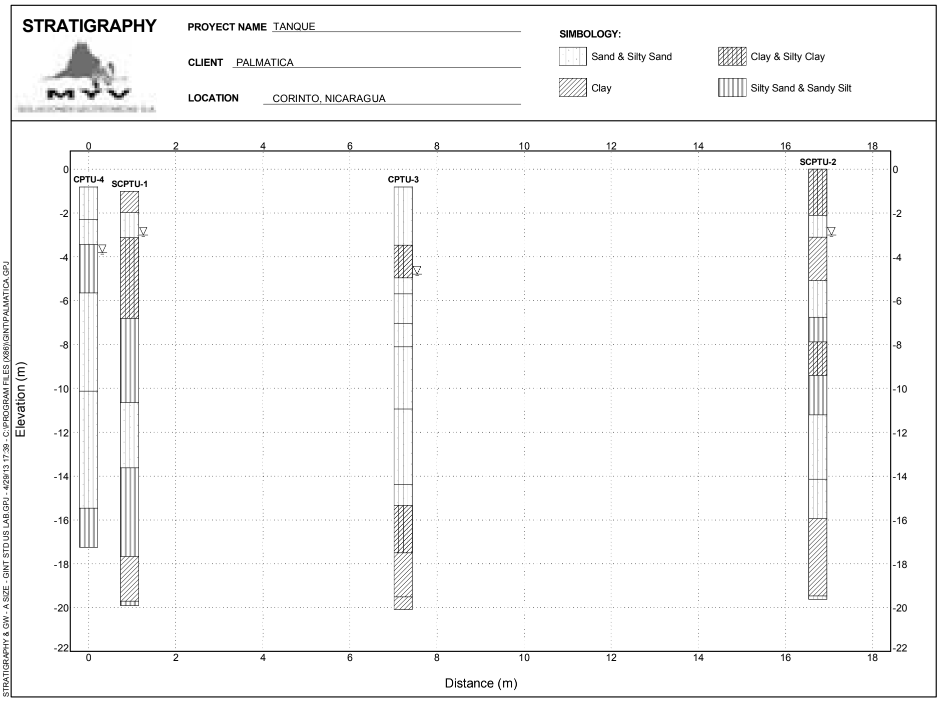

Fig. 15. Layer stratigraphy of the site.

Three different methods were used to estimate the liquefaction potential of the sand deposit: CPT-based method (Robertson 2009), DMT-based method (Monaco and Marchetti, 2007), and Vs-based method (Kayen et al, 2012). Based on the results from the above methodologies, the liquefaction prone layers were located from the surface to 13 meters depth. This finding was considered to be most beneficial in the foundation design of the tank.

Figure 15 shows the record of one of the seismic tests carried out with the seismic module.

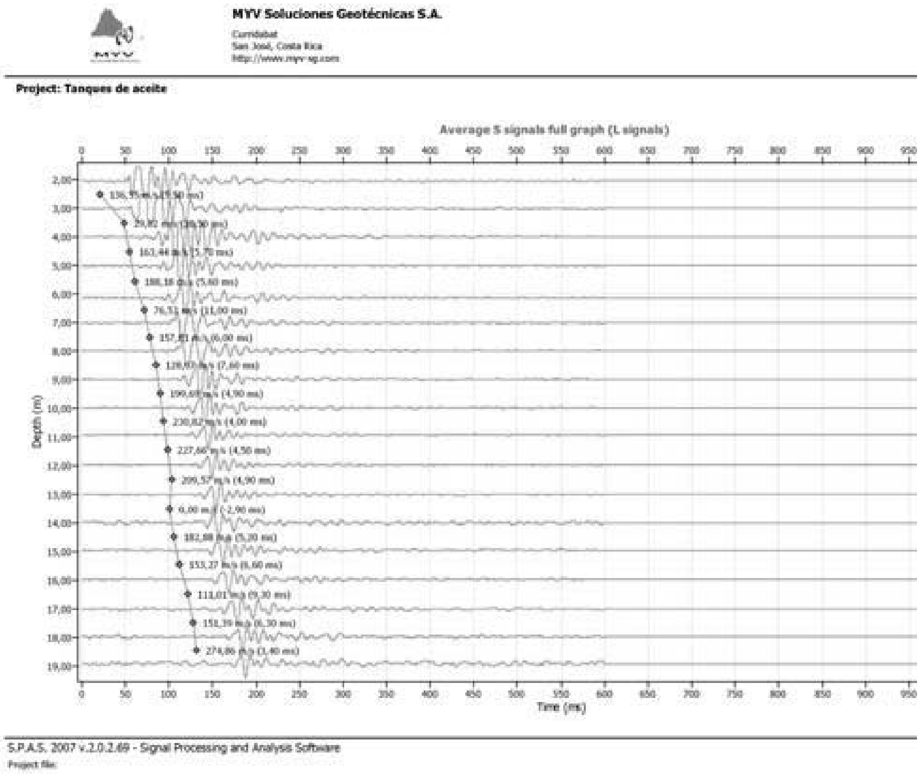

Fig. 15. Data output of the Seismic module.

Given that 2 seismic dilatometer and 2 seismic CPTs were carried out, a comparison among the different shear velocity profiles was possible.

The results of this comparison are shown in figure 16.

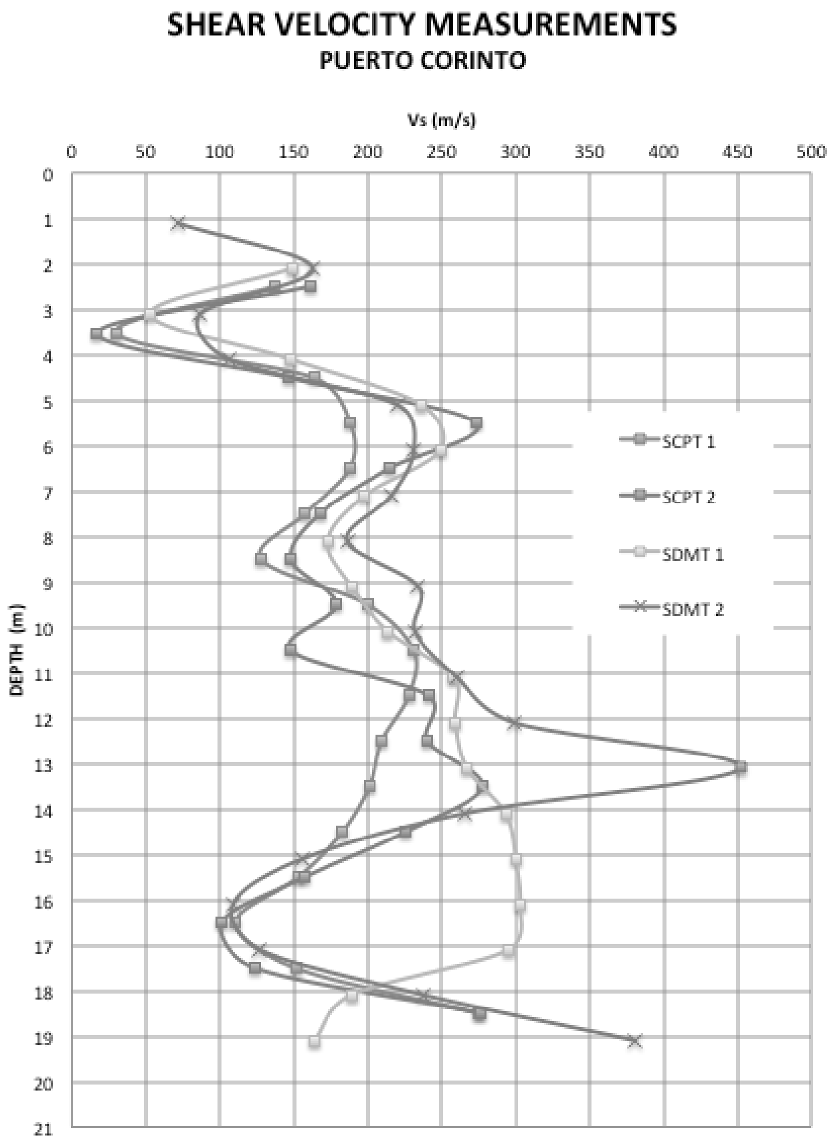

Fig. 16. Comparison of the shear velocity between two different sets of equipment.

As shown in the figure, the agreement between the different V_S profiles is rather good.

Conclusions

Cone Penetration Testing (CPT) as a technique for in-situ soil investigation, is a recognized and widespread method for efficiently performing of soil surveys. CPT is in the course of time continuously improved by the effective use of the latest state of the art. Recent developments, concerning the application of digital electronics inside the cone, offer a range of new features and benefits. The most prominant of these is the ability to easily extend the digital Icone by click-on modules to measure additional parameters. Any module is automatically recognized by the Icontrol data logger, creating a true plug & play system.

The Seismic module has proven to be an accurate and reliable measuring device, which can be used for different measuring techniques for soil analysis.

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Geogrids for improvement of serviceability of trafficked areas over soft clays and organic soils

Jacek Kawalec

Silesian University of Technology Department of Geotechnics and Roads, Gliwice, Poland
Tensar International, Cesky Tesin, Czech Republic

Abstract. Different geosynthetics are having different functions. one of recently discovered mechanisms, called stabilisation by geogrid, helps to understand differentiation in performance of various geogrids when used in trafficked areas over weak soils. Effect of non-reinforcing geogrids on stabilisation of unbound aggregate layer is difficult to measure within standard tests on geosynthetics. Paper contains basic classification of geogrids as well as basic explanation on difference in mechanisms of different geogrids used in aggregate layers. Some new research approaches are discussed with conclusion about future research directions.

Key Words: Geogrid, stabilisation of unbound aggregate, confinement, weak soils, trafficked areas,

Introduction

Any soft soil layer in ground closed to surface creates a problem for any kind of traffic. That could be impossibility to entry into area where CPT test should be carried out, that could be difficultness to transport aggregate or any constructive material to construction site, that could be short period of road service before maintenance works are required. Soft clays and organic soils due to their deformability under load do always require improvement, especially if load is repeatable. For any trafficked areas number of load cycles is typically higher than one, even for one site visit there is always pass in and pass out of the site typically on the same path.

Even single pass of any vehicle over weak ground could damage soil into depth of few tens of centimeters resulting in deep rut and making further site access impossible.

Traditional solutions to provide traffic over weak soils are typically assuming soil exchange to

some depth often resulting in additional soil damage during execution of such soil exchange. Another traditional approach is to place an aggregate layer over weak deposits. Thickness of such layer is depending on parameters of soil and on expected trafficking.
The most modern approach is a combination of mutually interacting aggregate and geogrid. Geogrids are known for over 30 years however many details on their interaction is still undiscovered, researches on geogrid interaction with aggregate, especially at low strain level, are ongoing at many laboratories worldwide.

Geogrids in light of other geosynthetics

Geosynthetics are used in construction for over half a century. With time many variations of them was introduced, however for large population of Engineers they fit into one very generic specification for „Geotextiles and relative products". Such understanding of geosythetic materials is often misleading end users due to fact that many of them has absolutely different properties than textiles as well as different dedicated application.
Within the enormity of different geosynthetics present on the market and available for engineering purposes geogrids are best for improvement of trafficked areas however even with geogrid family some of them do perform better than others, some of them do perform differently and some of them do not perform at all.
General ISO classification does not distinguishes different geogrids - current ISO definition describes geogrid as planar, polymer structure consisting of regular open network of integrally connected, tensile elements, which may be linked by extrusion, bonding or interlacing, whose openings are larger than the constituents. So within such definition it is easy to understand why geotextile, geomembrane or geonynthetic clay liner is covered by another definition. However for understanding of the performance of geogrid such definition is useless as completely different types of geogrids could fit into the picture, eg.:
- woven geogrids - flexible products manufactured by textile methods involving interlacing individual fibers of different polymer and strength
- welded geogrids - lat structure of individual fibers welded at junctions
- extruded geogrids - structure generated by extrusion of perpendicular strands of hot liquid polymer which effects in partial merge of strands in nodes. Final product has different perpendicular rib cross-sections.
- monolithic geogrids - structure generated in three step process (1) extrusion of polymer plate (2) punch of holes in extruded plate (3) multidirectional stretch in hot temperature to create ribs and aperture shapes.

All four type of products are commonly called geogrids however they performance in interaction with aggregate varies significantly.
There are also two types of mechanisms on how aggregate interacts with geogrid;
- load transfer by the tensioned membrane effect
- subgrade soil confinement

First mechanism was for many years understood as the main mechanism for road reinforcement. Now it's known that it's not the case as this mechanism requires rutting for geogrid mobilization so range of support to aggregate is seriously limited (Giroud 2009).
Second mechanism, underestimated for many years, is quite opposite to tensioned membrane. This mechanism do stabilize aggregate within stiff, hard-deformable aperture which prevents both vertical and lateral particle movement and as effect reduces deformation of aggregate layer under traffic. Thanks to confinement effect it is called geogrid stabilisation mechanism. It is important to not confuse the concept of stabilisation of the concept of reinforcement. Example of geogrid for stabilisation is presented on Figure 1.

Fig. 1. Hexagonal geogrid for the stabilisation of unbound granular layers

Stabilisation of aggregate under traffic load

The granular layers under any trafficked areas are during short period of time when the passing of the load occurs are in stressed in both vertical and horizontal directions. Vertical stresses are reduced with depth dependently on aggregate properties, strains from horizontal component of stress are distributed in all directions (see Figure 2) so it's critically important that geogrid acts uniform in all directions. It must be remembered that horizontal component of stress will case particles in the aggregate layer to move laterally and after single traffic pass thy will not recover fully as per condition before this pass. Any pass will add cumulative lateral movement that will case deformation of the layer under repeated traffic.

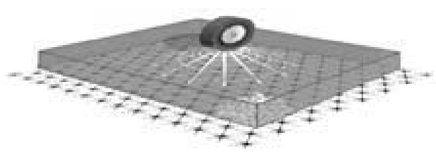
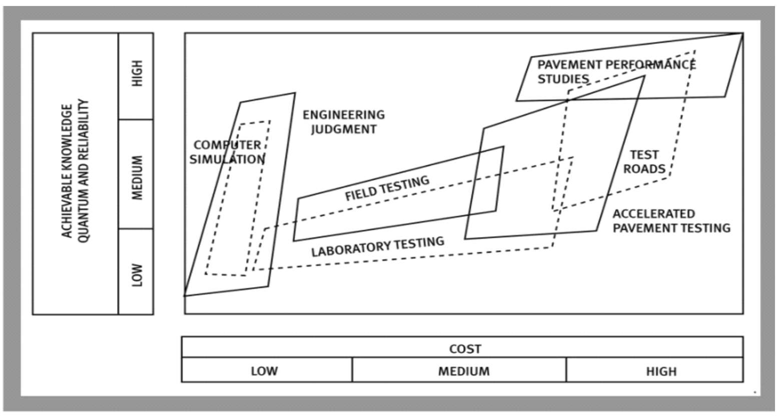


Fig. 2. Radial distribution of load from traffic (EOTA 2012)

There are many ways to test performance of construction products, very depending on costs. For pavements such tests could include (Hugo at al):

- computer simulation
- laboratory testing
- field testing
- pavement performance studies
- test roads
- accelerated pavement testing

Each approach gives different level of knowledge quantum and reliability (see Figure 3). With no means last two - three in situ types are the most expensive but preferable. It's hard to compare single laboratory test results with in situ full scale research on equivalency basis, however for full picture all listed types of test should be executed. In next paragraph examples of tests on geogrids for stabilisation is given with reference to Wayne at all, 2010 &2011)



Interrelationship between pavement engineering facets that collectively and individually contribute to knowledge (Hugo et al. 1991).

Fig. 3. A testing and trials matrix appertaining to the building of a knowledge base, (Hugo et al)

Laboratory tests

There are standard test protocols which allow one to evaluate the effect of geogrids on a given granular material. Figure 4 shows the results of triaxial cell testing by Wayne (Wayne 2011) where the resilient modulus of the aggregate material is derived following the AASHTO T307 procedure

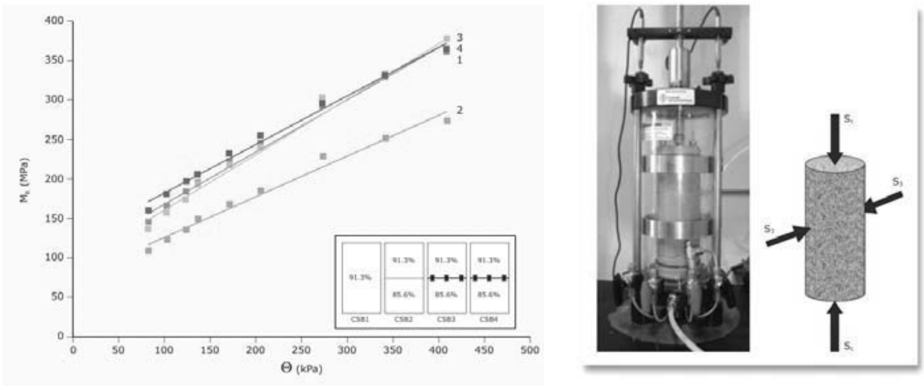


Fig. 4. Resilient Modulus (M_R) enhancement from AASHTO T 307

Four samples were prepared where:

- CSB1 was compacted to 91.3% of maximum dry density, ('high density' throughout)
- CSB2 was compacted so that the lower half of the cell was compacted to 85.6%, ('high density overlying 'moderate density')
- CSB3 introduced a geogrid to the interface of CSB2
- CSB4 repeated CSB3

The curves in Figure 4, where resilient modulus is plotted against the bulk stress applied to the cell, show that CSB1 has the expected 'stress dependency' with increase in resilient modulus with increasing bulk stress. CSB2 tends to represent what happens in practice where a second lift in a compacted fill is denser than its underlying layer. It can also represent a well compacted fill layer on a stiff subgrade. In either case, the curve drops to a lower range of resilient modulus values. The introduction of geogrid in CSB3, repeated in CSB4, shows an elevation of the range of resilient modulus values to those of a well compacted fill. For some workers in the field, this captures the benefit of geogrid use in pavements – to achieve predictability of performance as variability of the installation is largely overcome.

The curves can be used, for the particular fill type and geogrid tested, to indicate an enhanced modulus across a practical range of stress states.

Laboratory testing enables a unique combination of fill and geogrid to be tested, most likely with respect to a fill and geogrid proposed for a particular project, and the subsequent derivation of a useful parameter to represent enhanced modulus for use in such pavement analysis methods.

Field testing

Testing in the field is often unreliable but such testing is becoming increasingly used to 'prove' an installation prior to permitting the remainder of the pavement construction to be installed or trafficking of transportation tracks to commence. Full scale trafficking is usually too disruptive to the site operations and the contract's scheduling. An additional difficulty is that static plate loading tests sometimes fail to detect the geogrid benefit either accurately or at all. This can be explained by reference to Figure 5a and the use of cyclic plate loading. It can be seen that one or two load applications, rather like one or two wheel passes, does not depict the geogrid effect with any clarity. The response mechanism develops largely through subsequent loading.

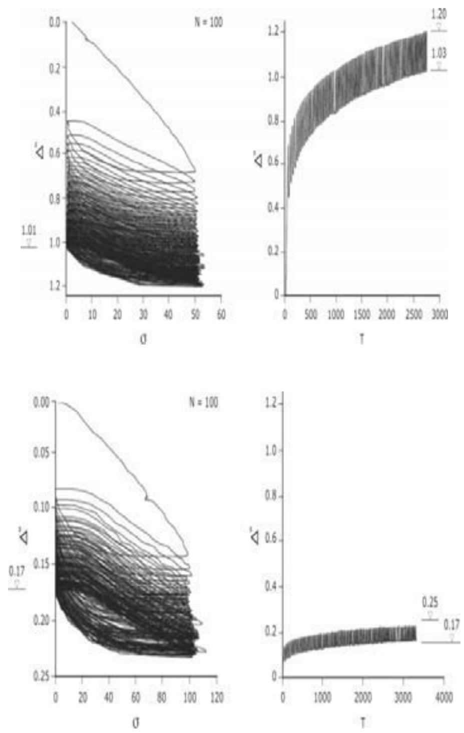


Fig. 5. Deflection against cyclic stress, (control to left and mechanically stabilised to right)

However, cyclic plate load testing provides promise for a rapid field test which evaluates an installation and is capable of identifying the geogrid benefit. Figure 5 shows comparative results of deflection against a cyclically applied stress through a plate. It is clear that, as both installations approach N=100 load cycles, the difference is seen by reference to the accumulated deflections at the zero stress part of the cycle, (1.01 inches v 0.17 inches).

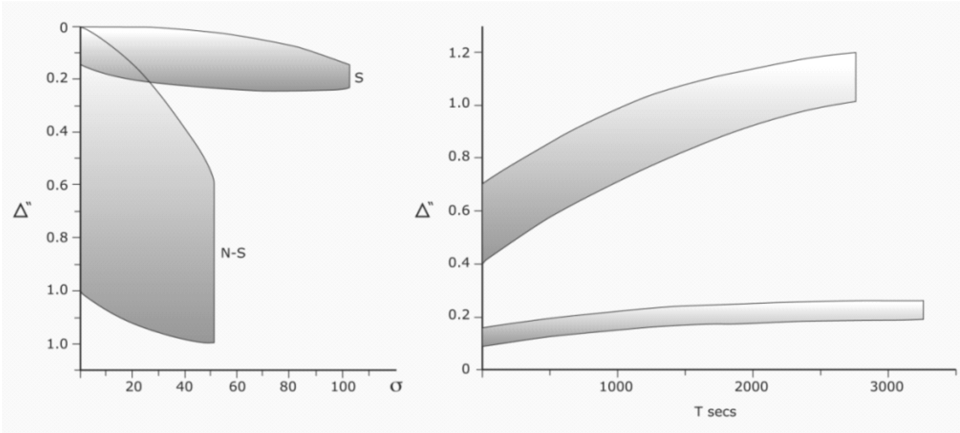


Fig. 6. Deflection against stress compared, (left), for stabilised (S) and non-stabilised (N-S)
And corresponding applied stress histories, (right)

Figure 5 is typical field data which is obtained to 'prove an installation' on a project. Figure 6 (left) combines the results for both non-stabilised (N-S) and stabilised (S) installations on common axes. It can be seen that the stabilised layer is stiffer, (load/deflection) and has an enhanced modulus. Figure 6 (right), shows the upper-bound and lower-bound limits of deflection over the time of the loading. The load is cycling between each limit 100 times. The same observation can be made that the stabilised installation is stiffer not only with respect to the smaller deflections of the stabilised installation but also from the narrowness of the limits of deflections under the applied stress.

Discussion on effect of geogrid for aggregate layer

From all these examples it appears that there is vast difference in performance of aggregate layer if properly stabilised by geogrid. What is not often remembered that supportive effect of geogrid in any measurement or trial test could be seen after number of cycles. As example given by Wayne 2010 the same aggregate layer performs much better after 20.000 cycles than layer without geogrid after 10.000 cycles of repeated load. Stabilised layer appears to reach a „stable" point on axial strain axis whilst non-stabilised failed before reaching 10.000 cycles

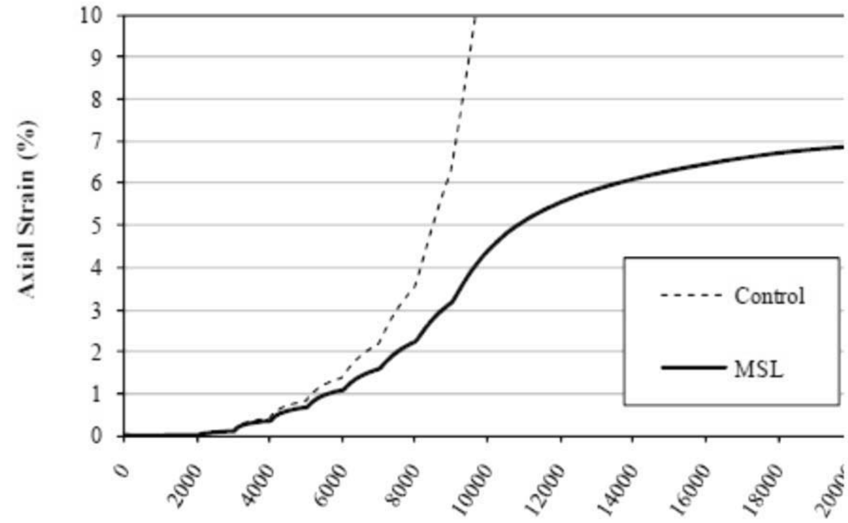


Fig. 7. Repeated load deformation tests for unbound control aggregate and mechanically stabilized layer incorporating one of TriAx geogrid (Wayne 2010)

Conclusions

Serviceability of any trafficked layers over weak soils when stabilised by geogrid could be improved significantly. Mechanism of stabilisation, in opposition to tensioned membrane effect, helps to reduce deformation. The effect of stabilisation however is difficult to identify in simple laboratory or site test with standard test procedure. There is a strong need to continue researches on full scale test sections and develop new methodology for testing effect of geogrid to aggregate layer. Examples given in a paper confirms however that with increased load cucles applied to granular layer differentiation is more clear. That could be seen as good direction for future research.

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