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Testing in Geotechnical Design

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ABSTRACT: Testing is an inherent and integral element of geotechnical design. This paper describes and discusses geotechnical testing in the design process from a consulting practitioner's perspective of the current state-of-practice. The role, objectives, types and interpretation of testing, limitations and recommended good practices are presented. Successful implementation of testing in design is demonstrated through examination of test data from a few case records.

1. INTRODUCTION

Testing is an inherent and integral part of the geotechnical design process. A primary objective of geotechnical testing is to assist in obtaining information for geotechnical site characterization and in developing the geotechnical model in terms of ground conditions (stratigraphy) and engineering parameters and properties. Ground is a complex engineering material with properties and parameters that are not linear, unique or constant. The key requirement for testing in geotechnical engineering is driven by the need to suitably and adequately characterize natural materials that are highly variable and subject to high degrees of uncertainty.

Geotechnical engineers must deal with the ground that exists at a given site. Unlike other civil engineering disciplines such as structural and materials engineering, the ground at a given site is generally not specified and manufactured to achieve desired engineering properties within a known degree of certainty or confidence. A primary role of testing in other engineering disciplines is generally to advance knowledge through research towards improved and more sophisticated methods for design and economic efficiencies.

In geotechnical engineering state-of-practice, the geotechnical engineer's foremost role and responsibility are to adequately characterize the site (i.e. assess what's there) to provide sufficient, reliable information of the ground conditions to facilitate good engineering decisions to be made during assessment, design and construction phases of a project that satisfies the client's/owner's needs, or as required by regulatory agencies. Analysis and design should proceed only after the stratigraphy and engineering properties have been appropriately defined. In the experience of the author, the tendency of many junior geotechnical engineers is that they want to jump right to analysis without sufficient thought and detail given to interrogation of all data and information to confirm that a representative geotechnical model has first been established. It takes discipline of thought and effective mentoring to avoid this urge of starting analysis and design too soon (i.e. before a reliable geotechnical model has been developed).

This paper describes and discusses geotechnical testing in the design process from a consulting practitioner's perspective of the current state-of-practice. The role, objectives, types and interpretation of testing, limitations and recommended good practices as part of the geotechnical design process are outlined. Successful implementation of testing in design is shown through the examination of test data from a few case records.

2. GEOTECHNICAL DESIGN PROCESS

2.1 General

The geotechnical design process is summarized schematically on Figure 1 and comprises the elements (architecture) of an integrated system that is described in greater detail below. A relevant code and guidelines of practice normally outline the design process and assist engineers in making appropriate design decisions, including testing requirements. From an interpretation of the results from site

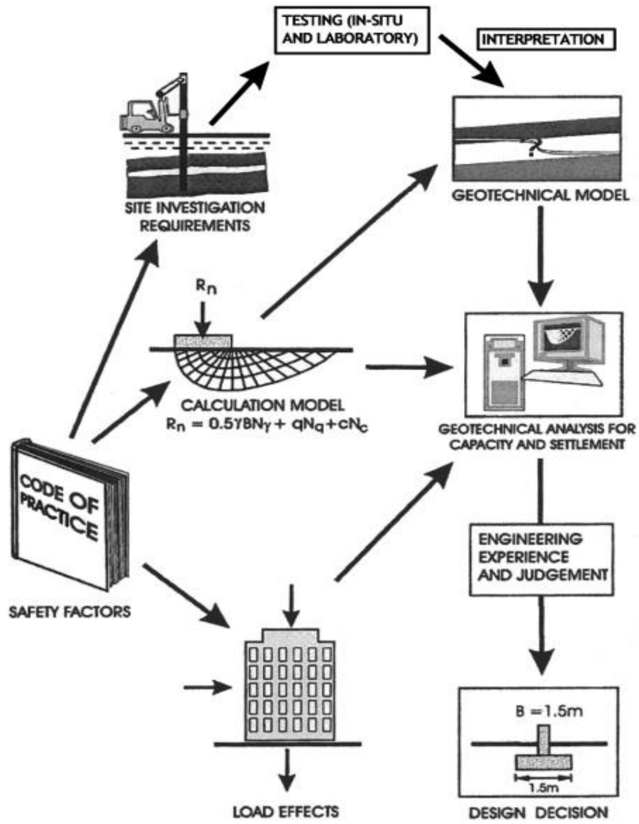

Figure 1: Geotechnical Design Process (after Becker 2006b)

characterization and testing programs, a geotechnical model is developed in terms of ground and groundwater conditions, and engineering properties. The analyses, calculation procedures and design equations for geotechnical resistance are based on relevant theoretical frameworks or on the basis of empirical correlations against a variety of laboratory, in-situ and field tests.

A sound design approach requires a thorough understanding of the key design issues, of the geological setting and geotechnical conditions, and of the interaction between them. In most cases, a good understanding of these factors is as important, if not more so, as the methods used for analysis and calculation. It is important to initially capture the essence of the problem, and then proceed with appropriate, simple testing and analysis followed by an increasing level of sophistication and complexity, as required or as the project demands. The results from the testing and analysis, when appropriately tempered or modified by engineering judgement and experience, are then used in the decision making process as to what constitutes the most appropriate designs.

Engineering judgement and experience play an integral role in geotechnical engineering analysis and design; they are vital for managing safety (risk) of geotechnical structures. There will always be a need for judgement, tempered by experience, to be applied to geotechnical testing, new technologies and tools. Uncertainties in loads, engineering properties, models, identification of potential failure modes (limit states) and geotechnical predictions all need to be considered collectively in achieving an adequate level of safety (reliability) in the design. The role of the geotechnical engineer through his or her judgement and experience, and that of others, in appreciating the complexities of geotechnical behaviour and recognizing the inherent limitations in geotechnical testing methods, models and theories is of considerable importance.

2.2 The Geotechnical Circle Concept

For any given project, geotechnical testing is an integral component of a successful design and in satisfying overall project requirements. Figure 2 shows the relationships that generally exist for projects.

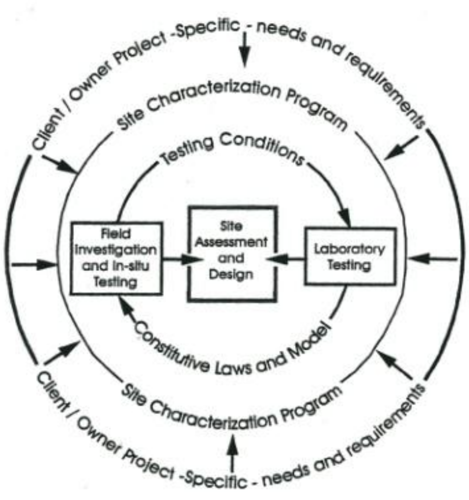

Figure 2: Geotechnical Circle Concept (from Becker 2001)

Testing and design lie at the kernel of the process, with specific needs and requirement of the project influencing the process. Important inter-relationship exist between field and laboratory testing, with linkage between them provided by constitutive relationships of the ground being investigated. Field investigation and in-situ testing results provide not only direct design information and parameters, but also provide data required to define appropriate test conditions in the laboratory. An example of this is the measurement of the coefficient of earth pressure at rest, K_o, in the field for use in specialized strength-deformation laboratory testing such as anisotropically consolidated triaxial tests. Laboratory test results, in turn, provide data to assist in proper evaluation and interpretation of in-situ test results.

The Geotechnical Circle concept was first used by the author and his colleagues in the early 1980's when characterizing Beaufort Sea clays towards demonstrating distinct characteristics found to exist in these clays (Jefferies et al. 1987[1], Becker et al. 2006a[2]). Similar concepts relating design, testing and analysis have been presented and described by others in the technical literature; notable examples include Peck (1980)[3], Burland (1987)[4], Randolph and House (2001)[5], Graham (2006)[6] and Mayne et al. (2009)[7].

The above process is iterative; as data are collected and interpreted, it is often necessary to make changes to the model and additional data collection may be required. During the assessment of data, one can't ignore data simply because they do not fit the general trend or expected behaviour. If it is concluded after thorough evaluation that the test was carried out properly, the test results must be kept. Good test data always tell something or suggest where next to look and investigate. Apparent anomalies often provide new insight and the stimulus or justification to further investigate and refine the geotechnical model towards obtaining a better understanding of the ground characteristics and expected behaviour. The discrepancy or anomaly may be indicative of special or unique behaviour. An example of this is the hypothesis and subsequent verification of unusually high K_o values in Beaufort Sea clays through the use of in-situ self-boring pessuremeter testing and laboratory testing as described by Jefferies et al. (1987)[1] and Becker et al. (2006a)[2].

2.3 Philosophy and Framework of Geotechnical Testing

Testing and results obtained should not be viewed as an entity in isolation from other components of the geotechnical design process. In most cases an appropriate combination of in-situ testing (including geophysics), sampled boreholes and laboratory tests on representative samples provide for suitable evaluation and definition of a representative geotechnical model. It needs to be acknowledged that the quality of test results obtained is commensurate with the extent of thought and effort applied in planning and performing in-situ and laboratory tests. Successful, cost-effective geotechnical characterization can only result from thorough and thoughtful planning and implementation. Lack of adequate and thoughtful planning, an incomplete understanding of fundamental soil or rock behaviour, and not having a consistent theoretical framework in which to assess, interpret and interrogate test results are probably the main factors that lead to wrong or misleading results. All of these conditions have consequential potential impact (i.e. detrimental effect) on the project in terms of inadequate or wrong design, undesirable surprises during construction, time delays, additional costs, long-term maintenance, and loss of licence and confidence by regulatory agencies. Additional information and discussion on these aspects are provided in authoritative references (e.g. British Standards Institution 1981[8], Site Investigation Steering Group 1993[9], , Hong Kong Government 1996[10], Becker 2001[11], CFEM 2006[12], FHWA 2002[13]).

Throughout his consulting practitioner career, the author has used critical state soil mechanics (CSSM) concepts and framework, including state parameter for sands (Been and Jefferies 1985[14], Jefferies and Been 2006[15]), yield envelope/effective stress path (YE/ESP) approach for clays (Folkes and Crooks 1985[16], Crooks et al. 1984[17], Becker et al. 1984[18]) and the relationship between average mobilized undrained shear strength (s_u) and preconsolidation pressure (σ'_p) as $s_u = 0.22\ \sigma'_p$ (Mesri 1975)[19]. The application of these considerations for assessing and understanding fundamental ground behaviour has been very useful and instrumental in the implementation and execution of hundreds of projects, both routine and complex.

The CSSM framework is used by many researchers throughout the world. Readers of this paper, in particular practitioners, are encouraged to adopt a critical state soil mechanics framework to achieve and enhance the development of meaningful testing programs and improved geotechnical design in their projects. However, although CSSM has many advantages and a proven track record, to date it is not commonly applied in practice in many parts of the world for a variety of reasons that are not discussed herein. Nevertheless, it is important that, for the benefit of projects, practitioners work within a suitable theoretical framework. It does not need to be CSSM based if the practitioner has not been so educated or trained and as such is not comfortable in using these concepts. It is more important for a practitioner to work within a mechanics framework compatible with their education, training and experience, than to say it must be CSSM based. The point is that a suitably consistent theoretical framework should always be applied by the practicing geotechnical engineer when testing is a key part of geotechnical design of a project. There needs to be a reason for each geotechnical test, whether routine or specialized, that is performed during execution of a project. Furthermore, the expected relationships between tests and their results need to be understood in order for the test program results to be fully interrogated and demonstrated to be consistent, reliable and representative. It is through this process that the credibility and reliability of the engineering properties and parameters can be established, and for reliable analysis and design to be achieved.

Quality is everyone's responsibility. The achievement of technical quality is realized when each person involved in the testing program (field, laboratory and engineering) do their part with appropriate care and scrutiny. Effective teamwork, experience, and a knowledge and awareness when a test result doesn't appear to "fit" are essential. The design and implementation of a successful testing program require a thorough knowledge of factors that control or significantly affect engineering properties and characteristics. The limitations of each type of test (field and laboratory) must also be known so that the most appropriate (in terms of technical merit and economic considerations) tests are carried out for a given project.

2.4 Geotechnical Testing Flowchart and Considerations

The testing process is summarized on Figure 3 which shows that the process is iterative in nature. A large part of the iterative process should take place during the field program, though iterations in the laboratory will assist in improved characterization of the ground and the geotechnical model. As data is obtained and interpreted, it is often necessary to make adjustments to the model and additional data collection is required.

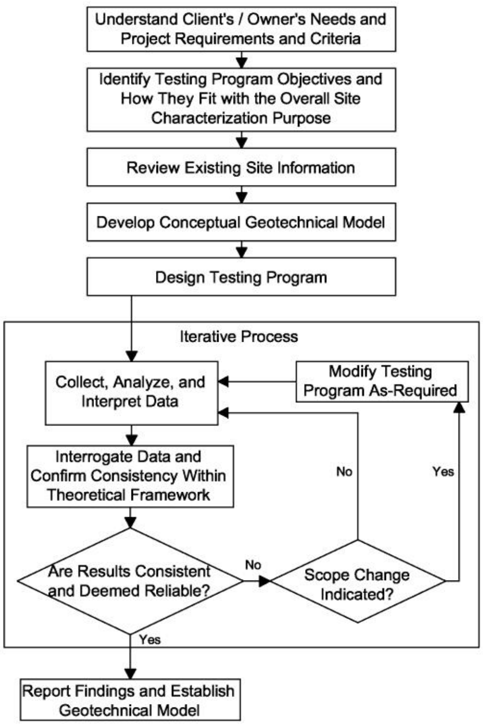

Figure 3: Geotechnical Testing Process Flowchart

Standard procedures and careful attention to detail must be carried out. For quality laboratory testing, good undisturbed representative samples must be used (i.e. the result is only as good as the sample and care taken in performing the test). In addition, due consideration must be given to assess the differences and scale effects between sample size and full scale field behaviour. These and other aspects are to be captured by the "characteristic" value that is cited in limit states and reliability based design codes. The characteristic value reflects the geotechnical engineer's best estimate of the representative or operational value of a geotechnical parameter/property that controls a specific limit state. The magnitude of the value selected needs to take into account all factors that potentially have influence on ground behaviour within the zone of influence (i.e. volume of ground) affected by the proposed structure or the applied loading from the structure, including the effects of construction and groundwater conditions during the service life of the proposed structures (Becker 2006b)[20].

Engineering judgement and experience assist in the selection of an appropriate characteristic value. For spread footings and raft foundations, the volume of affected ground (zone of influence) could be taken as the stress bulb associated with the footing or raft foundation. Factors that need to be considered when selecting a suitable characteristic value of a geotechnical property or parameter include: stress path imposed by the test relative to likely stress path imposed by the proposed structure; strain rate effects; anisotropy; fabric and structure (e.g. presence of varves); scale effects (e.g. intact strength of small sized samples or measured by in-situ tests relative to the spacing of factures of fissures in the ground); and other relevant factors.

It should be noted that a substantial amount of test data is not necessarily a good thing. Emphasis should be placed on quality not quantity. It is better to have only a few good data points than lots of data of poor and questionable quality. There is little benefit of having data from a large variety of tests for a given property (e.g. undrained shear strength, s_u) if the quality and relevance of each test type are different. For analysis and design, one should only report and use test results that are known to be of good quality and appropriate for the property or parameter to be determined. For example there is little merit in reporting field vane test or consolidated anisotropically undrained triaxial test results (higher quality) on the same graph as laboratory torvane or pocket penetrometer test results (known poorer quality).

Laboratory tests should always be carried out to simulate as close as possible or practical, the in-situ and imposed loading conditions. The types of test and effective stress path imposed on the specimen during testing should be representative of most likely field conditions to be imposed by the proposed development. For example in triaxial testing, the use of a back pressure approximately equal to the in-situ porewater pressure is recommended. Additionally if the value of K_o is anticipated to be significantly less than one, a few anisotropically consolidated tests should be performed in addition to standard isotropic consolidated tests to examine the influence of horizontal (radial) stress. In the experience of the author and as reported in technical literature, the effect of anisotropic consolidation has more influence on measured deformation parameters than on measured strength.

2.5 Use of Statistical Methods and Reliability Theory

Reliability and probabilistic theory constitute a key basis of limit states design based codes that have become mandatory for geotechnical aspects of foundations, retaining walls and other applications in many countries throughout the world (e.g. Eurocode, AASHTO Bridge Code, Canadian Highway Bridge Design Code and others). The use of these concepts is integral to the appropriate selection of characteristic value for geotechnical properties and design parameters (Becker 1996a[21] and b[22], Phoon et al. 2003[23] and Becker 2006b[20]).

The use of formal statistical techniques to analyze the results of laboratory and in-situ tests to determine realistic mean and standard deviation values (which define coefficient of variation) of geotechnical parameters is recommended. Geotechnical engineering practitioners need to better embrace these tools and concepts. Reliability analyses are being carried out much more frequently in geotechnical practice and will eventually become standard practice.

In-situ testing readily lends itself to statistical and probabilistic methods as large quantities of reliable data can be produced by the in-situ probes and data acquisition systems. The use of geostastiic and reliability based design for geotechnical engineering applications are discussed by Harr (1987)[24], Kulhawy (1992)[25] ,, Tang (1993)[26], Meyerhof (1995)[27], Fenton and Griffths (2008)[28] and in many other publications.

3. EMPIRICAL CORRELATIONS

Geotechnical engineering has a rich history in and is embodied with the use of empirical correlations. Numerous correlations between test type and engineering properties and parameters have been developed over the years. In particular, a comprehensive summary of commonly used correlations for laboratory and in-situ tests is provided by Kulhawy and Mayne (1990)[29].

3.1 Engineering Property Estimates and Comparison of Data

In many projects it is not feasible (from a budget/economic perspective) to measure all soil properties/parameters required. Therefore, estimates of engineering properties need to be made from other available data such as the results from laboratory and in-situ tests. Empirical correlations are also very useful and insightful towards establishing consistency and reliability of project specific test results. Although existing correlations are useful as a database against which project test data can be compared to assess consistency and validity, care must be taken to ensure that appropriate correlations are being used and that they capture the essence of behaviour. The source, extent and limitation of each correlation should be examined carefully so that extrapolation is not made beyond the intent of the original boundary conditions. Local or site-specific calibrations, where available, are preferred over the broad, generalized correlations. Many of the correlations reported in the literature have been developed from test data on relatively insensitive clays of low to moderate plasticity and on unaged quartz sands reconstituted in the laboratory. Extrapolation of these correlations to "special" soils (such as very soft and organic clays, sensitive clays, fissured clays, cemented soils, micaceous sands and collapsible soils) should be made with particular care and scrutiny because the existing correlations may not apply to these soil deposits. The same caution should be exercised in remote areas and where no prior experience has been gained.

Comparisons with existing correlations help in assessing if the ground at a given site is likely to display similar characteristics and behaviour as ground that has been more thoroughly investigated in other projects, or as documented in the technical literature. If a project specific test result does not conform with the existing well established correlations, it does not mean necessarily that it is wrong. The noted discrepancy may be indicative of special or distinct ground behaviour and performance. Such discrepancies should be resolved satisfactorily to confirm that potential risks to the project are within acceptable levels.

Particular care and scrutiny need to be placed on assessment of undrained shear strength s_u, which is not unique, but is a function of type of test and effective stress path induced within the clay during testing. Woo and Moh (1990)[30] provide an excellent summary of how undrained shear strength is related to test type and stress path considerations for clays in the Taipei Basin.

A wide variation in s_u can be measured. Which value is most appropriate will depend on the field loading conditions and field effective stress path to be imposed by the project. Undrained shear strength is also influenced by: rate effects (i.e. how fast test is carried out – generally, the faster the test, the higher the measured s_u; initial stress conditions (i.e. value of K_o and OCR); and anisotropy in terms of fabric and stress path. Table 1 summarizes the merit of commonly used tests for measuring undrained shear strength. The combined use of in-situ tests such as field vane and piezo-cone penetration tests (CPT) and laboratory tests such as oedometer and undrained triaxial tests can provide appropriate determination of s_u.

Table 1: Methods of Measuring Undrained Shear Strength

1. In-situ Vane Strength (FVT)	Reliable but generally limited to soft to firm clays.
2. In-Situ Piezo-Cone Penetration Testing (CPU)	Requires interpretation. Provides useful and reliable information.
3. In-Situ Pressuremeter (PMT)	Requires interpretation. Useful for stiff to hard clays.
4. Laboratory Unconfined Compression (UC)	Subject to sampling disturbance effects-underestimates strength.
5. Laboratory Unconsolidated Undrained (UU) Triaxial Compression	Some disturbance effects, likely underestimates strength.
6. Laboratory Consolidated Undrained Triaxial Compression (CIU/CAU) with Porewater Pressure Measurement	Most versatile of laboratory tests and provides useful information, but can overestimate strength.
7. Consolidation (Oedometer) (C)	$s_u = 0.22\ \sigma'_p$ is a reliable estimate of average mobilized strength

3.2 In-Situ Tests and Design Parameters

The results from in-situ tests are usually interpreted for the determination and assessment of the following engineering characteristics of soils:

- soil profiling and classification;
- strength parameters such as undrained shear strength of clays and effective friction angle of sands;
- deformation and stiffness characteristics such as modulus of deformation, shear modulus and maximum shear modulus (static and dynamic);
- initial states such as density, relative density, state parameter, in-situ horizontal stress (K_o), stress history (OCR or pre-consolidation pressure) and sensitivity;
- hydraulic characteristic parameters such as hydraulic conductivity and coefficient of consolidation; and,
- direct design applications such as foundation bearing capacity, pile capacities, ground improvement control and verification and liquefaction potential.

Most of the interpretation of in-situ test results for engineering properties and parameters, and for direct design applications, is based on a semi-empirical and correlation approach. The correlation and/or interpretation should be based on an appropriate theoretical framework and a physical appreciation and understanding of why the properties can be expected to be related. The correlations should capture the essence or first order controlling effects of the ground conditions and behaviour during in-situ testing. The influence of the time and duration of loading must also be taken into account when developing correlations. Time and hydraulic conductivity govern development of excess porewater pressure that, in turn, governs whether total stress (short-term, undrained) or effective stress (long-term, drained) parameters are appropriate. The type of test should consider rate of loading effects if it is to reliably represent field behaviour. If the soil response is essentially undrained, the test results should be used to infer or interpret undrained characteristics such as undrained shear strength. The results of piezo-cone penetration tests (CPT) tests should, therefore, be used to infer undrained characteristics in clays and drained parameters in sands.

Similar consideration should also be given to the degree of deformation or strain induced in the soil during testing. For example, an in-situ test which imposes very small deformation in the soil is best suited for interpretation of small strain response and in-situ stresses. In contrast, in-situ tests which impose large strain/deformation in the soil, such as the CPT and pressuremeter tests (PMT) should be used to correlate with strength and other large strain behaviour. Multi-stage or indirect correlations should be avoided wherever possible because they compound errors and often mask controlling factors.

Interpretation of the commonly used field vane shear test (FVT) provides an example that demonstrates the importance of considering key factors that influence strength. The FVT has been used as the reference test for assessing calibration factors for other in-situ tests such as the CPT. Traditional correlations involve soil index properties and FVT strength normalized with respect to vertical stresses. Undrained shear strength depends on the effective stress regime that controls strength on the mobilized failure surface. Approximately 90 % of the total resistance measured by a standard field vane is provided by the vertical circumscribed failure surface. Consequently FVT results are dominated by the strength mobilized on that vertical plane. In turn, it is the horizontal in-situ effective stress (i.e. K_o) and yield stress that is expected to predominately control FVT strength. Yet, to date, vane strength correlations have been considered primarily in terms of vertical stresses only.

Becker et al. (1988)[31] discusses an interpretation for the FVT within generalized state concepts characterized by both horizontal and vertical effective and yield stresses. The controlling influence of horizontal stresses was demonstrated using reliable data from 14 clay deposits, which exhibited both strain-softening and non-strain softening stress-strain characteristics from different countries, including back-analysis results of average mobilized strength during embankment failures. The controlling influence of K_o (horizontal stress) and OCR is indicated by the relationships shown on Figure 4 through a comparison of the scatter in the plotted data. The effect of K_o is embodied within the expression for mean ambient stress, I_o', as follows:

$$I_o' = (1 + 2K_0)\frac{\sigma_{vo}'}{3} \quad (1)$$

The data plotted in terms of I_o' (Figure 4a) display a well-defined narrow band compared to the data plotted on Figure 4b in terms of σ_{vo}' only, suggesting that first order (primary) effects have been better captured when horizontal stress is considered. The normalized strength (s_u/σ_{vo}') is represented well by the well-established expression

$$(s_u/\sigma_{vo}')_{oc} = (s_u/\sigma_{vo}')_{nc}\, OCR^m \quad (2)$$

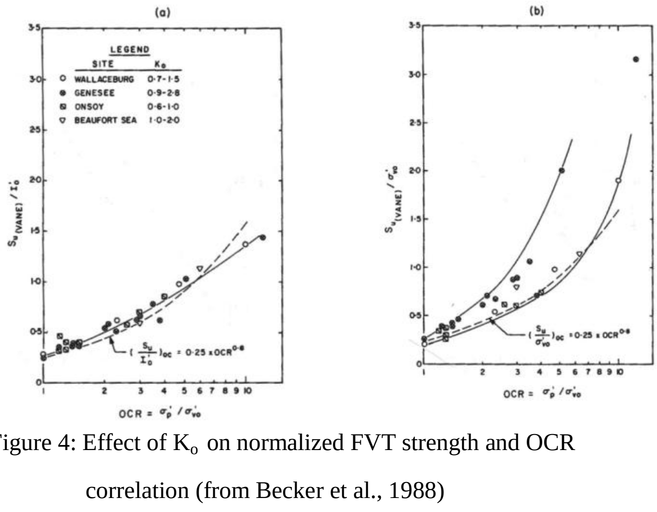

Figure 4: Effect of K_o on normalized FVT strength and OCR correlation (from Becker et al., 1988)

where nc and oc stand for normally consolidated and overconsolidated conditions, respectively, and the value of m is approximately 0.8 for a wide range of clays (Ladd and Foott, 1974[32]). A normalized strength of about 0.25 is demonstrated for normally consolidated condition, which is consistent with theoretical considerations and approximates the relationship $s_u = 0.22\ \sigma_p$'. Additional supporting evidence of the primary effect of horizontal stress on FVT strength is shown on Figure 5.

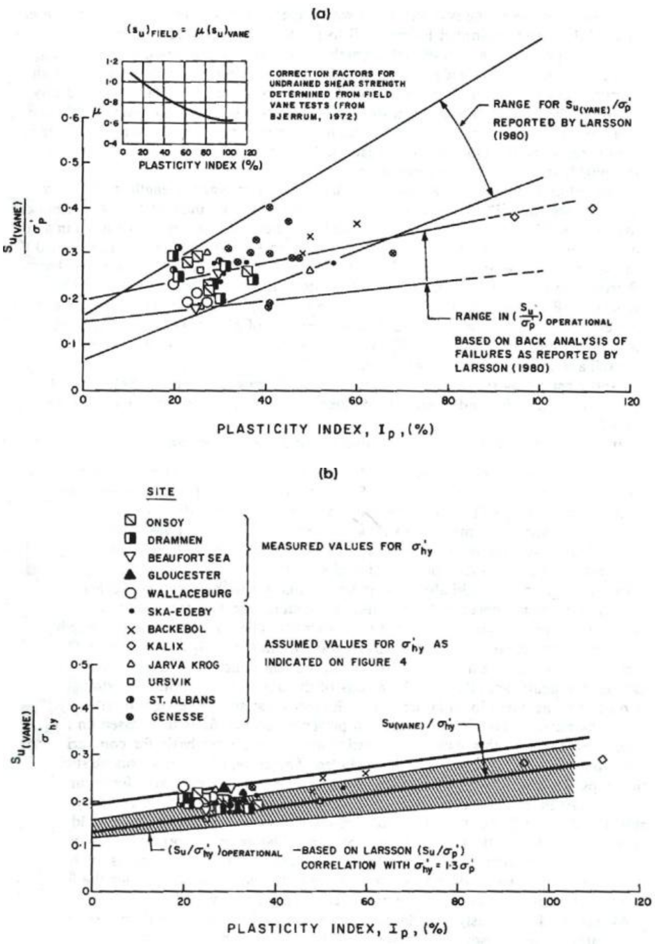

Figure 5: Comparison of normalized FVT strength with operational strength at failure during field testing (from Becker et al. 1988)

The traditional interpretation involving the classical Bjerrum correction factor is shown on Figure 5a where vane strength is normalized relative to σ_p' (vertical yield stress). The range in operational strength based on back-analysis of failures as reported by Larsson (1980)[33] is also shown. It is seen that many of the measured field vane strengths fall outside of the back-analyzed range in operational strength. However when the same data are plotted in terms of horizontal yield stress σ'_{hy}, a well-defined narrow band in the data exists and are within the back-analyzed range in operational strength. This striking comparison is difficult to deny and strongly indicates that horizontal stress regime governs measured field vane shear strength.

The above discussion demonstrates the importance that meaningful empirical correlations need to be based on a physical understanding of the test conditions and a suitable theoretical basis.

Numerous correlations between the results of in-situ tests and engineering properties and designs have been developed and are used in practice. The choice of the most suitable correlations depends on the property or design application being considered, the experience and background of the engineer, and local state-of-practice. The correlations available have been identified and discussed by several researchers and have been the subject of many national and international conferences and specialty symposia (e.g. Jamiolkowski et al. 1985[34], ASCE 1986[35], ISOPT 1988[36], Lunne et al. 1990[37] and 1997[38], ICE 1996[39] and ISC 1998[40]). Becker (2001)[11], FHWA (2002)[13], Ladd and DeGroot (2003)[41] and Mayne et al. (2009)[7] also provide a summary of the applicability and usefulness of the more common in-situ tests for the assessment of engineering properties and design applications.

3.3 Key Factors Controlling Soil Behaviour

The engineering properties and behaviour of clayey soils are largely controlled by void ratio, stress history and in-situ stresses (i.e. the "state" of the soil or the degree of overconsolidation (OCR)). Reliable definition of the stress history of a clay is important in quantifying and understanding its stress-strain, yield and compressibility behaviour. Definition of OCR requires reliable interpretation of the preconsolidation pressure. Furthermore, knowledge of the in-situ, geostatic effective stress state (σ_{vo}', σ_{ho}', (or K_o')) is important because it provides necessary information for: (i) appropriate interpretation of in-situ test data; (ii) reconsolidation of laboratory specimens for stress path testing and other laboratory strength-deformation tests; (iii) a starting point for analysis; and (iv) a general better understanding of fundamental soil behaviour.

The following sections summarize key findings obtained from the results of applied research project work performed on Beaufort Sea clay by the author and his colleagues. It is also demonstrated that the findings are applicable to many other natural clays.

3.3.1 Preconsolidation Pressure and In-Situ Stresses

Traditionally, preconsolidation pressure (σ'_p) is determined from the results of the standard oedometer test. Various methods for the interpretation of σ'_p from the void ratio (e) – logarithm of vertical effective pressure (log σ_v') relationship are available. In general, these methods are satisfactory for soils that exhibit an e-log σ_v' relationship with a well-defined break in the vicinity of σ'_p. However, for rounded e-log σ_v' curves, such as those typical for Beaufort Sea clays and other on-land clays encountered by the author and his colleagues, there is considerable uncertainty in the estimation of σ'_p using methods such as the classical Casagrande construction. Frequently σ'_p is reported in terms of a probable value with an associated range of possible values. Similarly, prediction or evaluation of clay behaviour is also subject to a range of interpretation, which is not particularly helpful in analysis and design. For these clays an alternate method for determining σ'_p was needed. This was achieved through the use of a work per unit volume (W) criterion to define the onset of yielding in the oedometer test (Becker et al. 1987)[42]. In this method, the cumulated work per unit volume is plotted against the vertical effective stress at the end of a given load increment using arithmetic scale axes. The incremental work done during a given load increment can be calculated as:

$$\Delta W = \left(\frac{\sigma'_{i+1} + \sigma'_i}{2} \right) \times \left(\varepsilon_{i+1} - \varepsilon_i \right) \tag{3}$$

Where σ'_{i+1} and σ'_i are the effective stresses at the end of the i+1 and i loading increments, respectively; ε_i+1 and ε_i are the natural strains at the end of the i+1 and i loading increment, respectively. It is noted that the above expression, when cumulated over the stress range of the test, corresponds essentially to the area beneath the stress-strain curve of the oedometer test. The Work method is therefore also referred to as the Strain Energy approach.

In the Work method, the work done per unit volume is used as a yield criterion to define, in an unambiguous manner, the change from small strain response to large strain response. Becker et al. (1987)[42] demonstrates that this vertical yield stress, σ_{vy}', is equivalent to the preconsolidation pressure σ_p'. It was also demonstrated that horizontal yield stresses could be interpreted from the results of test specimens that were trimmed at 90 degrees to the usual horizontal orientation (i.e. vertically). An interpretation to estimate current effective vertical and horizontal stresses (i.e. K_o) is also described by Becker et al. (1987)[42].

Although the Work approach was developed specifically to facilitate enhanced interpretation of σ_p' for Beaufort Sea clays, its use has been subsequently proven valid for many natural clays throughout the world. Many researchers and practitioners now use and advocate the use of the Work method as one of the most reliable methods for estimating σ_p' (C.C. Ladd 2002 personal communication, FHWA-IF-02-034, 2002[13], Ladd and DeGroot, 2003)[41]. In addition, Grozic et al. (2003)[43] compared systematically a variety of methods reported in the technical literature for determining preconsolidation pressure and concluded that the Work method was one of two methods that: (i) produced the most consistent results overall; (ii) was the most straightforward to interpret; and (iii) provided the best agreement with actual preconsoldation pressure values.

A typical e-log σ_v' curve obtained from a conventional oedometer (load increment ratio, LIR = 1) on a natural, almost normally consolidated Beaufort Sea clay specimen is presented in the upper plot on Figure 6. The probable value of σ_p' (Casagrande construction) is also shown together with a range of possible σ_p' values. The lower plot on Figure 6 shows the same oedometer test data but interpreted in Work-σ_v' space using the Work method. The data at low stress levels are shown at an expanded scale and indicate that a linear relationship is a good approximation. Similarly, the data at high stress levels define a second linear relationship. A distinct vertex exists between the two linear trends; this vertex is defined as σ_p' (σ_{vy}'). The mapping of oedometer data in Work- σ_v' space to define σ_p' has the added advantage (over conventional log σ_v' presentation) that stress is plotted on a arithmetic scale, which increases the level of interpretation precision.

3.3.2 OCR and Preconsolidation Pressure Correlated to CPT Tip Resistance

Many correlations between OCR and the results from the CPT have been developed and reported in the technical literature since about the mid-1980s. Figure 7 summarizes the correlation developed by the author in the early 1980s for the Beaufort Sea clays investigated. The correlation is based on a direct comparison of the results of OCR measured on quality samples in the oedometer test and the CPT normalized tip resistance measured at the same elevation of the oedometer sample. The preconsolidation pressure σ_p', and OCR were interpreted using the Work approach. OCR is plotted on a log scale to reflect that the state of clay is represented by OCR (i.e. stress difference between the preconsolidation pressure and in-situ current vertical effective stress (log σ_p' – log σ_{vo}')).

The tip resistance, q_t, used has been corrected for unequal area effects. A normalized tip resistance, q_t*, has been used where

$$q_t^{\bullet} = (q_t - I_o)/I_o' \tag{4}$$

I_o' = effective mean stress = $(1+2K_o)\sigma_{vo}'/3$

I_o = total mean stress

The above normalized tip resistance is similar to the conventional normalized tip resistance expressed in terms of vertical stress as:

$$q_t' = (q_t - \sigma_{vo})/\sigma_{vo}' \tag{5}$$

Mean effective and total stresses were used to incorporate the effect of K_o which, as described earlier, tend to be higher for some Beaufort Sea clays than for other clays for which correlations have been developed. The correlation presented on Figure 7 can be represented with reasonable accuracy as:

$$OCR = 0.4q^*_t = 0.4(q_t - I_o)/I_o' \tag{6}$$

The above equation when the value of K_o is taken into account can be expressed in terms of preconsolidation pressure σ_p', and vertical stress as approximately:

$$\sigma_p' = 0.30\ (q_t - \sigma_{vo}) \text{ for } K_o = 1.5 \tag{7}$$

$$\sigma_p' = 0.24\ (q_t - \sigma_{vo}) \text{ for } K_o = 2.0 \tag{8}$$

The above expressions for σ_p' are similar to the well established correlations generally expressed as:

$$\sigma_p' = f\ (q_t - \sigma_{vo}) \tag{9}$$

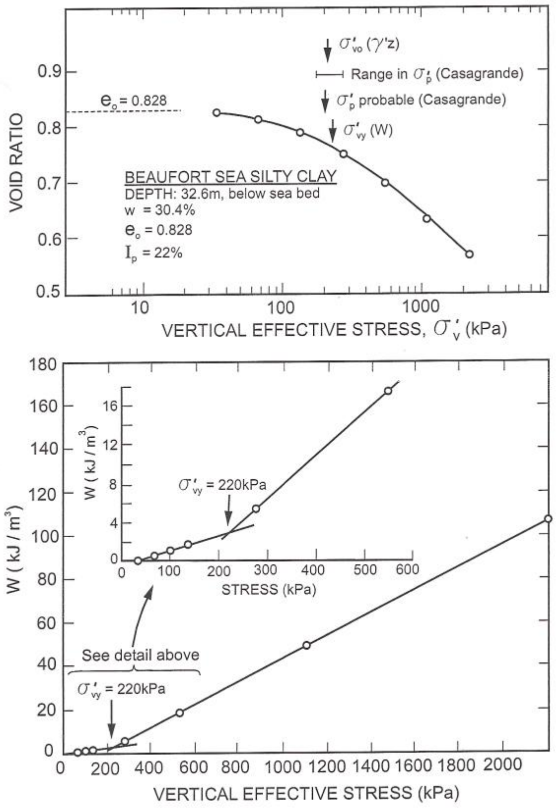

Figure 6: Determination of yield point using Work Method

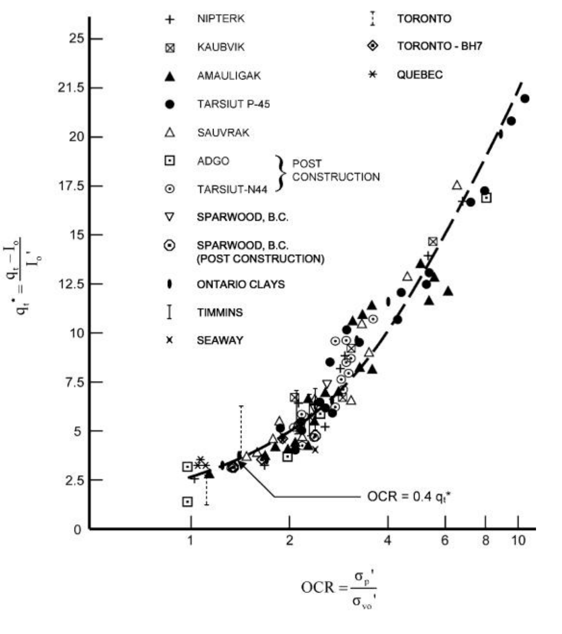

Figure 7: CPT Interpretation of OCR

The value of f reported in the technical literature ranges from 0.28 to 0.33 (Kulhawy and Mayne 1990[29], Mesri 2001[44], and Demers and Leroueil 2002)[45]. The correlation developed initially for Beaufort Sea clays appears to be consistent with other published correlations that consider only vertical stress. Nevertheless, it is opined that CPT interpretation in terms of mean stress (i.e. considering K_o) is more rational, especially for clays that may exhibit high K_o values. An approach to assess K_o from CPT porewater pressure results is described by Dittrich et al. (2001)[46].

Since the initial development of the relationship shown in Figure 7, the author and his colleagues have found that project data from many natural clays fit within the scatter band of Beaufort Sea clay data. Some of these data are also plotted on Figure 7. These project data are from oedometer tests performed on quality samples recovered from boreholes located close to the CPT investigated locations in order to obtain a high degree of direct comparison

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3.3.3 In-Situ Undrained Shear Strength

The in-situ undrained shear strength is usually assessed from CPT data by:

$$s_u = (q_t - \sigma_{vo})/N_k \qquad (10)$$

where q_t = tip resistance corrected for unequal tip area

σ_{vo} = total vertical (overburden) pressure

N_k = "cone factor"

The reference strength selected was that of the self-boring pressuremeter (SBP) and field vane tests (Becker et al. 2006a[2]). Figure 8 demonstrates the importance of K_o in the rational and consistent interpretation of CPT data. The values of OCR were based on the results of oedometer tests. The values of K_o were based on the results of SBP tests and supplemented by the results of laboratory oedometer tests on vertically trimmed specimens, according to the procedures described by Becker et al. (1987)[42] and 2006a[2]). An appropriate selection of N_k should take K_o into account. Moreover, this figure illustrates that within a geological unit, for which K_o is reasonably constant, N_k will also be practically constant.

As an alternative approach, the reference undrained strength was taken as $s_u = 0.22\ \sigma_p$'. It was found that the resulting values of N_k are very similar to those presented in Figure 8.

It is also noted that values of undrained shear strength can be obtained from the interpreted OCR value from Figure 7 using $s_u = 0.22\ \sigma_p$' or rewritten as $s_u = 0.22\ (\sigma_{vo}$'OCR).

4. GEOTECHNICAL TESTING APPLICATIONS

Testing is a key integral component of site characterization and is also essential in:

- design improvement and efficiency of design;
- design performance verification for geo-structural elements such as ground anchors, micro-piles, driven steel piles, bored cast-in-place piles, jet grouted columns and other ground improvement technologies and processes; and,
- Quality Assurance and Quality Control (QA/QC) for earthworks, foundations, ground improvement, retaining walls, etc.

The commonly performed tests in practice include: laboratory tests; in-situ tests; field performance tests; physical modelling tests; and tests as part of instrumentation and monitoring. Due to page limitations, this paper will discuss only three of the above primary test types.

4.1 In-Situ Testing

In-situ testing is a category of field testing corresponding to the cases where the ground is tested in place by instruments that are inserted in or penetrate the ground. In-situ tests are normally associated with tests for which a borehole either is unnecessary or is only an incidental part of the overall test procedure, required only to permit insertion of the testing tool or equipment. The in-situ tests that are most commonly used in practice are the Standard Penetration Test (SPT), field vane test (FVT), piezo-cone penetration test (CPT), pressurementer (PMT) and dilatometer (DMT). Other common field tests include plate bearing tests, pumping tests and other tests to determine hydraulic conductivity, and geophysical surveys. The applicability of these tests together with their advantages and disadvantages are summarized by Becker (2001)[11], CFEM (2006)[12] and Mayne et al. (2009)[7].

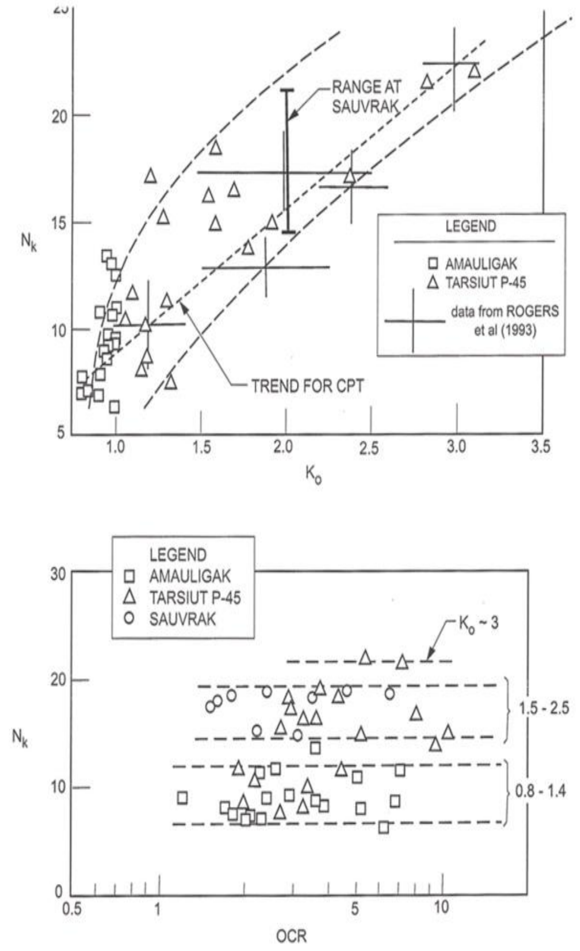

Figure 8: Relationships between N_k, K_o and OCR for Beaufort Sea clays (from Becker et al. 2006a)

The role of specialized, in-situ testing for site characterization and the research and development of in-situ techniques has received considerable attention over the last 25 years or so. The use of specialized in-situ testing in geotechnical engineering practice is rapidly gaining increased popularity. Improvements in apparatus, instrumentation, technique of deployment, data acquisition and analysis procedure have been significant. The rapid increase in the number, diversity and capability of in-situ tests has made it difficult for practicing engineers to keep abreast of specialized in-situ testing and to fully understand the benefits and limitation. Table 2 summarizes the primary advantages and disadvantages of in-situ testing. The description and applicability of wide range of geotechnical in-situ tests for soils have been summarized by numerous researchers in technical symposia and conferences, and reported in the technical literature.

4.2 Field Performance Tests

Field performance (full-scale) tests to achieve design and construction efficiency include:

- pile axial loading testing (e.g. Osterberg O-Cell, Statnamic and Pile Driving Analyzer (PDA));
- lateral load tests on piles;
- tensile and compression axial testing on micro-piles; and,
- tensile axial load tests on ground anchors.

Table 2: Summary of Advantages and Disadvantages of In-Situ Testing

Advantages	Disadvantages
• Tests are carried out in place without sampling disturbance which can cause detrimental effects and modifications to stresses, strains, drainage, fabric and particle arrangement. • Continuous profiles of stratigraphy, engineering properties and characteristics can be obtained. • Detection of planes of weakness and defects is more likely. • Methods are usually fast, repeatable, and produce large amounts of information and are cost effective. • Tests can be carried out in soils that are either impossible or difficult to sample without the use of expensive specialized methods. • A larger volume of soil may be tested than is normally practicable for laboratory testing. This may be more representative of the soil mass.	• Samples are not generally obtained; the soil tested cannot be positively identified. • Fundamental behaviour of soils during testing is not well understood. • Drainage conditions during testing are not well known. • Consistent, rational interpretation is often difficult and uncertain. • The stress path imposed during testing may bear no resemblance to the stress path induced by the full scale engineering structure. • Most push-in devices are not suitable for a wide range of ground conditions. • Some disturbance is imparted to the ground by the insertion or installation of the instrument. • There is usually no direct measurement of engineering properties.

The use of the above tests during design allows the designer to use higher geotechnical resistance factors as specified in limit states design based codes or lower global factors of safety in working (allowable) stress design because an increase degree of confidence, certainty and reliability is obtained through these tests. However, to take full advantage and increase design efficiency, the tests should be taken to failure so that the ultimate value of geotechnical resistance components such as skin (bond) resistance and end-bearing resistance are measured (e.g. an appropriately designed Osterberg O-Cell test). If failure is not induced during testing, the test results essentially only confirm, albeit to a higher degree of certainty (reliability), the design basis and anticipated performance of the structure. These tests may be viewed as proof tests. If the failure load is not measured (reached during the test), there is reduced opportunity to refine the design to its greatest potential by shortening the piles or making them a smaller size (width or diameter). The load test nevertheless has confirmed the design and a higher geotechnical resistance factor can be used in design which in itself significantly refines and increases efficiency of both design and construction.

In many codes including the National Building Code of Canada (2005)[47] and Canadian Highway Bridge Design Code (CHBDC 2000)[48], the specified geotechnical resistance factor for pile design increases substantially with the use of static pile load tests (e.g. increases from 0.4 to 0.6 – a 50 % increase, assuming that the estimated failure load equals that of the measured failure load). However, this does not directly translate into a 50 % saving on actual foundations because other factors of the overall structural system need to be taken into account such as redundancy, pile spacing and other aspects.

The reason for reduced design efficiency if a test is not taken to failure is explained by Figure 9 which shows a hypothetical axial load vs. Deformation relationship for load tests. Figure 9a corresponds to the case where the geotechnical engineer has estimated an ultimate axial resistance, R_n (estimated), and no load test is performed. The initial portion of the plot is in solid line to represent the higher confidence in this portion of the overall prediction. The design resistance is $0.4R_n$ as per code. The remainder of the plot is represented by dashed lines to reflect uncertainty (lower degree of confidence). Figure 9b shows the results of a load test taken to achieve failure. The design resistance now becomes $0.6R_n$ (measured).

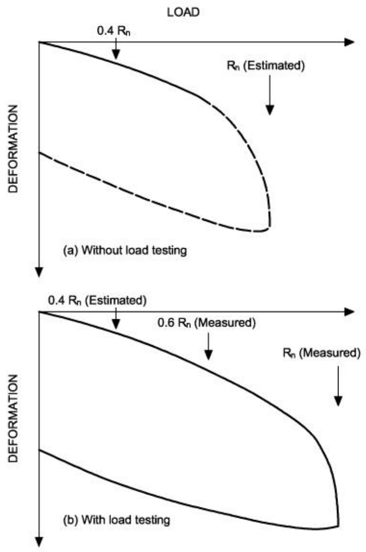

Figure 9: Load-Deformation Response of a Pile

The hypothetical results shown on Figure 9 are common in practice in that the measured ultimate resistance (failure) is often higher than predicted. Significant design efficiency is obtained because the failure load (R_n) and the geotechnical resistance factor are higher, and thus the factored geotechnical resistance is substantially higher than the no load test case. However, it should be noted that sometimes the measured failure load is less than estimated, but design still remains refined because a higher geotechnical resistance factor can be used.

If the load test has not been carried out to failure, say only to twice the design resistance as is common in practice, the revised design resistance would become $0.6R_n$ (estimated), which is significantly less then $0.6R_n$ (measured).

Without the benefit of a load test, the reliability of the design and foundation performance for the entire area (footprint) of proposed development is a function of how close the design resistance ($0.4R_n$ (estimated)) lies relative to the actual ultimate geotechnical resistance (R_n (measured)). If the design resistance lies far away from R_n (measured), it is likely that the actual reliability for performance would be higher than the relevant code specified target reliability, even considering ground variability and variability in ultimate geotechnical resistance. However, if the design resistance happens to lie much closer to the measured failure load (as would happen if R_n (estimated) was significantly overestimated), it is possible that actual reliability may be less than the target value, and some of the piles may not perform as well as anticipated.

The above discussion is intended to provide insight and awareness that even with testing a degree of uncertainty remains. It is for these reasons and to truly refine design to its greatest potential that load tests should be taken to failure as part of the design process. It is recognized that this is not always feasible and in these cases proof tests should be carried out as part of QA/QC during construction. In the author's (and most likely other practitioners) experience with the larger projects, the cost benefits associated with load testing significantly offset the costs of testing, which can be substantial for full scale testing. When presenting the justification for such testing, it is useful to refer to the tests as investments – not costs because clients and owners will then better understand why the tests should be performed. In practice the use of full scale load tests is usually an economic decision and is normally undertaken when a large number of piles are to be constructed, otherwise there is usually no economic advantage in conducting the tests.

Full-scale pile load tests including Osterberg O-Cell or Statnamic testing programs have been implemented in several large projects undertaken by the author, including the Confederation Bridge (Becker et al. 1998)[49] and other transportation projects (e.g. Thomson et al. 2007[50], Skinner et al. 2008[51] and Leew et al. 2008[52]). These case records and many others published in the technical literature describe the tests and results obtained, and discuss key design interactions.

4.3 Physical Modelling Tests

Physical modelling testing in geotechnical engineering is used extensively despite of its high investment costs for experimental facilities and the current decline in computing costs associated with high end capability numerical and analytical analysis (Randolph and House 2001)[5]. Physical modelling includes testing of load-response relationships of reduced scale geotechnical elements such as spread footings and piles at l-g (gravity) conditions (i.e. low stresses). Centrifuge modelling can simulate in excess of 100 times gravity conditions and produce more realistic (higher) stress fields associated with full-scale field conditions.

Physical modelling is used because sometimes the complex nature of natural soils and rock can't be captured sufficiently by analytical and numerical methods alone. Physical modelling is undertaken usually to assist in the investigation of three-dimensional effects, complex construction processes (e.g. soil-structure interaction effects, performance of piles, anchors, retaining walls, pipelines, downdrag on piles, improved stability due to vertical elements (e.g. piles, micropiles, etc.)), cyclic and dynamic loading effects (e.g. earthquakes, offshore foundation, etc.), creep, non-linear effects and other factors.

Testing at reduced, rather than full, scale is generally governed by cost and budgetary constraints. It is important that, prior to undertaking physical modelling, a clear understanding of a theoretical framework and what aspect of soil response may be better captured through physical testing rather than numerical analysis alone. Physical testing also assists in calibrating numerical and analytical models.

The Geotechnical Circle (or Burland's Geotechnical Triangle[4]) is also applicable to physical modelling as discussed by Randolph and House (2001)[5]. A key consideration of the validity of reduced scale physical testing is "scale effects" which have been the source of much discussion and debate in the geotechnical engineering community. However with caution and care, scale effects can be properly handled.

Although physical modelling/testing is frequently used in research, it is much less used in engineering practice, except for very high importance, significant projects, or those of an applied research basis. Examples of the latter case include large scale prototype tests of pipes subject to frost heave forces that have been and are currently being undertaken by Golder Associates in Calgary, Alberta, Canada for the pipeline industry (Liu et al., 2004)[53]. Pipes with diameters as much as 400 mm are tested in a steel box that contains 2 m^3 of soil (0.9 m x 0.9 m x 2.4 m) and frame set-up in a controlled low temperature (as low as -20°C) cold room. The results from these physical tests are used together with numerical modelling to gain a better understanding of this practical and relevant soil-structure interaction process. In addition, recently the author and his colleagues have tested miniature steel piles in similar cold rooms to investigate the effect of both static and cyclic loading on pile axial performance with different soil-grout-pile interfaces. This work was carried out to assist in the design of petroleum production facilities in the Canadian Arctic. A technical paper on this work is under preparation.

Another example of physical testing undertaken to advance state-of-the-art and state-of-practice is the large scale (1.4 m diameter by 1.0 m high) cone (CPT) calibration chamber that was developed and built by Golder Associates in Calgary, Alberta. This equipment, which provides radial, vertical and backpressure stress control, and the use of a standard 60 degree piezo-cone penetrometer, was essential to the development of state parameter for sands (Been and Jefferies 1985)[14] and the interpretation of state parameter from CPT tip resistance (Been et al. 1986[54] and 1987a[55], and Jefferies and Been 2006[15]). The cone calibration chamber and results of testing is described in Been et al. (1987b[56]). State parameter is widely used today in both research and practice, and is viewed in the geotechnical community as an important development in the enhanced understanding of fundamental sand behaviour (ICE 2008[57]).

4.3.1 Centrifuge Modelling

Centrifuge modelling is also not carried out routinely in geotechnical practice though its consideration and use have received increasing attention over the past 15 years or so. The author and his colleagues used centrifuge modelling and experienced its significant attributes and advantages during the analysis and design of circular and oval-shaped ring foundations for the approximately 13 km long Confederation Bridge that spans Northunberland Strait to connect the provinces of Prince Edward Island and New Brunswick in Atlantic Canada (Becker et al. 1998[49]). The bridge at that time was (and may still be) the longest, continuous, marine span bridge over ice-covered water in the world. It was a design-build-operate and transfer project. Pre-cast concrete 22 m diameter ring foundation units were placed in water as deep as 35 m and the bridge superstructure is as high as 60 m above water level. The high eccentric loads and applied moments, imposed primarily by ice loads, coupled with the complex and variable soft sedimentary bedrock posed significant engineering challenges. The integration of test results from analytical/numerical methods and physical modelling (centrifuge) tests were used to investigate potential mechanisms of foundation failure to examine how these mechanisms vary with changes in foundation and loading conditions, and to refine the foundation design analysis and design. The centrifuge model tests were carried out by C-CORE at Memorial University of Newfoundland. Models were built at a 1/160 scale to simulate specific pier footings and foundation rock stratigraphy and were tested at 160g acceleration. Details of the centrifuge model preparation, test procedures, and test results are summarized by Kosar et al. (1996[58], 1997[59]).

The centrifuge test results provided an improved understanding of failure mechanisms in terms of failure surface profile and its depth, and on load inclination and eccentricity effects. The results showed that, under high eccentricities and associated reduced bearing area, the three-dimensional characteristics and influence of a loaded ring footing diminished to the extent that, in some cases, simple two-dimensional limit equilibrium analysis of strip footings was adequate to predict bearing resistance. In addition, for failure mechanisms associated with high eccentricities, it was not appropriate to apply an inclination factor correction that is based on a failure mechanism associated with vertical loading only. The information gained from the centrifuge tests was used to refine the design methodology for determining bearing resistance of ring footings, as described by Becker et al. (1998[49]).

5. CONCLUDING REMARKS

Testing is an inherent and integral part of the geotechnical design process; it should not be viewed as an entity in isolation from analysis and design. The Geotechnical Circle concept as described in this paper is a useful framework that provides linkage between the various components of geotechnical design and captures its iterative nature. Adequate planning and working within a consistent theoretical framework, such as critical state soil mechanics and state concepts, enhance the development of meaningful testing programs and the assessment and interrogation of test results. The design and implementation of a successful testing program require a thorough understanding and knowledge of the factors that control or significantly affect engineering properties. The limitations of each test type must also be known.

The basis and limitations of the numerous empirical correlations that exist in the technical literature should also be understood. Although three correlations can be very useful and insightful towards establishing consistency and reliability of project specific data, care must be taken as described in this paper to ensure that appropriate correlations are used and that they capture the essence of behaviour.

Correlations used or developed should be based on suitable theoretical considerations and a physical appreciation and understanding of expected behaviour and why the properties can be expected to be related. Correlations should be based on and capture first order controlling effects as described in the examples and case records provided in this paper. In addition, the use of formal statistical methods and reliability theory to analyse, interrogate and integrate test results is recommended.

Geotechnical testing is also a key integral part of achieving design improvement, design efficiencies and verifying performance of geotechnical structures and geo-structural components. It is recommended that performance tests such as pile load tests be taken to failure (instead of proof tests) to maximize design refinements and construction efficiencies. If the ultimate geotechnical resistance is not measured there is reduced opportunity to refine design to its greatest potential as described in this paper. In many cases, the cost benefits associated with full scale load testing significantly offset the cost of testing. The tests should be described to clients in terms of an investment.

Physical modelling testing, such as reduced scale tests under normal gravity conditions and centrifuge tests, also provides insightful and beneficial results to projects if their limitations are understood and the results used to complement results from analytical and numerical modelling. The results from physical model tests when used together with numerical modelling provide an improved understanding of soil-structure interaction and failure mechanisms. They also assist in refining analysis and design.

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