INVESTIGATION AND DESIGN OF SMALL EMBANKMENTS ON SOFT CLAY IN NORTHERN ONTARIO

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Soft, varved lacustrine clays exist throughout much of Northern Ontario, having been deposited in proglacial and postglacial lakes that existed 18,000 to 6,000 years B.P., during the retreat of the Wisconsin Ice Sheet. These soft soil deposits present problems for stability of small embankments such as roadways, small water dams and mine waste disposal embankments. Because such structures usually do not warrant extensive field and laboratory investigations for design, site investigations are usually carried out by conventional methods using borehole sampling, field vane shear tests, cone penetration tests and laboratory index tests. This paper discusses the uses of these techniques to characterize soft soil sites and selection of parameters for stability analysis. Case history examples are presented.

INTRODUCTION

Soft, freshwater varved clays were deposited in glacial Lakes Agassiz and Barlow/Ojibway which covered much of Northern Ontario between 18,000 and 6,000 years B.P. during the retreat of the Wisconsin Ice Sheet (Quigley, 1980). The distinctive microstratigraphy of varved clays plays a key role in the geotechnical behaviour of these deposits. Understanding of the glaciolacustrine depositional environment and mechanisms for varved clays is therefore key to understanding their geotechnical properties and behaviour.

Typically, the varved clays form a mantle within lowlands in Northern Ontario, and overlie bedrock or more competent soils such as glacial moraines or fluvial-glacial sands and gravels. Initially deposited in a very soft state under water, the clays have been subsequently consolidated by lowering of the water table, weathering at the ground surface, and ageing effects. Among these, the greatest factor is the desiccation and freeze-thaw consolidation of the soils which results in a lightly to heavily over-consolidated surface crust up to 6 m in thickness. At high elevations where the clays are thin, this surface crust may extend entirely through the clay and, under such conditions, the clays can form an acceptable foundation for roadway embankments, light building foundations and mine waste dumps. At lower elevations and in valley bottoms, the varved clays can be over 20 m thick and the surface crust is underlain by thick sequences of soft to stiff clays which are weak and compressible. Because such low areas often form the best sites for small embankment dams and mine waste disposal facilities, these soft soils present a recurring design problem.

Figure 1 presents a typical design section for a small embankment founded on a thick deposit of varved clays. The main soil parameters for the stability analysis of such

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embankments are the shear strength of the embankment fills, the over-consolidated clay crust and the underlying soft clays. Because the clay crust forms a raft of stiffer soil overlying the softer clays, and the selection of the appropriate strength for the clay crust is considered by the authors to be a critical factor in analyzing the stability of small embankments. Where the soft clays are too soft to support embankment construction in one stage, staging construction over several years may be necessary to permit consolidation and strength gain in the soft clay. In this case, the rate of consolidation and therefore of strength gain is theoretically estimated using the coefficient of consolidation, C_v.

As will be discussed in this paper, the authors routinely adopt the field vane shear test and piezocone penetration tests (CPTU) as the most cost-effective methods of obtaining the undrained shear strength of soft clays for design of small embankments up to about 8 m in height. Consolidation parameters are obtained from laboratory oedometer tests on Shelby tube samples. These strength and consolidation parameters are then compared with empirical relationships developed using index properties and data from other sites.

This paper briefly describes the deposition and general characteristics of varved clays in northern Ontario. The paper then discusses site investigation procedures that are used to obtain parameters for stability and settlement analysis of small embankments. Case histories illustrating the authors' recommended approach for site investigation and stability analysis of small embankments are presented.

GENERAL CHARACTERISTICS OF VARVED CLAYS

Depositional History and Environment

A comprehensive review of the geology, mineralogy, and geochemistry of the lacustrine clays in central Canada is given by Quigley (1980). Proglacial and postglacial lakes covered much of central Canada and the Prairie Provinces between 18,000 to 6,000 years B.P., during the retreat of the Wisconsin Ice Sheet. Freshwater and marine clays, which exhibit different behaviour, were deposited during this retreat. Areas covered by freshwater glacial lakes and saline seas are shown on Figure 2 (Quigley, 1980). Varved clays were deposited in freshwater lakes that formed in lowlands at the ice front, while sensitive, flocculated marine clays were deposited in coastal areas where saline water encroached upon the lowlands.

With the retreat of the ice sheet, the continental crust has undergone an estimated 800 m of isostatic uplift since the beginning of the retreat 18,000 years B.P. (Andrews and Peltier, 1976; Andrews, 1968). About 200 m of this uplift is considered to have taken place since deglaciation, about 8,000 years B.P.

Several major water bodies that existed during various phases of the Wisconsin Ice Sheet retreat have been identified (Quigley, 1980). The most significant of these with respect to the lacustrine clays of Northern Ontario are freshwater Lakes Agassiz and Barlow-Ojibway. The geology and mineralogy of the Lake Agassiz clays have been studied by Elson (1967), Teller (1976), and Baracos (1977). Lake Agassiz extended east from Lake Winnipeg to Lake Nipigon, and south from Hudson Bay to 350 km into the United States (Quigley, 1980). Lake Agassiz drained into Lake Superior. During glacial advances and retreats, the Lake Superior outlets were periodically opened and blocked, resulting in large lake level fluctuations in Lake Agassiz. These fluctuations resulted in drying intervals, forming desiccated crusts and erosional surfaces within the deposits. The Lake Agassiz clays in Ontario are derived from rock flour from the Canadian Shield (Quigley, 1980), supplemented by carbonates and shales from the Hudson Bay lowlands. The Lake Agassiz clays tend to be rich in montmorillonites.

Lake Barlow-Ojibway existed as a proglacial lake south of James Bay between 10,000 and 8,000 years B.P. (Quigley, 1980), and during its latter stages may have been connected with Lake Agassiz (Elson, 1967). The Lake Barlow-Ojibway clays indicate source rock flour from the Canadian Shield rich in quartz, feldspar, illite, chlorite, and smectite (Soderman and Quigley, 1965; Ballivy et al., 1971).

Depositional Mechanisms

Varved clays consist of layered, fine-grained sediments deposited in freshwater lakes. A single varve represents one year of deposition, with a couplet of summer silt and fine sand (light in colour), and winter clay (dark in colour). In proglacial lakes, turbidity currents represent the controlling depositional mechanism. During the summer, the streams feeding the lakes are sediment rich, and so have a higher density than the lake water. As a result, a current of the sediment-laden water flows along the bottom of the lake, depositing the silt-rich summer layers, which are typically multi-laminated due to the intermittent nature of the bottom turbidity flows. These currents travel long distances, resulting in varves continuous for significant distances. In the winter the streams contain little sediment, resulting in a lower density, causing the sediment-deficient inflow to overflow the lake water. As a result, only slow-settling clay particles are deposited during the winter, forming the dark layers.

The depositional mechanics are different in postglacial lakes. Because these lakes are located at some distance from the retreating ice front, the sediment load of the streams feeding them is significantly lower than for streams feeding proglacial lakes. As a result, the streams enter the lake as overflows and/or interflows (Quigley, 1980). Deposition of sediments occurs year-round, with silt and fine sand settling during the summer, and clay particles settling during the winter. Varves are less continuous in postglacial lakes than in proglacial lakes because the overflow/interflow currents are less efficient at transporting sediments than the turbidity currents that characterize deposition in proglacial lakes (Quigley, 1980).

Microstratigraphic Control of Geotechnical Properties of Varved Clays

As discussed above, the distinctive structure of varved clays is controlled by the mechanics of their deposition. The typical structure of thick-layer varves from glaciolacustrine clay from New Liskeard, Ontario is shown on Figure 3 (Chan and Kenney, 1973; Quigley, 1980). Significant aspects of this microstratigraphy are:

- the silt-rich summer layer fines upwards through a transition zone, grading into the clay-rich winter layer;

- the transition and fine layers represent settlement of suspended solids in autumn and winter;

- the moisture content of the flocculated clay varves is much higher than that of the silt/sand varves, due largely to the open flocculated structure; and

- the natural moisture content of the clay varves is very near the liquid limit.

The microstratigraphy of varved clays clearly controls their geotechnical properties, and hence their geotechnical behaviour. The key properties of varved clays therefore vary significantly on a scale that cannot be easily assessed in field and laboratory testing.

Figure 4 presents a plasticity chart showing typical ranges for light summer varves and dark winter varves (Milligan et al., 1962). The dark winter layers are classified as clay of high plasticity, while the light summer layers are classified within the range of silt of low plasticity to clay of intermediate plasticity. If during a laboratory investigation of varved clays no attempt is made to differentiate the summer and winter varves when taking samples for determination of moisture contents and Atterberg limits, then results will tend to reflect the average of the two extremes. Accordingly, such results should be used with caution, and with full understanding of their limitations.

The liquidity index of varved clays, which relates the natural moisture content to the liquid limit, is used for several empirical correlations with other geotechnical properties. Figure 5 shows ranges of liquidity index values for summer and winter varves. In many cases, the liquidity index of the summer varves exceeds that of the winter varves, but generally they are not disimilar with values ranging between 1 and 2 below the surface crust. Figure 6 illustrates an empirical correlation established between liquidity index and the undrained shear strength (S_u) ratio (S_u/σ_p') for normally consolidated clays. The correlation incorporates data by Bjerrum and Simons (1960), and field data from the authors' files. For varved clays that are truly normally consolidated, the strength ratio can be as low as 0.13. Higher ratios, usually between 0.2 and 0.3, are more generally found because of apparent over-consolidation from ageing and slight cementation.

Figure 7 presents typical site investigation data obtained in varved clays for a site in northwestern Quebec. The wide ranges of moisture content, liquidity index and field vane shear strength probably reflect the variations in properties between summer and winter varves. The lower moisture contents and liquidity index values, and the higher field vane undrained shear strength values obtained at shallow depth are indicative of a desiccated crust. The lowest undrained shear strengths are measured directly below the crust where the liquidity index is highest. The shear strength then increases with depth due to consolidation of the clay.

Desiccated Crust

A hardened, over-consolidated crust often forms at the surface of varved clay deposits as a result of desiccation, frost action, and weathering in general. The water content becomes significantly reduced in this zone, often resulting in liquidity index values of less than 0.5, as compared with values of 1 and greater in the soft, unweathered varved clays.

The dessicated zone usually occurs in thicknesses ranging between 1 m and 5 m, and is usually in the order of 3 m (Lefebvre et al., 1987). A significant characteristic of the desiccated crust is its high undrained shear strength as compared with strengths of the clay underlying the crust. Another very significant feature of the desiccated crust is its often fissured structure, which has an important effect on the strength and drainage characteristics of this material.

SITE INVESTIGATION TECHNIQUES

General Approach to Site Investigations

Extensive work on the strength characterization of soft clays has been carried out by numerous researchers and practitioners. Discussions of such work by Bjerrum (1972), Tavenas et al. (1978), Trak et al. (1980), and Ladd (1991) are recommended as background reading. Three basic lines of approach to the characterization of soil strength for the stability evaluation of embankments have developed:

- The "field vane shear strength" approach employed by Bjerrum assesses the undrained strength of the foundation soils by field vane shear tests or, more recently, cone penetration tests. Because of the tendency of field vane shear tests to over-estimate the strength mobilized in the clay foundations of embankments, an empirical correction factor, μ, is applied to the vane shear strength used in stability analysis. The value of μ has been determined from back-analysis of embankment failures on soft clays.

- The "undrained strength - stress history" approach evaluates the undrained shear strength from the past stress history of the clays. In this case, the undrained shear strength has been shown to be approximately equal to 0.22 σ_p' where σ_p' is the pre-consolidation stress determined from laboratory oedometer tests or interpretation of in situ testing. The "current" undrained strength is applied to each soil unit to analyze the stability of the embankment.

- The "effective strength" approach considers the pore pressures generated in the foundation clays by the embankment loading and employs these with the drained soil friction angle in an effective stress stability analysis. In its most simplest form, the prediction of the generated pore pressures requires a knowledge of the pre-consolidation stress and pore pressure parameter $\bar{B}$ for the foundation soils, a stress analysis to estimate the changes in vertical stresses in the foundation soils, and the effective stress strength parameters (c' and ϕ').

The application of the second and third approaches requires extensive sampling laboratory testing programs to characterize the pre-consolidation stress, drained friction angle and correlation of these parameters with in situ test measurements. The use of one or both of these approaches is appropriate for design of large embankment structures and knowledge of the basic elements of the approaches is important to practitioners working in this area. Use of the third approach is facilitated in instances where field pore pressure measurements are available, such as during staged construction, where the value of $\bar{B}$ can be estimated directly from field measurements. The authors have routinely used the first and third approaches together in such applications. However, for new sites where no field performance data is available for prediction of pore pressure response in varved clay foundatons to embankment loading, the authors routinely employ the first approach for design of small embankments. The first approach is also favoured because of the higher cost and, in some cases, the longer time involved in carrying out the more detailed work required for the second and third approaches.

Some other practical advantages of the first approach are that relatively sophisticated field vane test equipment, such as the Nilcon vane apparatus, can be easily transported to remote sites to make reliable measurements of in situ shear strength even for the smallest project. For larger projects, CPTU equipment is now an economical alternative to the field vane. Furthermore, because in situ shear strengths are measured directly in the field, embankment designs can be initiated almost immediately after the investigation work. For many projects, this is an important consideration because the time between the site investigations to the start of construction can be as short as several weeks. Finally, the design methodology of the first approach is appealing because it is directly related to the field behaviour of embankments through the correction factor, μ.

The following sections summarize typical site investigation results from two soft soil sites in Northern Ontario, taken from the authors' files. For both of these projects, involving construction of embankments on soft clay foundations, the first approach, involving field vane shear tests and CPTU tests, was used.

Site 1

Site 1 is located in Northwestern Ontario, near the Manitoba border. Foundation conditions at this site consist of very soft to soft varved clays up to 25 ft in thickness, overlain typically by about 5 ft of stiff, desiccated clay crust. Key index properties of the varved clays at Site 1 are given in Table 1.

Table 1. Summary of Index Properties at Site 1 and Site 2

SITE	PLASTIC LIMIT PL(%)	LIQUID LIMIT LL(%)	PLASTICITY INDEX PI(%)	MOISTURE CONTENT W_c(%)	LIQUIDITY INDEX LI
1	24	70	46	71	1.02
2	19	47	28	30	0.4

The plasticity data for Site 1 is plotted on the plasticity chart shown on Figure 4. The data indicates a clay of high plasticity according to the Unified Soil Classification System (USCS). Grain size analyses of the clay yielded silt and clay contents of 30% and 70% by weight, respectively.

For Site 1, the undrained shear strength (S_u) was determined from Nilcon Vane shear tests, CPTU soundings, Undrained Unconfined (UU) triaxial tests, and from correlations with index properties using empirical relationships described in the literature.

Site 2

Site 2 is located in Northeastern Ontario. Foundation conditions at this site consist of soft varved clay and silt up to 55 ft in depth, overlain in some areas by a desiccated clay crust up to 3 ft in thickness. Key index properties of the varved clay and silt from Site 2 are summarized in Table 1. Grain size analyses yielded silt and clay contents of 60% and 40% respectively by weight. The varved clay and silt at Site 2 is clearly different from the varved clay at Site 1, in terms of gradation and index properties.

Similar methods to those described above for Site 1 were used to characterize S_u for Site 2.

Nilcon Vane Shear Test Results

For both investigations, the raw field vane shear data was corrected based on established empirical relationships between field vane shear strength and back-analysed S_u values from embankment failures. The relative values of shear strengths have been examined by a number of researchers (Bjerrum, 1972, 1973, Ladd, 1975 and Ladd et al, 1977). The relationship between the ratio of "true" undrained shear strengths (based on back-analysis of embankment failures) to vane shear values versus plasticity index of the clay materials from their work is presented in Figure 8. Bjerrum (1973) attributed this discrepancy between the measured vane strength to field shear strengths to strain rate and anisotropy effects. For layered and varved clays, a lower bound relationship, as shown in Figure 8, is considered by the authors to be appropriate, given the continuous planes of formed by the clay varves.

The S_u value is corrected, using Bjerrum's approach, as shown in Equation 1:

$$S_u = S_u(\text{field}) \times \mu \tag{1}$$

where:

S_u	=	Corrected undrained shear strength
S_u(field)	=	Field vane shear strength
μ	=	Field vane correction factor

As noted by Aas et al (1986), the scatter of data points about Bjerrum's recommended curve is significant. This scatter has led Milligan (1972), Schmertmann (1975) and Schmertman and Morgenstern (1977) to question the validity of this design approach. Aas et al (1986) suggested that the scatter may be due in part to the fact that Bjerrum's μ versus plasticity index relationship did not account for clays subjected to different stress histories. Aas et al (1986) further suggested that Bjerrum's approach could be extended to develop a correlation in terms of the plasticity index, field vane strength normalized to the in situ effective overburden stress, and whether the clay is young, aged or overconsolidated.

This refinement of Bjerrum's approach was not incorporated in the embankment designs at Sites 1 and 2 because use of the lower bound relationship for varved clays was considered appropriately conservative for the relatively low embankments that were constructed. However, for larger embankments, where this degree of conservatism can have significant economic impacts, the refinement suggested by Aas et al (1986) merits serious consideration.

For Site 1, with an average plasticity index of 46%, a correction factor $\mu = 0.67$ was obtained from the lower bound relationship for varved clays. A typical corrected vane shear profile for Site 1 is shown on Figure 9, which indicates S_u values of 800-900 psf in the desiccated clay crust, and 340-540 psf in the underlying soft clay.

For Site 2, with an average plasticity index of 19%, the lower bound relationship for varved clays shown on Figure 8 yielded $\mu = 0.88$. A typical corrected vane shear profile for Site 2 is shown on Figure 10. For the desiccated clay crust and the underlying soft clay and silt, S_u values of about 1200 psf and 500 psf, respectively, were obtained.

CPTU Results

Comprehensive reviews of S_u evaluation from CPTU data are presented by Baligh et al (1980), Jamiolkowski et al (1982), Lunne and Kleven (1981), Robertson et al (1986), and Aas et al (1986).

CPTU soundings were carried out at both Sites 1 and 2. Empirical relationships have been developed between the cone bearing (q_T) and excess pore pressure measurements (Δu), as follows:

(2)

$$S_u = \frac{q_T - \sigma_{vo}}{N_k}$$

and

(3)

$$S_u = \frac{\Delta_u}{N_{\Delta_u}}$$

where:

q_T	=	cone tip resistance
σ_{vo}	=	total overburden stress
Nk	=	empirical cone factor
Δu	=	dynamic pore pressure - equilibrium pore pressure
$N_{\Delta u}$	=	empirical pore pressure factor

In very soft clays, q_T values can be much lower than the sensitivity of the cone load cell. In normally consolidated clays, the dynamic pore pressures generated during testing become significant. Accordingly, where measured q_T values are less than the dynamic pore pressures, the pore pressure relationship is considered to yield more realistic S_u values.

The values of Nk and $N_{\Delta u}$ vary depending on the type of cone used, and on the nature of the soil. It is good practice to obtain these values based on site-specific correlations with corrected field vane shear strength data.

Typical CPTU profiles for Site 1 and Site 2 are shown on Figures 11 and 12, respectively. Both profiles indicate the stiffer, desiccated crust at shallow depth, underlain by soft clays in which significant dynamic pore pressures are generated by cone penetration. In the case of Site 1 (Figure 11), q_T values of near zero were obtained between depths of 9 m and 13 m. The q_T profile for Site 2 clearly indicates a more competent material. For the Site 1 profile, dynamic pore pressures of up to 300 kPa (100 ft of water) were recorded, compared with q_T values as low as 10 kPa. Clearly, for this profile, use of the q_T relationship for S_u determination would not be appropriate. Figure 9 shows the S_u versus depth profile obtained for the CPTU profile for Site 1 shown on Figure 11. This profile was obtained using an $N_{\Delta u}$ factor of 15. For the soft clay, a lower bound undrained shear strength of about 380 psf was obtained. Other CPTU profiles at Site 1 indicated S_u values in the soft clay increasing with depth from 300 psf at the base of the dessicated clay crust to 450 psf at the base of the soft clay, giving an Su/σ_p' ratio of about 0.14.

For the Site 2 profile, q_T values of about 800 kPa were obtained, compared with dynamic pore pressures of up to 392 kPa (40 m of water). In this instance, both the q_T and pore pressure relationships could be used for determination of S_u. Three typical CPTU-based S_u versus depth profiles for Site 2 are shown on Figure 10. These profiles were determined using the q_T relationship, with an Nk of 15. The profiles indicate a lower bound S_u of about 600 psf in the soft clay and silt, and an S_u of about 1500 psf in the desiccated clay crust. The profiles also indicate an S_u/σ_p' ratio of about 0.3 for Site 2, indicating that the clay and silt is normally consolidated to slightly overconsolidated.

Unconsolidated Undrained (UU) Test Results

Unconsolidated Undrained (UU) tests were carried out on Shelby tube samples of the soft clay from both Sites 1 and 2, primarily because this is a quick and inexpensive means of obtaining an estimate of S_u. The significance of results from this test for varved clays is suspect, as the failure plan is forced to cut across the varves, while actual failure in the field is often more likely to fail along the varves, which represent planes of weakness in the embankment foundation.

For Site 1, a single UU test was carried out, and yielded a peak S_u of 350 psf at 5% strain level, and a large strain (residual) S_u value of 200 psf.

Three UU tests were carried out on samples from Site 2, and the results are plotted on Figure 10. The results indicate reasonable agreement with the vane shear and CPTU-based S_u profiles.

Correlations with Liquidity Index

The S_u of clays from different databases have been correlated to liquidity index by various authors. Figure 6 illustrates a correlation chart between liquidity index and the S_u/σ_p' ratio. Data from Sites 1 and 2 are plotted on this chart. The data plot for Site 1 gives an S_u/σ_p' ratio of about 0.14 for an average liquidity index of 1.02. For Site 2, the data gives an S_u/σ_p' of about 0.3 for an average liquidity index of 0.4. The results from both sites agree with the respective empirical trend line shown in Figure 6, and show good agreements with the S_u/σ_p' ratios obtained from the CPTU profiles.

Consolidation Properties

Consolidation properties represent a key aspect of the characterization of the S_u of varved clays. The key consolidation properties for characterization of the S_u of varved clays are:

- the maximum past preconsolidation pressure to which the clay has been subjected in situ;

- the coefficient of consolidation (C_v); and

- the drainage path length for consolidation of the foundation clays.

Comparison of the maximum past preconsolidation pressure with the estimated overburden pressure in situ allows a direct computation of the overconsolidation ratio. This also allows the designer to predict whether embankment loading will result in generation of excess pore pressures within the clay foundation, and to determine whether the use of the total stress analysis approach, using undrained shear strengths, is appropriate.

The coefficient of consolidation is a critical parameter in instances where the rate of construction is limited by the time required for gain in foundation strength due to consolidation. This is a common consideration, for instance, in the construction of tailings impoundments, where the tailings dykes are raised each year to meet increasing storage capacity requirements. Knowledge of C_v, and selection of the appropriate S_u/σ_p' ratio, allows the designer to predict the rate of consolidation, and therefore gain in S_u, in the clay foundation. The designer is therefore able to estimate the allowable rate of construction when foundation strength gain is required to assure embankment stability.

The rate of consolidation is also controlled by the length of the drainage path in the field. It is therefore critical for the designer to distinguish between single drainage and double drainage conditions when estimating the rate of field consolidation. Proper characterization of the drainage conditions allows the designer to determine the location within the clay sequence where consolidation and strength gain will occur most slowly. For stability analysis of embankment stability for subsequent raises of the structure, the designer will then be able to predict at what depth within the clay sequence the lowest undrained strength will occur.

For the investigations at Sites 1 and 2, Oedometer tests were carried out to determine the consolidation properties of the clay deposits. The coefficient of consolidation (C_v) results from these investigations are plotted on Figure 13, which shows an empirical relationship between Liquid Limit and C_v. The results fall below the relationship for undisturbed clay samples from Navdoc (1971). For a representative C_v of 5×10^{-4} cm/sec, a period of 6 years is rrequired for full drainage of excess pore pressures at a typical site with 6 m of soft clay and double drainage conditions.

Pneumatic piezometers are frequently used to provide field verification of design assumptions regarding generation of pore pressures upon embankment construction and the gradual dissipation of these pore pressures as consolidation proceeds. Settlement gages are less useful in this regard, as for stability analysis the designer is interested in the rate of consolidation rather than in its absolute amount. Pneumatic piezometers installed within the soft clay foundation represent an absolutely essential component of design and construction, due to the uncertainties inherent in application of laboratory data to field conditions. Good practice is to install pneumatic piezometers at several depths within the clay sequence, but most importantly at the point at which the designer predicts consolidation to proceed at the slowest rate. Reliance on pneumatic piezometer readings to assess foundation strength gain, when those piezometers are not in the critical location, can lead to overestimation of foundation strength gain.

A more direct approach to measure strength gain due to consolidation is to carry out vane shear tests or CPTU tests at regular time intervals. As an example, Figure 14 shows the corrected vane shear strength profile at a soft clay site that was measured 4 years after loading of the site with 4 ft of rock fill. Laboratory oedometer tests on the clay yielded a C_v of 2×10^{-4} cm/sec. The predicted shear strength profile using this C_v value and double drainage conditions in the clay agreed reasonably well with direct measurements. Clearly, a long period of many years will be required to achieve significant consolidation and strength gain at the middle of the soft clay layer. This slow strength gain was a consideration in recommending the embankment at this site not be raised without additional measures to increase the rate of drainage or enhance stability.

STABILITY ANALYSIS METHODS

Empirical Method

A key step in assessing the design of embankments on soft clays is to determine whether staged construction will be required to allow for consolidation and strength gain in the clay foundation. Figure 15 shows that there is a good empirical relationship between the height of failed embankments, H, and the corrected field vane shear strength, S_u, in the underlying soft clay. As also shown on Figure 15, the relationship from embankment failures correlates very well with simple foundation stability analysis where the embankment load is treated as an equivalent footing load. Based on Figure 15, the maximum embankment height to cause failure can be estimated as $H = 0.05\ S_u$ (psf). Allowing for a suitable Factor of Safety of 1.3 to 1.5, the allowable embankment height can be estimated as $H_{allow} = 0.035\ S_u$ (psf). Typically, the allowable embankment heights range from 3 ft (1 m) to 20 ft (6 m) for undrained strengths between 100 psf and 600 psf.

Where the required embankment height exceeds H_{allow}, staged construction with suitable allowance between stages for consolidation of the soft clay will be required. The increase in shear strength with time can be estimated using one-dimensional consolidation analysis as demonstrated in the previous section. Field verification of the strength gain, preferably by in situ strength measurements, in concert with field pore pressure measurements, is strongly recommended.

Where the time required for sufficient consolidation is longer than can be allowed by the project, then additional construction measures must be implemented. Common measures are the installation of wick drains to speed the consolidation, partial or full excavation of the clay and replacement with compacted fill, and large stabilizing berms.

Detailed Stability Calculations

The authors have reviewed a number of failures of mine waste embankments founded on soft clay in Northern Ontario. The common elements among the failures were considered to be as follows:

- the embankments were designed using short-term end of construction undrained stability analyses with full peak undrained shear strength assigned to the desiccated clay crust; and

- no appropriate intermediate stability analyses were carried out which considered the effects of rapid drainage of excess pore pressures in the over-consolidated clay crust or strain compatibility between the soft clay and stiffer clay crust.

It is the authors' opinion that the short-term loading condition determined using the peak undrained shear strength in the clay crust is misleading for typical soft soil sites as shown on Figure 1. This is because the undrained shear strength for the over-consolidated clay crust relies on high negative pore pressures sustained during rapid shearing. Rapid drainage in over-consolidated soils allows these negative pore pressures to dissipate at a rate many times faster than in the underlying soft clays and, therefore, the shear strength in the crust quickly reduces to a drained condition. In the case histories examined, it is believed that this time for drainage to occur resulted in the short time delays, up to 4 days, for the failures to occur after the end of construction. Therefore, the authors advocate that appropriate intermediate stability analyses should be carried out to assess stability with a reduced drained strength in the clay crust.

Strain compatibility between the stiffer embankment fill and clay crust, and the underlying soft clay is also a factor which can impact embankment stability (Ladd, 1991). Because of the smaller strains to peak strength in the stiffer fill and crust, it can be argued that a lower strength should be applied to the underlying soft clay which has a larger strain at peak strength. Because strain compatibility calculations require appropriate test data, such calculations are not usually carried out in a rigorous manner for the design of small embankments. However, the authors note that by using an drained friction angle for the desiccated clay crust in an intermediate stability calculation, strain compatibility is indirectly addressed because the strength parameters represent the soil strengths mobilized at the common point of peak strain in the soft clay.

The important effect of the stiff clay crust on the stability of the embankments has been previously recognized by other engineers. Empirical reductions of the undrained strength in the crust have been employed to compensate against overestimating the soil resistance (Trak et al., 1980). These include limiting the maximum strength to that measured at the mid-depth of the desiccated layer or, in the most conservative case, using the minimum undrained shear strength at the bottom of the layer. The authors consider that these reductions crudely represent shear strengths which are closer to the actual mobilized strength of the clay crust. However, the selection of the reduced strength is subjective and, at some sites, these reductions may still represent an unsafe assumption.

In summary, the stability of embankments founded on soft clays with an over-consolidated crust must be carefully evaluated for the effects rapid drainage of pore pressures in the clay crust and effects of strain compatibility. For design of small embankments, the authors recommend that the following stability conditions be considered as a minimum requirement:

- The short-term stability is analyzed using the corrected undrained shear strength in the soft clay and field vane shear strength in the clay crust. The embankment materials are usually assigned a drained friction angle with a suitable pore pressure ratio, r_u, for the compacted fill. For low embankments, r_u values between 0 and 0.3 are typical for soils compacted near optimum water content.

- An intermediate stability calculation is carried out to address the effects of rapid pore pressure drainage and strain compatibility. For this condition, the stability is analyzed using a drained friction angle in the clay crust and an appropriate phreatic level. The undrained shear strength is still used in the soft clay because of the very slow consolidation and strength gain which is typically experience in these soils (see Figure 14). The drained friction angle and r_u value used in the short-term condition is applied to the embankment fills.

- A long-term stability calculation is carried out using drained friction angles in all materials and appropriate long-term phreatic levels.

Target Factors of Safety for the short-term and intermediate conditions are typically set at 1.3. The intermediate stability calculation usually represents the critical condition for the embankment design. The long-term condition represents a more stable embankment condition and Factors of Safety are typically in excess of the minimum target of 1.5.

Case History Example

To demonstrate the application of the short-term, intermediate and long-term stability conditions, the authors applied the recommended approach to the New Liskeard Embankment failure discussed by Lacasse et al. (1977) and Trak et al. (1980). The New Liskeard case history involved approach embankments constructed on a 43 m deep deposit of medium to soft varved clay with a 2.7 m thick silty clay crust. Because of the low strength of the varved clay, a staged construction scheme was adopted. An unexpected failure occurred when the fill had almost reached the design height of 6.1 m of the first stage.

Figure 16 shows the configuration of the failed embankment and the uncorrected field vane shear strengths for the New Liskeard clay taken from Lacasse et al. (1977). Table 2 summarizes the short-term, intermediate and long-term shear strength parameters used in the analysis by the authors. The granular embankment fill was assigned a friction angle of 40° and r_u = 0. The average field vane shear strength of 900 psf in the clay crust was used for short-term conditions. For intermediate and long-term conditions, a friction angle of 23.5° and c' = 200 psf was used to represent drained conditions. The shear strength of the soft clay was estimated using a field vane correction factor of μ = 0.7 based on the average PI of the soft varved clay of 49%. A drained friction angle of 21° and c' = 100 psf was used in the soft clay for long-term conditions.

Table 2. Shear Strength Parameters for Case History

MATERIAL	SHORT-TERM CONDITION	INTERMEDIATE CONDITION	LONG-TERM CONDITION
Embankment Fill	ϕ' = 40° r_u = 0	ϕ' = 40° r_u = 0	ϕ' = 40° r_u = 0
Clay Crust	Average Su = 900 psf	ϕ' = 23.5° c' = 200 psf	ϕ' = 23.5° c' = 200 psf
Soft Clay	Field vane x 0.7	Field vane x 0.7	ϕ' = 21° c' = 100 psf

The calculated Factors of Safety for the critical slip surface inferred from the post-failure investigations are given in Table 3. The short-term and intermediate Factors of Safety for the embankment were computed to be 1.07 and 0.94, respectively. The low short-term Factor of Safety indicates that it is probable that the embankment was only marginally stable during construction and little drainage in the crust was required to trigger the on-set of failure as indicated by the intermediate Factor of Safety. This agrees with the almost immediate failure of the embankment during construction. Although the computed long-term Factor of Safety was 2.1, this calculation clearly has no significance in terms of the actual embankment stability during construction.

Table 3. Computed Factors of Safety for Case History

CONDITION	FACTOR OF SAFETY
Short-Term	1.07
Intermediate	0.94
Long-Term	2.1

Interestingly, the Factors of Safety calculated for a field vane correction factor of μ = 1 in the soft clay was 1.29 for short-term conditions and 1.18 for intermediate conditions. These higher Factors of Safety show that the field vane tended to overestimate the actual mobilized shear strength of the soft clay and validates the need for applying the field vane correction factors as first proposed by Bjerrum (1972).

Table 4 summarizes the short-term Factors of Safety for a range of selected undrained shear strengths. The Factors of Safety range from 0.81 and 1.32 over the range of undrained strengths between 400 psf and 1400 psf. Failure is predicted using an average undrained shear strength equal to that near the lower 1/3 of the crust thickness. While the authors do not advocate the use of the short-term condition in this manner for design, limiting the maximum undrained shear strength of the clay crust to the value measured at the bottom third of the crust is indicated to be advisable using this design procedure. This is in agreement with empirical reductions reported by Trak et al. (1980).

Table 4. Variation in Short-Term Factors of Safety with Assumed Shear Strength of the Clay Crust

UNDRAINED SHEAR STRENGTH OF CLAY CRUST (psf)	COMPUTED SHORT-TERM FACTOR OF SAFETY
1400 (maximum at top of crust)	1.32
900 (average)	1.07
733 (lower 1/3 of crust)	0.96
400 (minimum at bottom of crust)	0.86

CONCLUSIONS

This paper has given an overview of the authors' approach to site investigation and design of small embankments founded on soft, varved clay in Northern Ontario. Because these clays are often found to be normally consolidated, they are weak and consolidate very slowly under loading. Embankment heights are generally restricted to less than 6 m unless staged construction, flat slopes, or other stability measures are implemented.

For small embankments, the authors advocate the use of the field vane shear and CPTU for determining the undrained shear strength of the clay crust and soft clay. The lower bound relationship for the field vane correction factor, μ, is recommended for the soft varved clays, unless a detailed evaluation of the influence of the varves on the field vane shear test is carried out. Where the soft clays exhibit extremely low shear strength, a more accurate interpretation of the in situ shear strength can sometimes be obtained from the CPTU dynamic pore pressure measurements. The undrained shear strength of the clay crust is usually not corrected.

The over-consolidated clay crust plays an important role in the stability of small embankments because the high passive resistance of the crust acts as a stabilizing strut against the dam toe. The selection of the appropriate strength for the crust must take into account such factors including rapid pore pressure drainage and strain compatibility with the underlying soft soils. For this reason, the authors recommend that an intermediate stability analysis be carried out which considers a reduced strength in the clay crust. This strength is preferably the drained shear strength of the crust soils. Alternatively, an average undrained strength selected from the bottom to lower third of the crust could be used.

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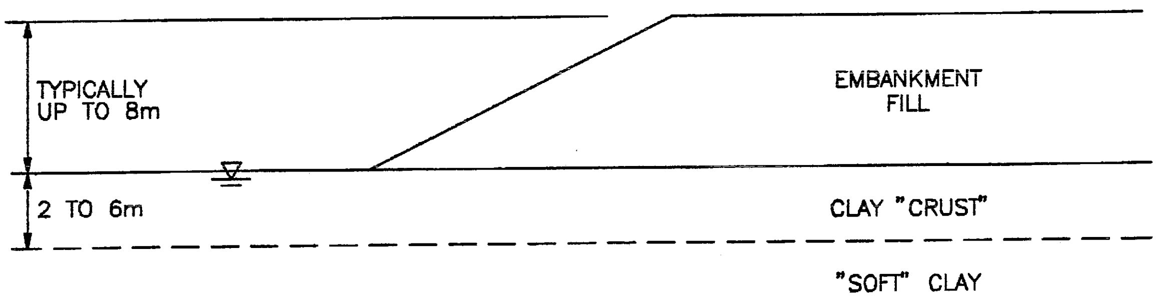


KEY DESIGN PARAMETERS

EMBANKMENT FILL: ϕ', c', γ, r_u

CLAY CRUST: S_u, ϕ', c', γ

SOFT CLAY: S_u, γ, C_v

Figure 1. Design Configuration of Typical Embankment.

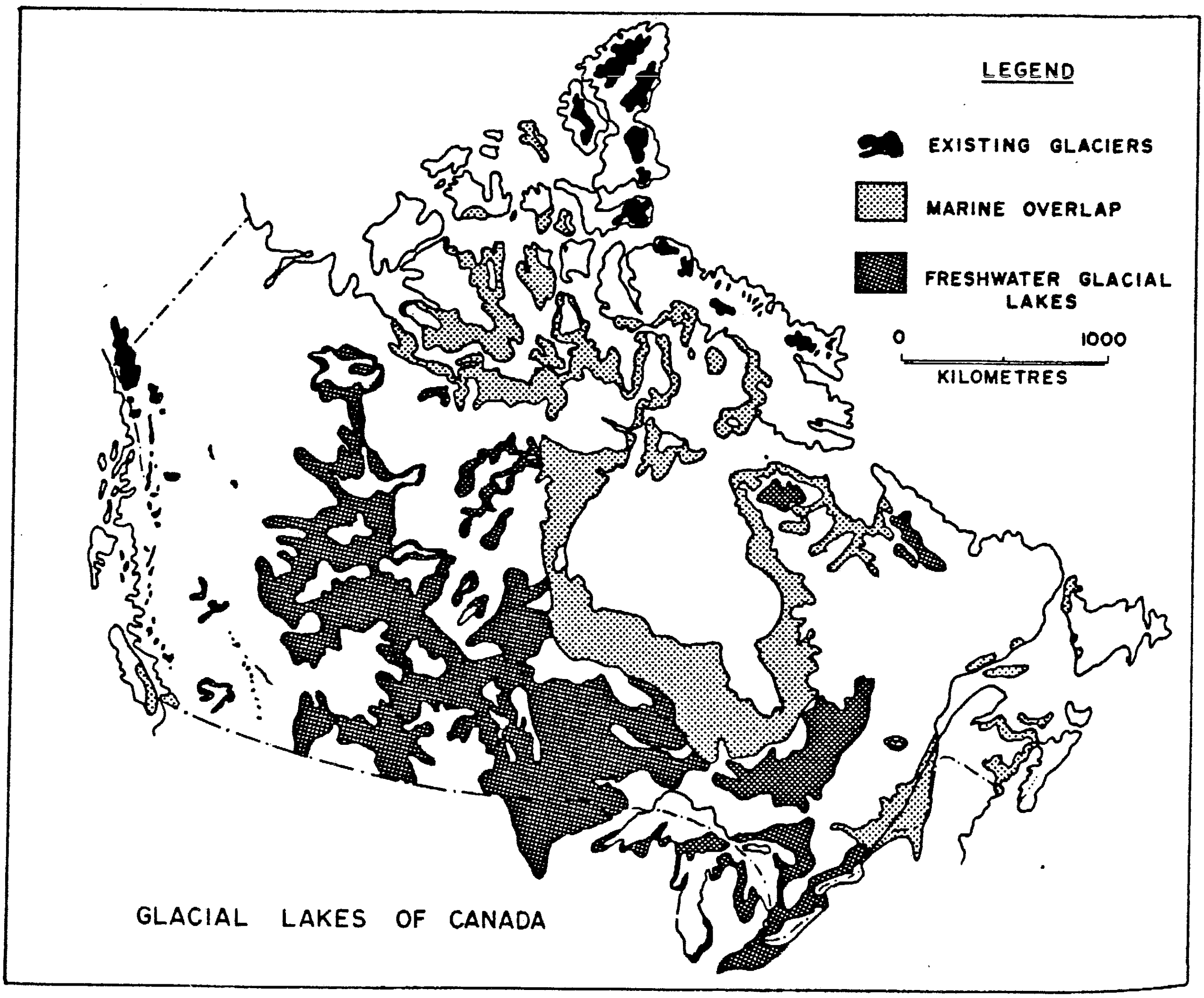


Figure 2. Distribution of marine and freshwater glacial and postglacial lakes in Canada (Quigley, 1980)

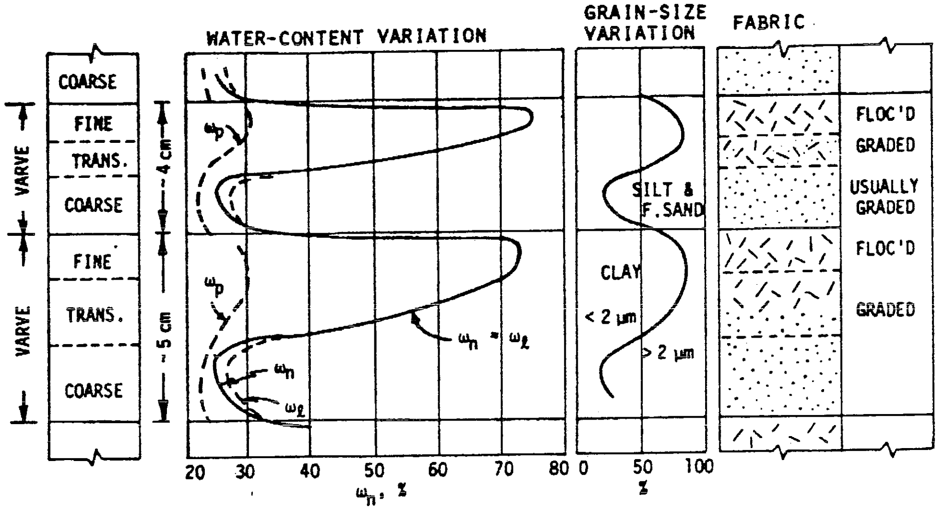


Figure 3. Topical thick–layer varves from New Liskeard, Ontario (Channel Kenney, 1973; Quigley, 1980)

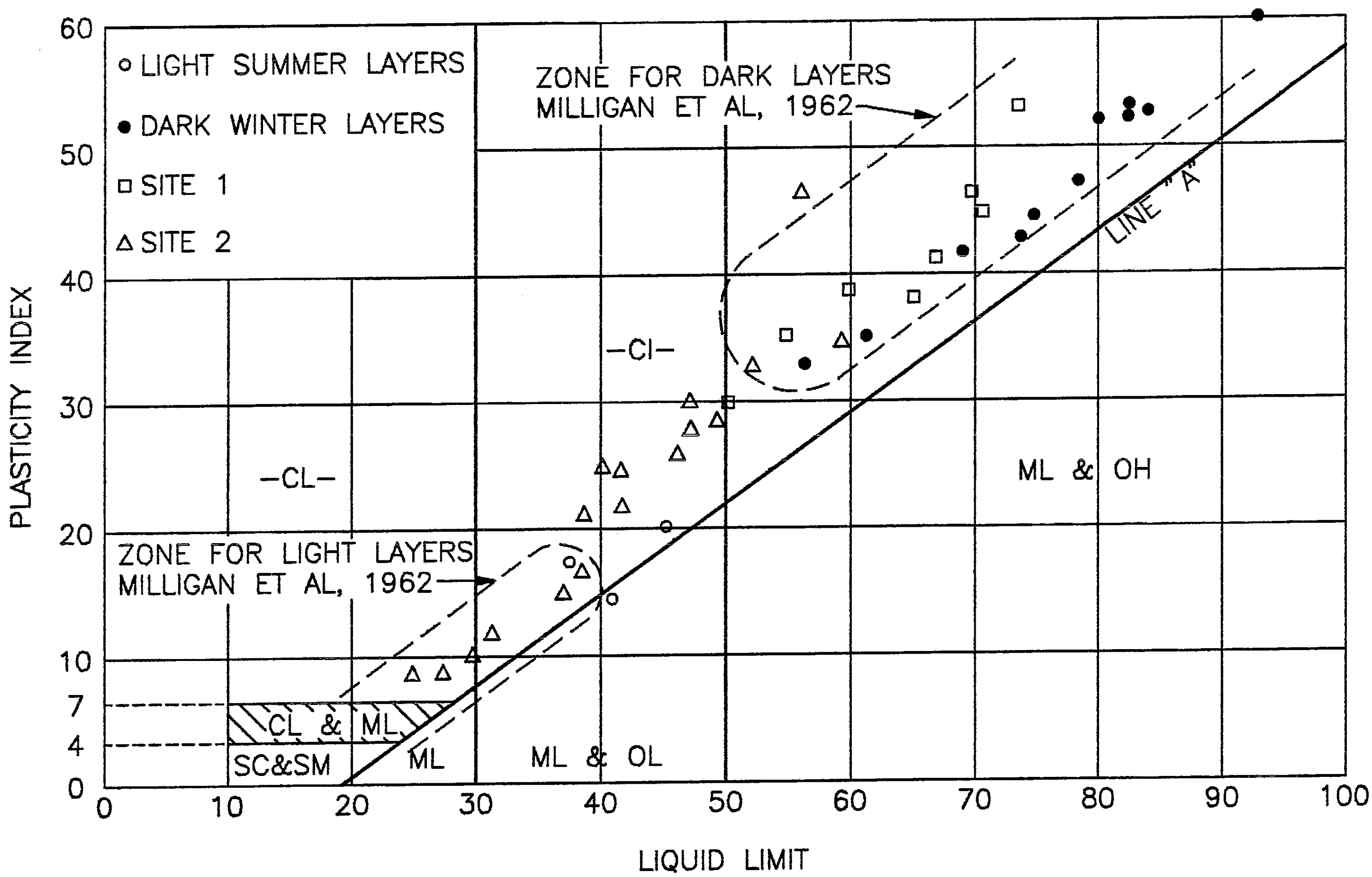


Figure 4. Plasticity chart for dark, winter varves and light, summer varves.

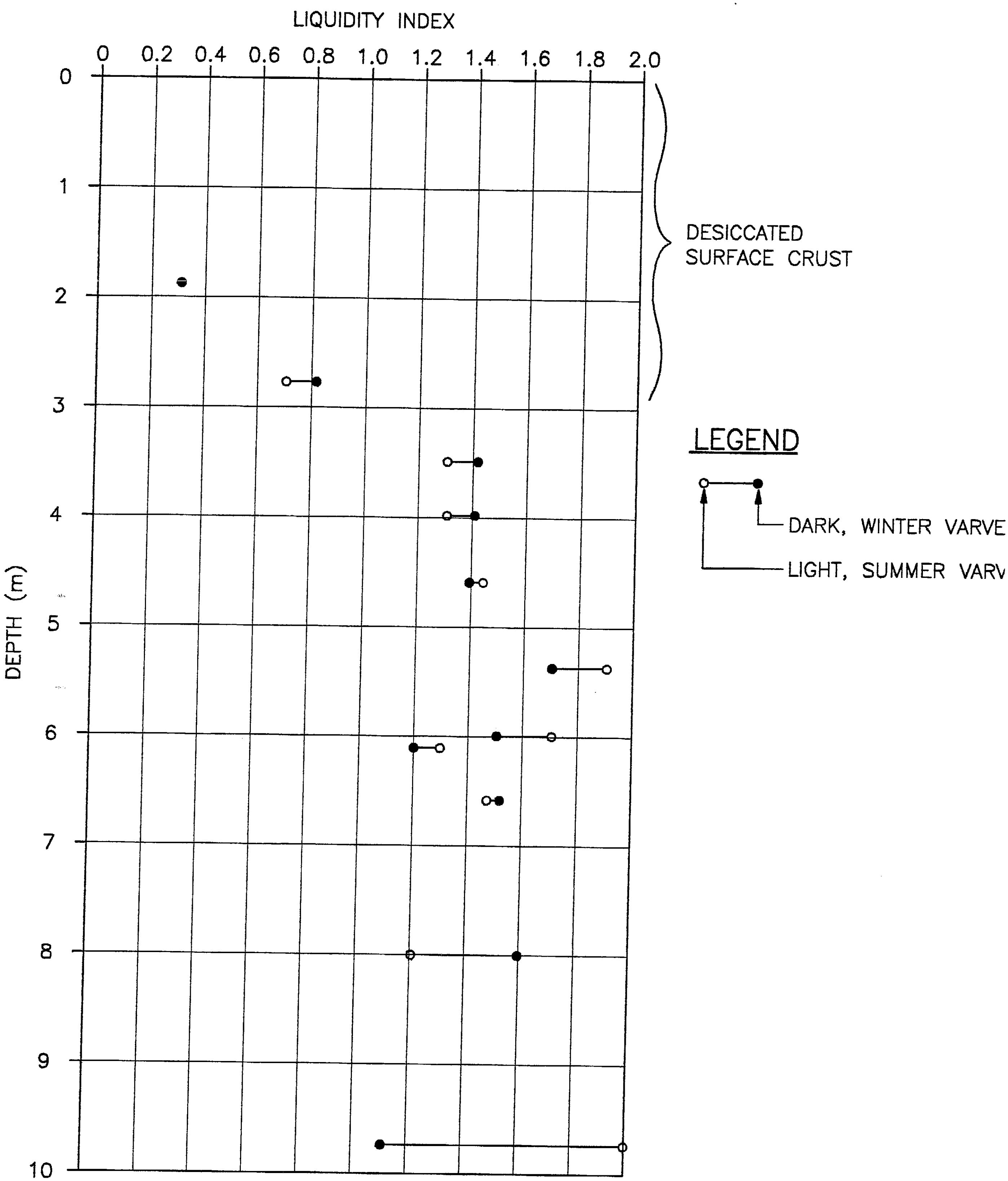


Figure 5. Liquidity Index of varves.

(QUIGLEY ET AL, 1977)

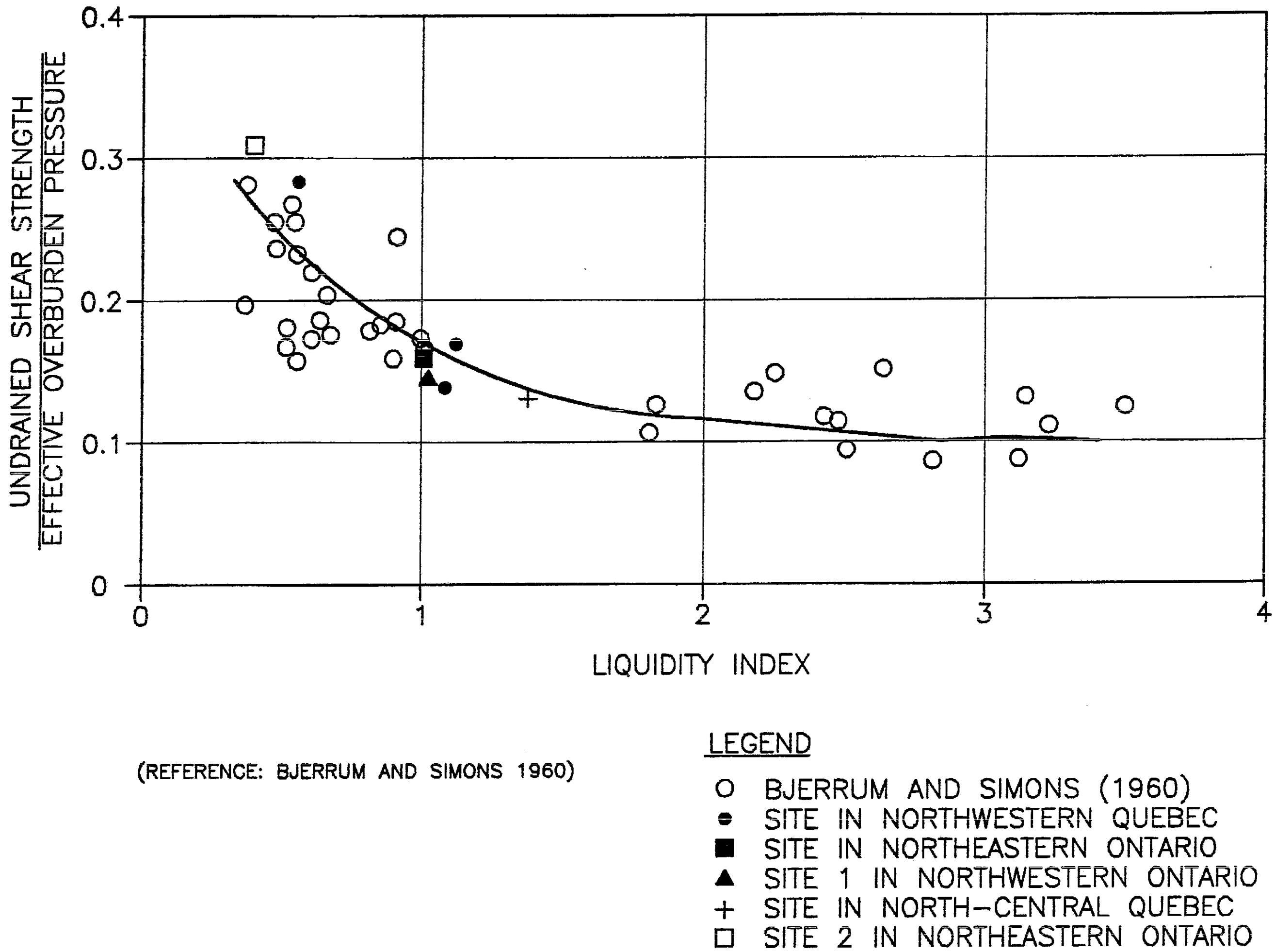


Figure 6. Correlation of Undrained shear Strength Ratio with Liquidity Index for Normally Consolidated clays.

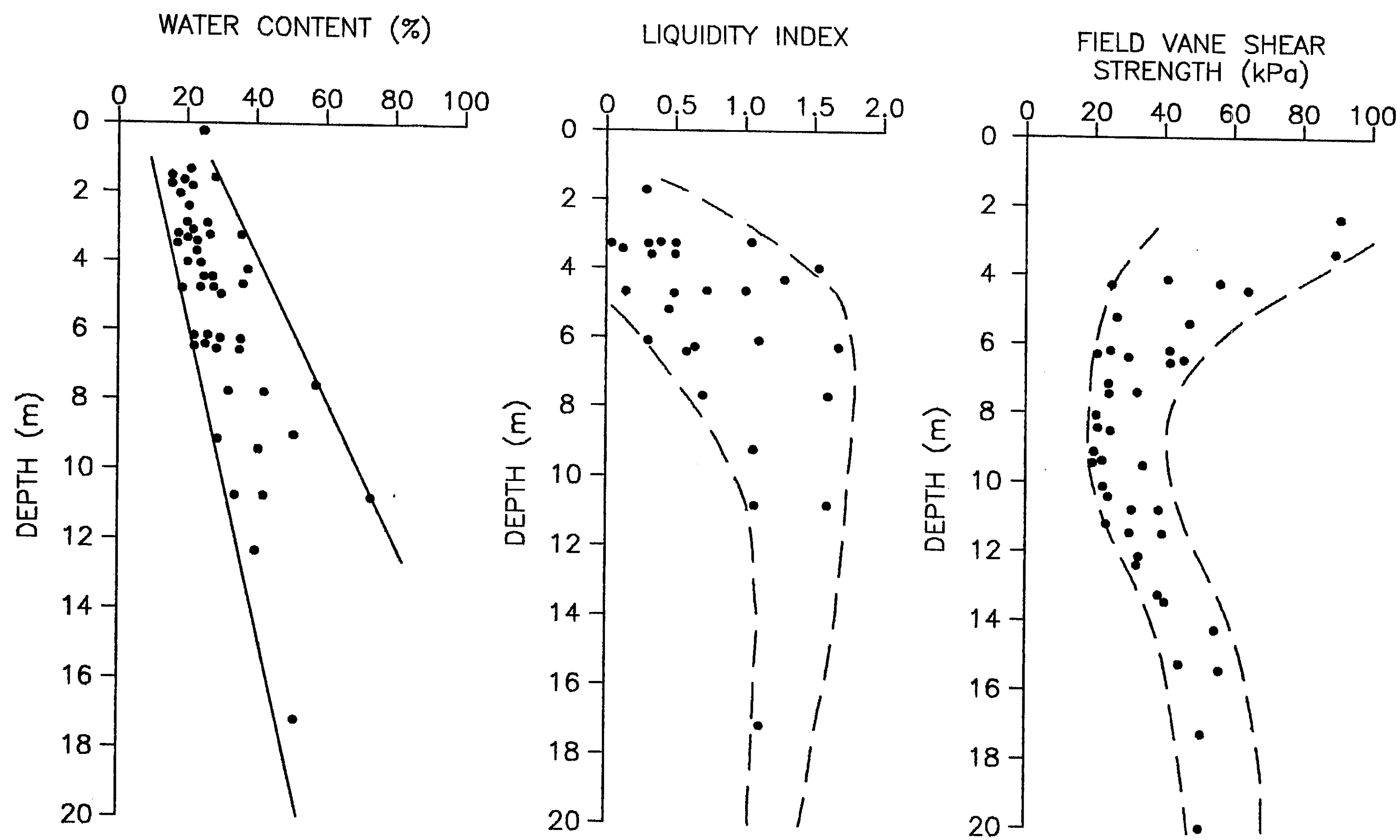
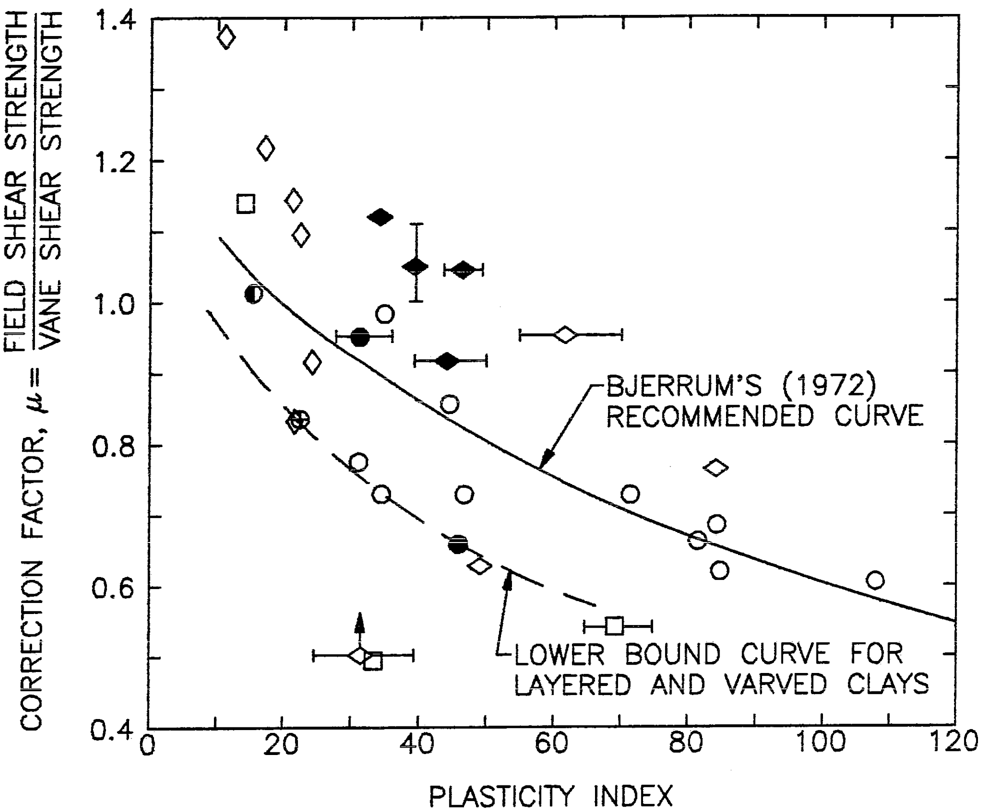


Figure 7. Typical Site Investigation Results for Site in Northern Quebec.



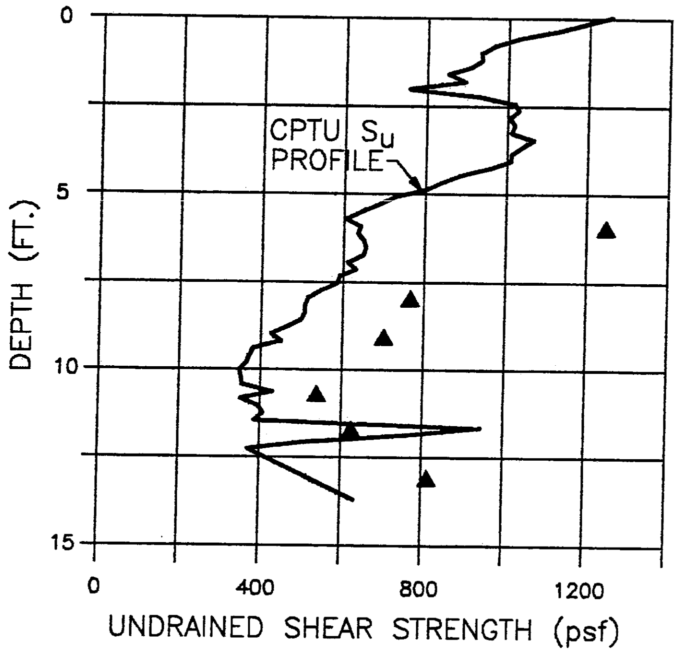
(REFERENCE: LADD, 1975 AND LAD ET AL., 1977)

LEGEND

○●* BJERRUM (1972)
◇◆* MILLIGAN (1972)
□ LADD AND FOOTT (1974)
◇ FLAATE AND PREBER (1974)
⊕ LaROCHELLE ET AL. (1974)
□ HOLTZ AND HOLM (1979)

*LAYERED AND VARVED CLAYS

Figure 8. Correction Factor for Field Vane Test Results



LEGEND:

▲ FIELD VANE RESULTS (CORRECTED, μ=0.67)

NOTE:

CORRECTION FACTOR μ OBTAINED USING LOWER BOUND CURVE FOR LAYERED AND VARVED CLAYS (SEE FIGURE 8)

Figure 9. Typical CPTU and Field Vane Based Su Profile – Site 1.

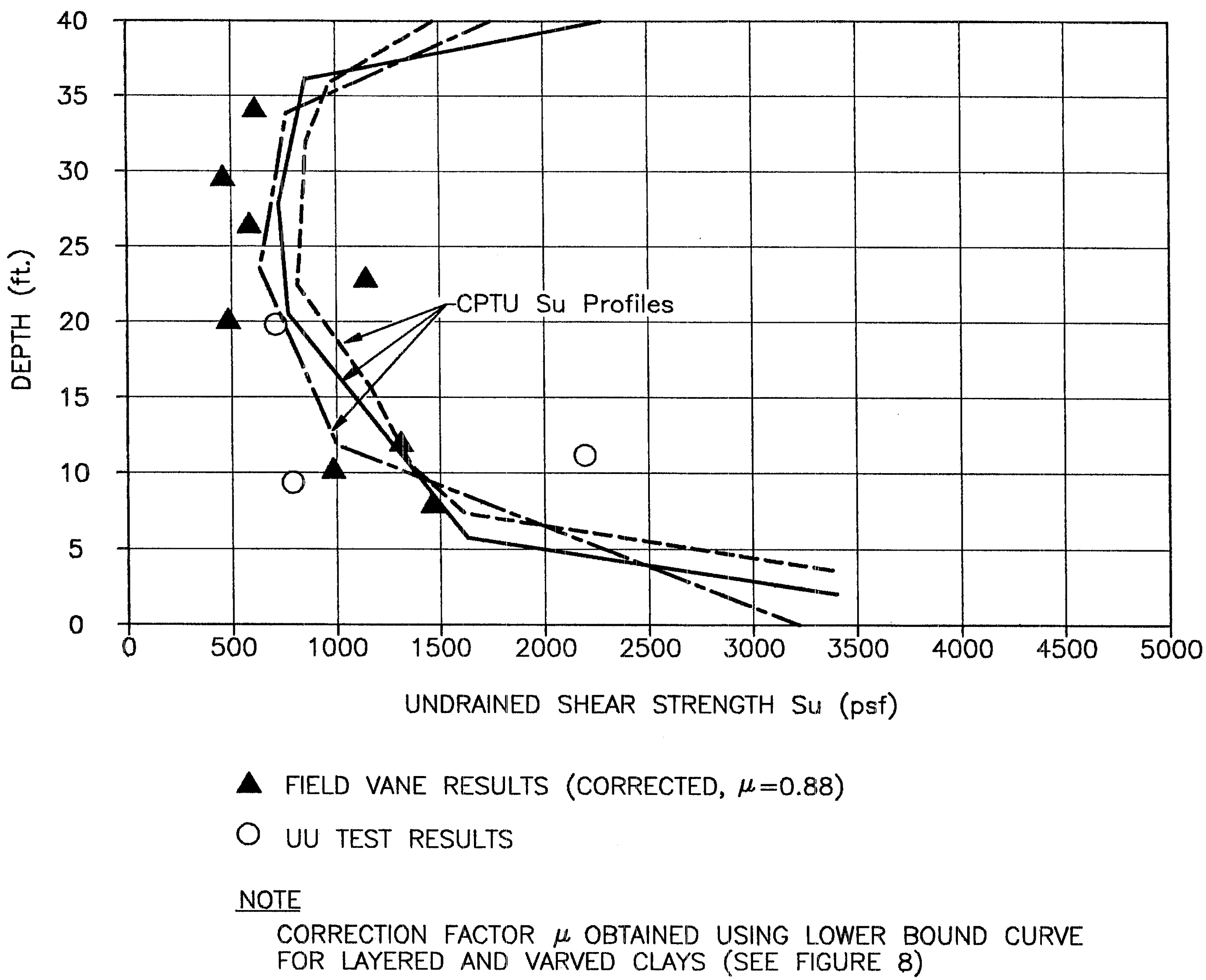


Figure 10. CPTU and Field Vane based Su Profile – Site 2

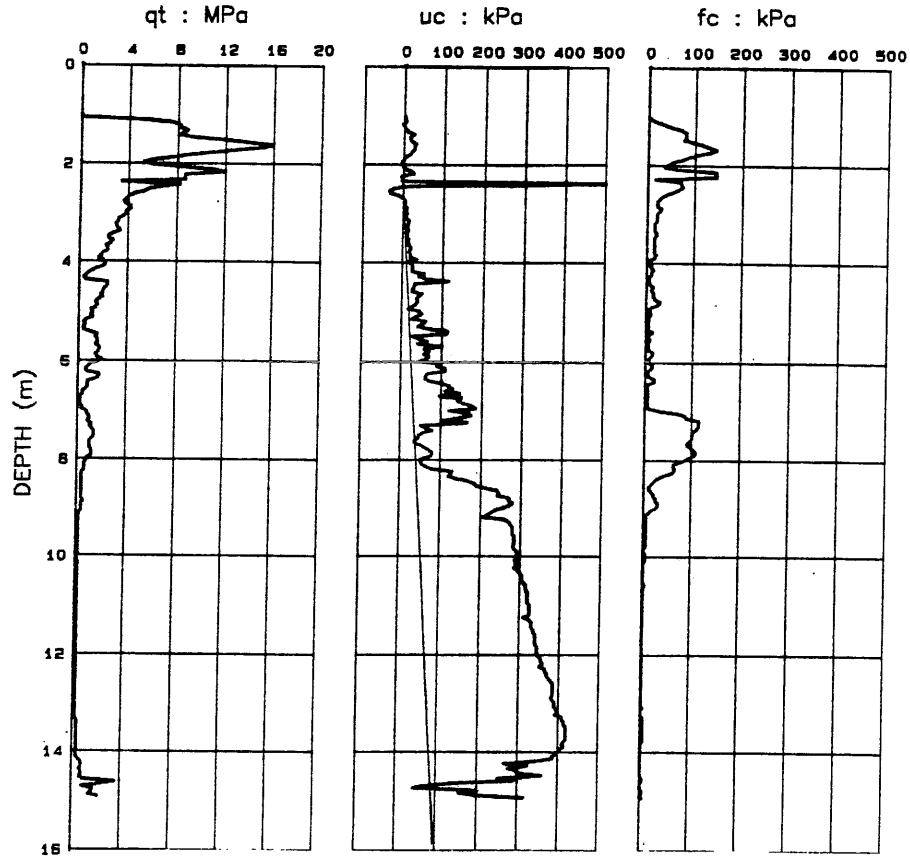


Figure 11. Typical CPTU profile – Site 1

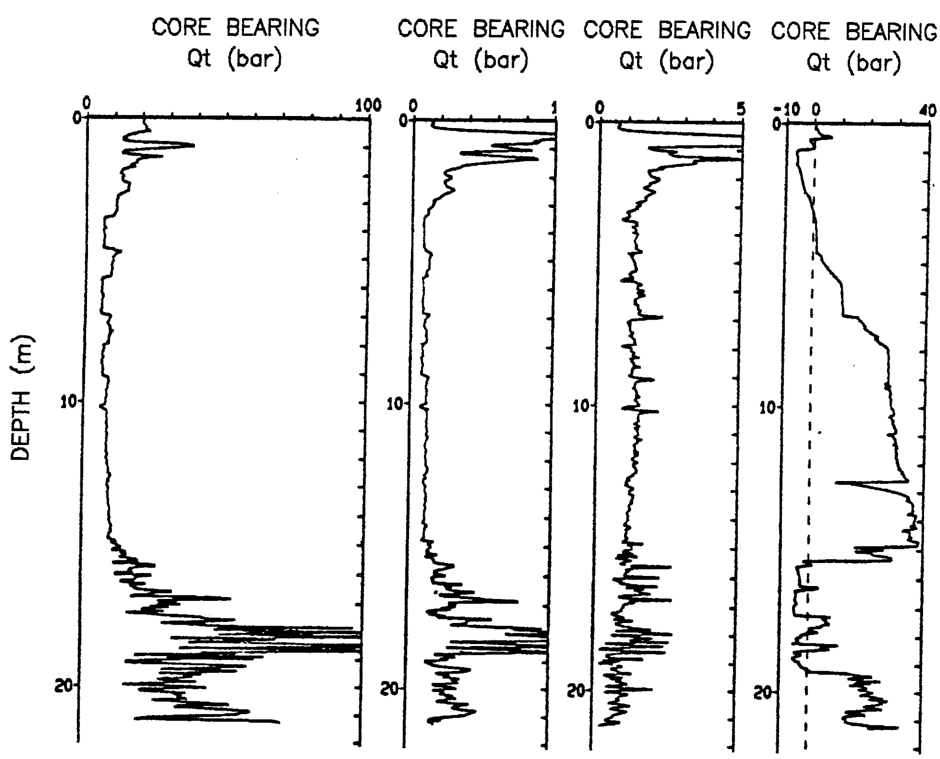


Figure 12. Typical CPTU profile – Site 2

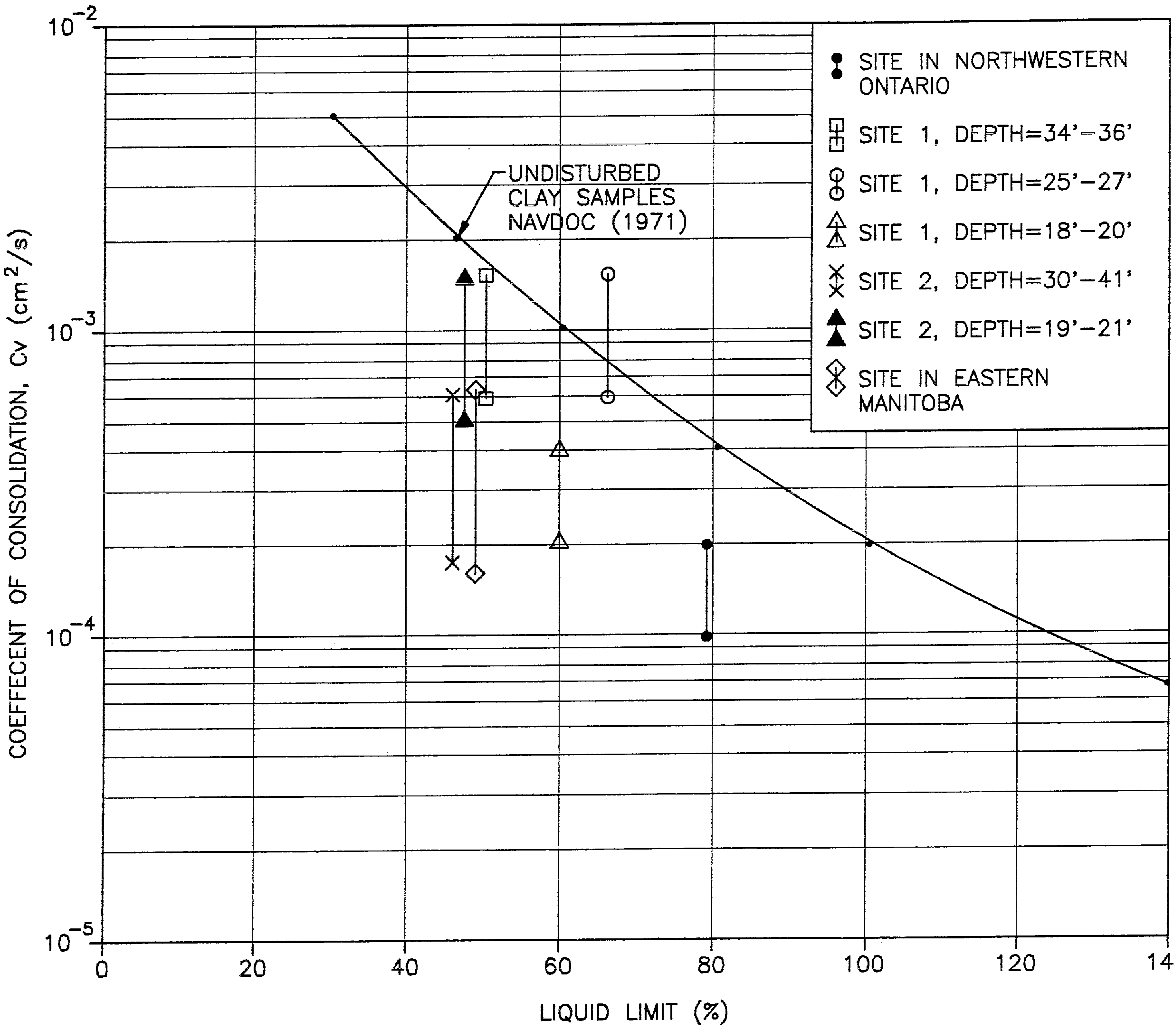
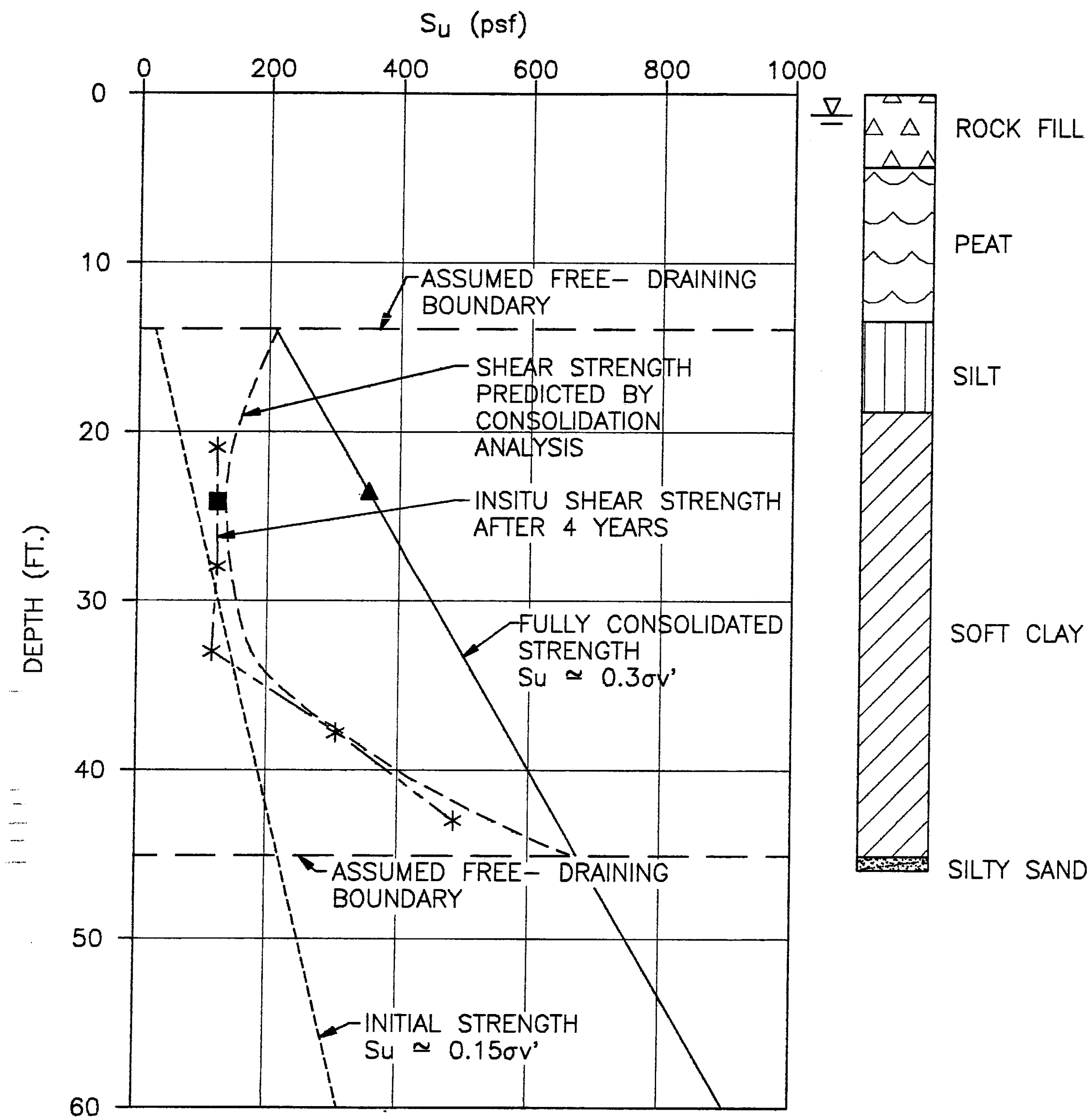


Figure 13. Laboratory Measurements of Cv for Normally Consolidated Clays



LEGEND:

- * CORRECTED VANE SHEAR STRENGTH
- ■ UNCONSOLIDATED – UNDRAINED TRIAXIAL TEST
- ▲ CONSOLIDATED – UNDRAINED TRIAXIAL TEST

Figure 14. Shear Strength gain with Time at Site in Northwestern Ontario

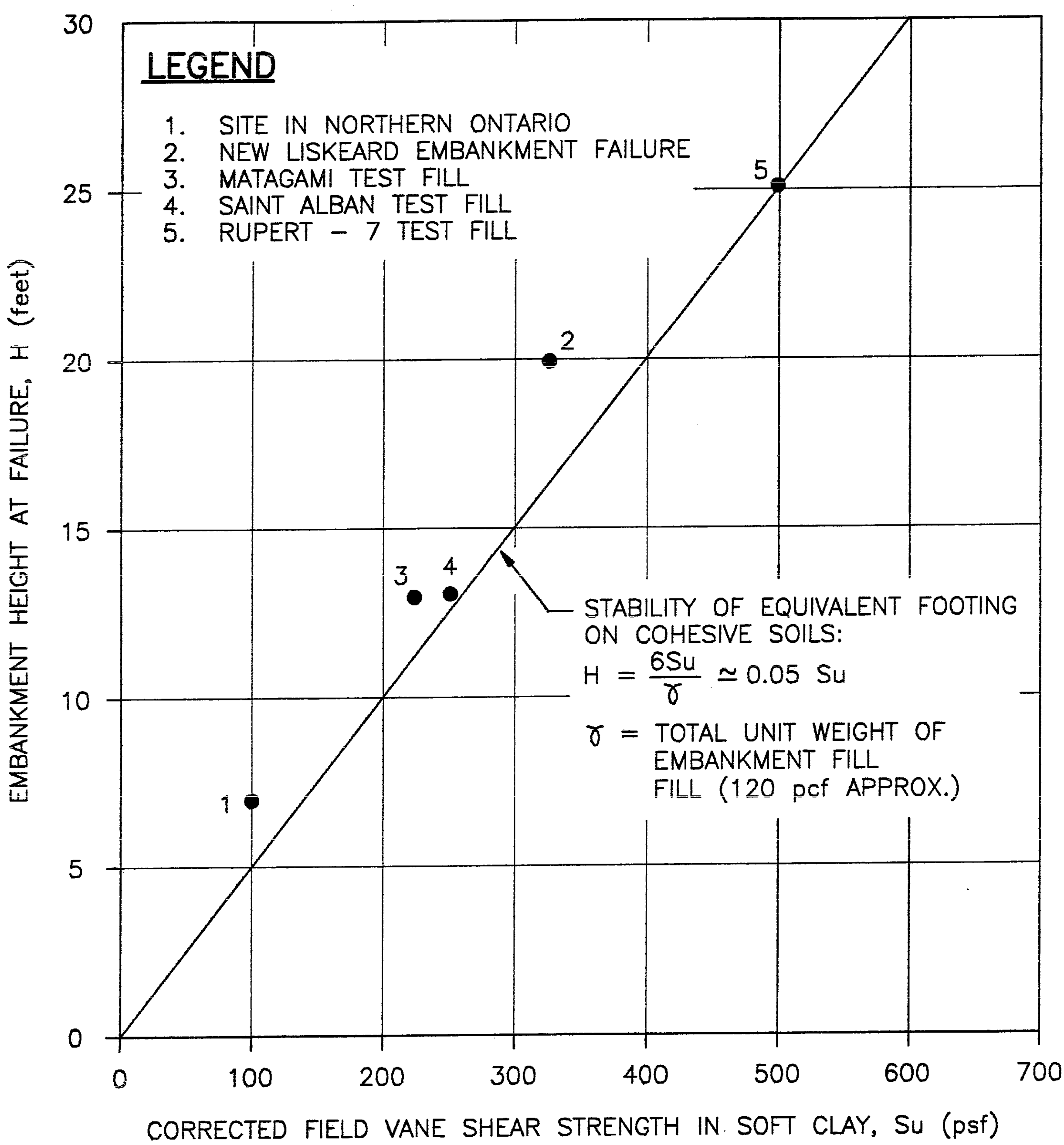
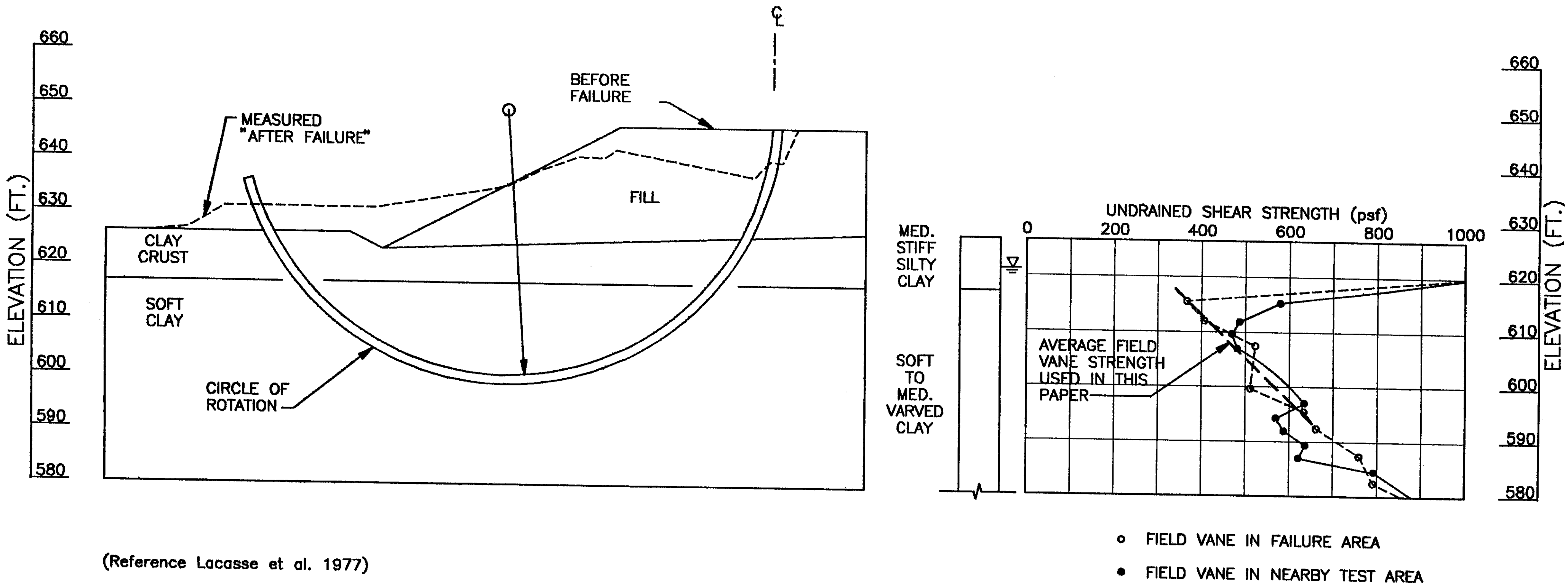


Figure 15. Empirical relationship for Estimating Height of Embankments at Failure.



(Reference Lacasse et al. 1977)

Figure 16. Embankment Failure on New Liskeard Varved Clay