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16. Abstract
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Control of Embankment Settlement Field Verification on PCPT Prediction Methods

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ABSTRACT

Piezocone penetration tests (PCPT) have been widely used by geotechnical engineers for subsurface investigation and evaluation of different soil properties such as strength and deformation characteristics of the soil. This report focuses on the verification of the PCPT settlement prediction methods for estimating the magnitude and time-rate of consolidation settlement of embankments over fine-grained soils. The settlement prediction methods involve the interpretation of piezocone penetration soundings and dissipation tests to determine the consolidation parameters, which include constrained modulus (*M*), overconsolidation ratio (OCR), and the horizontal and vertical coefficients of consolidation (c_h ,c_v). This Louisiana Transportation Research Center (LTRC) research team selected two case study sites, Juban Road Interchange Bridge at I-12 and Bayou Courtableau Bridge, to verify the PCPT predicted magnitude and time-rate of settlement. The embankments at each site were instrumented with horizontal inclinometers and vertical extensometers to monitor/measure their settlement with time. Both conventional one-dimensional consolidation tests and PCPT tests were performed to determine the consolidation parameters needed to calculate the magnitude and time-rate of consolidation settlements. The predicted magnitude and time-rate of consolidation settlements estimated using the laboratory one-dimensional consolidation tests and the PCPT tests were compared with field measurements. The results of this study showed that the piezocone penetration and dissipation data can reasonably estimate the magnitude and rate of consolidation settlement within the same range of accuracy as of the laboratory calculation. Friendly, visual basic software (Louisiana Embankment Settlement Prediction Program from PCPT, LESPP-PCPT) was also developed to calculate the magnitude and time-rate of consolidation settlements for symmetrical and unsymmetrical embankments utilizing the PCPT and dissipation tests for use by geotechnical engineers.

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IMPLEMENTATION STATEMENT

The results of the two case studies presented in this report showed that the PCPT based prediction of magnitude and time-rate of consolidation settlement of embankments are in close agreement with the field measurements. The PCPT method satisfactorily predicted the settlement of four embankments monitored in two sites. This demonstrates that the PCPT method can reasonably estimate the magnitude and time-rate of consolidation settlement of embankments. It is noted that the predicted magnitude of settlement is more reliable than the predicted time-rate of settlement. The study also showed that the PCPT interpreted consolidation parameters were in close agreement to the laboratory derived and those back-calculated from field measurements.

The increasing use of the cone penetration soundings at LADOTD will eventually make the estimation of the magnitude and time-rate of settlements easier, faster, cheaper, and more reliable as compared to the more expensive and time-consuming sampling and subsequent laboratory testing of soil samples. In addition, in-situ PCPT tests can provide the data needed to estimate the parameters of soils that are difficult or near impossible to obtain using normal means. Based on the results of this study, it is recommended that LADOTD initiate implementing PCPT technology to estimate the consolidation settlement of fine-grained soils, in conjunction with the traditional laboratory calculation of settlements. It is the researchers' recommendation that LADOTD engineers continue to compare the consolidation settlements predicted from the PCPT data to the calculated settlements from laboratory consolidation parameters and to the field measured settlements to gain experience and confidence of using PCPT for settlement estimation purposes. With increasing confidence and experience, LADOTD engineers can gradually move toward replacing the conventional subsurface exploration with piezocone penetration and dissipation tests for the estimation of consolidation settlement. It is anticipated that implementation of the PCPT methods will result in substantial cost and schedule benefits and an improvement in settlement prediction.

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INTRODUCTION

Subsurface saturated fine-grained soils subjected to loading can undergo significant consolidation settlements over a long period of time. The presence of this type of soil deposit is very common in southern Louisiana. Depending on the type of applied structure and embankment loadings, the magnitude and progression rate of consolidation settlement with time can significantly impact the safety and serviceability of infrastructures constructed on fine-grained soils. Therefore, the construction of embankments, bridge abutments, and other structures on soft Louisiana soils requires a reasonable estimate of the magnitude and time-rate of consolidation settlement of the natural subsurface soil deposits in order to conduct a rational and safe analysis and design of these structures. This indirectly implies a better and more accurate evaluation of the consolidation parameters of the subsurface soil layers.

Reinforced concrete approach slabs are commonly used in Louisiana to connect bridge decks to adjacent pavements to provide a smooth transition. Motorists often complain about “bumps” when approaching or leaving such bridges. Such “bumps” are a result of differential settlement of the approach slabs. These slabs are typically supported by a deep foundation pile-supported bridge abutment at one end and by a strip footing on embankment fills at the other end. The consolidation settlement of the natural soil underneath the embankment can cause excessive differential settlement between the embankment fill beneath the approach slab footing and the bridge deck, creating two possible “bump” problems: faulting at the connection between approach slab and flexible pavement and/or a sudden change in the slope of the approach slab at the end of the bridge deck. The “bump” can cause uncomfortable and unsafe ride for motorists, damage to bridge decks, and costly frequent maintenance. In an attempt to solve this problem, LADOTD’s current practice is to preload the embankment site for a period of time prior to the construction of approach slabs and pavements. Additional surcharge load and installation of vertical drains are sometimes used to accelerate the settlement. The task of LADOTD engineers is to reasonably estimate the magnitude and time-rate of consolidation settlement. This estimation requires a more accurate evaluation of the consolidation parameters of subsurface soils.

Calculation of embankment settlement is usually made using Terzaghi’s consolidation theory based on soil parameters derived from laboratory tests. Laboratory tests such as the oedometer consolidation test are usually conducted on small, assumed undisturbed samples recovered from the embankment sites at various depths. Inevitably, almost all samples are usually subjected to some degree of disturbance, which makes the consolidation parameters determined from laboratory tests not truly representative of the actual in-situ conditions. Moreover, laboratory testing on samples obtained from interbedded soils or soils containing fissures can be misleading. Additionally, profiling the consolidation characteristics from laboratory tests on small samples taken from different depths can easily miss significant thin drainage layers *[1]*.

In contrast to laboratory tests, in-situ tests such as the PCPT can provide comparable results in evaluating the actual strength and consolidation properties of the subsurface soils under in-situ stress and drainage conditions. The PCPT has gained wide popularity and acceptance for subsurface investigation and soil characterization. It is a robust, fast, and economical in-situ test device that can provide continuous soundings of subsurface soil with depth. The piezocone penetrometer is capable of measuring the cone tip resistance (q_c), sleeve friction (f_s), and pore pressures (u) at different locations, depending on the location of transducer (at the cone tip = u_1, behind the cone base = u_2). These measurements can be effectively used for soil identification and evaluation of different soil properties such as the consolidation characteristics of cohesive soil.

The constrained modulus (M) and the coefficients of consolidation (c_v or c_h) are the two parameters needed for calculation of magnitude and time-rate of consolidation settlement from PCPT data. There are several interpretation methods available in literature to estimate constrained modulus from PCPT data, which were based on direct correlations with the laboratory-measured constrained modulus *[2], [3], [4], [5], [6], [7], [8]*. Many interpretation methods were also available for estimating the horizontal coefficient of consolidation (c_h) of cohesive soils by analyzing the piezocone dissipation test curves *[9], [10], [11], [12], [13]*. Some of these methods rely on estimating the time for 50 percent dissipation (t_{50}) (e.g., *[9], [10], [13], [14]*), some on the gradient of initial linear dissipation (e.g., *[12]*), and others on the rate of dissipation at a given dissipation level (e.g., *[10]*). The rigidity of the soil ($I_r = G/s_u$) was included in some methods, where G is the shear modulus and s_u is the undrained shear strength of the soil *[13]*. The vertical coefficient of consolidation (c_v) can then be calculated from c_h using the relation suggested by Levadoux and Baligh, which is based on the ratio of the vertical to the horizontal coefficients of hydraulic conductivity (k_v/k_h) of the soils *[11].* Since the deformation characteristics of soils are highly dependent on the stress history represented by the OCR, it is therefore necessary to profile the OCR with depth for a proper selection of the relevant soil parameters to estimate the consolidation settlement. Several correlation methods were proposed to evaluate the OCR from the PCPT data. These methods are based on either the undrained shear strength (s_u) or directly from the PCPT profile using either tip resistance (q_c) or pore pressure (u) *[5]*, *[8], [15], [16], [17]*.

Several case studies were reported in literature to estimate the consolidation settlement of subsurface soils using parameters derived from the PCPT data *[8], [18]*, *[19]*, *[20]*, *[21]*.

Oakley and Richard used the cone penetration test (CPT) data to calculate the settlement of a chemically stabilized landfill *[18]*. They reported reasonable agreement between the calculated settlement from PCPT data and the actual measured settlement, while the time-rates of settlement were within ± 50 percent of the actual field measurements. Crawford and Campanella compared the measured settlements of earth embankment with settlements calculated from the laboratory consolidation test, PCPT test, and dilatometer test *[19]*. They found that there was a good agreement among the three methods, but the actual settlement was about 60 percent greater than the average calculated value. The calculated rates of settlement were also compared with observed values. Kuo-Hsia et al. compared the CPT-predicted settlement with the measured settlement of an instrumented test embankment *[20]*. They reported that the CPT is the most valuable basis for evaluating the constrained moduli and calculating the total settlement of soft soils. Abu-Farsakh et al. compared the magnitude and time-rate of consolidation settlements estimated using PCPT data and laboratory consolidation parameters with the field measurements at three different sites *[21]*. They demonstrated that the PCPT gave a better settlement prediction than that estimated from laboratory oedometer tests.

A previous research project was conducted at LTRC to evaluate the current PCPT interpretation methods for their capability to reasonably predict the soils' consolidation parameters needed to calculate the magnitude and time-rate of consolidation settlement of cohesive soils as well as the OCR. The predicted consolidation parameters from the PCPT tests using the different interpretation methods were compared with the reference soil parameters obtained from the laboratory testing. The capabilities of the different methods were evaluated and new interpretation methods were also proposed. The findings of the research study by Abu-Farsakh showed that the Sanglerat method predicted M values better than the other methods *[2]*, *[8]*. However, in this method, there is a wide range of α factors to be determined from a wide range of q_c values given in a table, which is subjected to the user's judgment. The results also indicated that the current PCPT prediction methods overestimated the OCR by a factor ranging from 2.0 times to 4.27 times. Therefore, new correlations were developed to predict M and OCR, which require further verification from field measurements. Evaluating c_v (or c_h) values using dissipation tests showed wide variations between the predicted and measured values, which reflects the reliability of these methods to predict c_v. This finding is consistent with other reported comparisons and considered acceptable and within the range of variations of laboratory-calculated c_v values *[22]*. As a result, it is not clear whether this scatter is due to the variations in PCPT dissipations or from laboratory variations. The predicted consolidation settlements from the PCPT methods were compared with the field measured settlement at three selected sites. The findings of this research were presented in a report submitted to LTRC *[8]*.

In this study, two case study bridge embankment sites: Juban Road - I-12 Interchange Bridge and Bayou Courtableau Bridge - LA 103 were selected to verify the methods proposed in the previous research. The two embankment sides of each bridge site were instrumented and the settlements were monitored. Comprehensive laboratory and field testing programs were performed to enable settlement analyses by both laboratory and PCPT methods. The PCPT tests included both continuous piezocone penetration tests using u_1 and u_2 configurations and dissipation tests at pre-specified depths. From PCPT tests, the constrained moduli, *M*, were estimated using Abu-Farsakh's proposed equations and the Sanglerat equation, *[2]*, *[8]*. Horizontal and vertical coefficients of consolidation were estimated from dissipation tests using the Teh and Houlsby method *[12]*. Prediction of the OCR was also verified using the data from the Juban Road - I-12 Interchange Bridge site.

OBJECTIVE

The objectives of the proposed research study were to:

- Verify and calibrate the findings of a previous research study on estimating the magnitude and time-rate of consolidation settlement of soils from PCPT data.
- Compare the laboratory-calculated settlements under embankments with settlements predicted from in-situ PCPT data as well as measured field settlements.
- Develop a computer program to estimate the consolidation settlement of embankments from PCPT data or other inputs.

SCOPE

This research study focused on the verification of the PCPT based prediction methods for estimating the magnitude and time-rate of consolidation settlement of embankments using the findings from a previous research project. A comprehensive testing program including laboratory and field tests was conducted to estimate the soils' consolidation properties needed for settlement calculations. The laboratory testing included moisture content, Atterberg limits, soil density, oedometer tests, unconsolidated undrained (UU) triaxial tests, and k_o-consolidated undrained (CU) triaxial tests. The field testing program included PCPT tests, dissipation tests, and settlement monitoring. The PCPT and dissipation tests were conducted on each embankment site. The average PCPT measurements (q_c, u_1, and u_2) that correspond to the same depths of the extracted Shelby tube samples were calculated and used to predict the consolidation parameters (*M, OCR*) using the proposed PCPT interpretation methods. The dissipation tests (with u_1) were used to predict the horizontal and vertical coefficients of consolidation (c_h, c_v) at different penetration depths using the Teh and Houlsby interpretation method. The predicted consolidation parameters obtained from the interpretation methods were compared with the laboratory measured parameters from one-dimensional oedometer tests and back-calculated parameters from field measurements. The predicted magnitude and time-rate of embankments' settlements from PCPT and laboratory derived consolidation parameters were compared with actual field settlements measured using horizontal inclinometers and/or vertical magnet extensometers.

METHODOLOGY

Background

PCPT is similar to a standard friction cone penetrometer, except that it measures pore water pressure besides tip resistance and sleeve friction. The PCPT has been used to evaluate various soil engineering properties for decades. Popular methods for estimating the parameters needed for settlement calculation are introduced herein.

Settlement of Saturated Fine-grained Soil

The deformation characteristics of fine-grained soils can be calculated from deformation moduli such as the one-dimensional compression or constrained modulus (*M*) defined as:

$$M = \frac{\partial \sigma'}{\partial \varepsilon} = \frac{2.3(1+e)\sigma'_v}{C_c} = \frac{1}{m_v} \tag{1}$$

where, C_c is the compression index, e is the void ratio, σ'_v is the effective vertical stress, and m_v is the coefficient of volume compressibility.

The total magnitude of consolidation settlement (S_c) of fine-grained soils can then be estimated utilizing the PCPT data through evaluating the constrained modulus (*M*) using the following equation:

$$S_c = \sum H_i \frac{\Delta\sigma_i}{M_{avi}} \tag{2}$$

where, H_i is the thickness of the soil layer i, $\Delta\sigma_i$ is the induced stress in the mid height of layer i, M_{avi} is the average constrained modulus for a stress range from the effective vertical overburden stress in the mid height of layer i (σ'_{voi}) to $\sigma'_{voi} + \Delta\sigma_i$, given as *[3], [4]*:

$$M_{avi} = M_i \sqrt{\frac{\sigma'_{voi} + \Delta\sigma_i / 2}{\sigma'_{voi}}} \tag{3}$$

where, M_i is the constrained modulus estimated from the PCPT data. The time-rate of consolidation can be calculated using the vertical and horizontal coefficients of consolidation (c_v, c_h) that can be evaluated through interpreting the PCPT dissipation test curves with time as will be discussed in the following sections.

Interpretation of Constrained Modulus, *M*

The compressibility of the soil can be expressed by the constrained modulus (M), which varies with the effective stress (σ'_v) as described in the following expression *[23]*:

$$M = mp_a \left(\frac{\sigma'_v}{p_a} \right)^{1-b} \tag{4}$$

where, m is a dimensionless modulus number, p_a is a reference stress (2.1 ksf or 100 kPa), and b is a stress exponent (b = 1 for preconsolidated stress range, and b = 0 for normally consolidated stress range).

Several correlations have been proposed to estimate the constrained modulus (M) from either the cone tip resistance (q_c) or the corrected cone tip resistance (q_t). The corrected cone tip resistance (q_t) is given by:

$$q_t = q_c + u_2\ (1\text{-}a) \tag{5}$$

where, u_2 is the pore water pressure measured behind the cone base, and a is the cone area ratio, equals to 0.59 for both the 10 and 15 cm^2 piezocones used in this study. The general relationship for M can be expressed as follows:

$$M = \alpha\ .\ q_c \quad \text{or} \quad M = \alpha\ .\ q_t \tag{6}$$

Sanglerat proposed a correlation between the cone tip resistance (q_c) and the constrained modulus (M) and presented a comprehensive array of α values for different soil types with different cone tip resistance values *[2]*. Jones and Rust found that for South African alluvial clay, a value of $\alpha = 2.75 \pm 0.55$ can provide good correlation between M and q_c *[6]*. Senneset et al. correlated M to the corrected cone tip resistance (q_t). For silty soils, they obtained the following linear correlation *[3]*:

$$M = \begin{cases} 2\,q_t & \text{for } q_t \le 26.1\ tsf\ (2.5\ MPa) \\ 4\,q_t - 5 & \text{for } 2.5 < q_t < 52.2\ tsf\ (5\ MPa) \end{cases} \tag{7}$$

For clayey soils, Senneset et al. related the constrained modulus (M) to the net cone tip resistance, by a linear interpretation of the net cone tip resistance ($q_n = q_t - \sigma_{vo}$). They proposed the following relations *[4]*:

$$M_{pc} = \alpha_p\ .\ q_n = \alpha_p\ .\ (q_t - \sigma_{vo}) \qquad \text{For the pre-consolidation range} \tag{8}$$
$$M_{nc} = \alpha_n\ .\ q_n = \alpha_n\ .\ (q_t - \sigma_{vo}) \qquad \text{For normally consolidated range} \tag{9}$$

where, $\alpha_p = 10 \pm 5$, $\alpha_n = 6 \pm 2$ for most clays, and σ_{vo} is the total overburden stress.

Kulhawy and Mayne studied the relationship between M and the net cone tip resistance (q_t - σ_{vo}) for different soils and suggested the following relation *[5]*:

$$M = 8.25(q_t - \sigma_{vo}) \tag{10}$$

Even though these relations might correlate well in some cases, local experience is essential to develop a better correlation between the cone tip resistance and constrained modulus that reflects the local soil types with greater reliability. To examine the possibility for obtaining better correlations in Louisiana, a comprehensive study was conducted on data collected from seven sites to reasonably estimate the constrained modulus (M) needed to calculate the consolidation settlement of cohesive soils in Louisiana *[7], [8], [21]*. The following linear correlation was obtained between M and q_t:

$$M = 3.15\, q_t \quad (R^2 = 0.91) \tag{11}$$

And the following linear correlation was also obtained between M and ($q_t - \sigma_{vo}$):

$$M = 3.58\,(q_t - \sigma_{vo}) \qquad (R^2 = 0.88) \tag{12}$$

The two proposed correlations will be used in this study to calculate the consolidation settlement of embankment from PCPT data.

Interpretation of Coefficient of Consolidation

The flow and consolidation characteristics of cohesive soils can be evaluated using the vertical (and/or horizontal) coefficient of consolidation (c_v or c_h) and the hydraulic conductivity (k). The two parameters are related through the following equation:

$$c_v = k\frac{M}{\gamma_w} \tag{13}$$

The coefficients of consolidation (c_v or c_h) that are used to calculate the rate of soil settlement can be evaluated from the piezocone dissipation tests. The PCPT dissipation test is conducted by inserting the cone to the designated depth and recording the dissipation of excess pore pressure (Δu) with time. The difference between the penetration pore pressure (u) and the static equilibrium pore pressure (u_o) is called excess pore water pressure (Δu).

Several empirical, semi-empirical, and analytical interpretation methods have been developed to evaluate the horizontal coefficient of consolidation (c_h) of cohesive soils from analyzing the piezocone dissipation test data curves. These methods are based either on the cavity expansion theories, the strain path method, or a combination of the strain path method with the finite element technique *[1]*, *[12]*, *[14]*, *[18]*, *[24]*.

Torstensson proposed the following relation for the interpretation of the horizontal coefficient of consolidation (c_h) from piezocone dissipation tests *[13]*:

$$c_h(piezo) = \frac{T_{50} r_o^2}{t_{50}} \tag{14}$$

where, T_{50} is the time factor at 50 percent dissipation, (r_o) is penetrometer radius for cylindrical model or equivalent penetrometer radius for spherical model, and t_{50} is the time for 50 percent dissipation. A similar equation was proposed by Senneset et al. with the time factor given as a function of soil properties and degree of pore pressure dissipation ($\Delta u_t/\Delta u_i$) *[9]*.

Teh proposed a method to interpret the coefficient of consolidation from the square-root plot of pore pressure dissipation and calculate the gradient of the initial linear section (m). Next, $c_h(piezo)$ can be estimated using the following equation *[11]*:

$$c_h(piezo) = (m/M_G)^2 \sqrt{I_r}\ r_o^2 \tag{15}$$

where, M_G is a gradient of theoretical dissipation curve for a given penetrometer geometry and filter location, $I_r = G/s_u$ is the rigidity index, G is the shear modulus, and s_u is the undrained shear strength. The shear modulus at 50 percent of yield stress (G_{50}) is usually used, which represents an average of stress levels.

Senneset et al. also suggested an equation to estimate $c_h(piezo)$ from the dissipation rate diagram as follows *[9]*:

$$c_h(piezo) = \lambda_c r_o^2 \left| \Delta \dot{u}_t / \Delta u_i \right| \tag{16}$$

where λ_c is the rate factor, $\Delta \dot{u}_t$ is the rate of dissipation at a given dissipation level, and Δu_i is the initial excess pore pressure at $t = 0$. The rate factor is a function of soil properties and degree of pore pressure dissipation, $\Delta u_t/\Delta u_i$.

The most popular interpretation expression for estimating $c_h(piezo)$ utilizing the piezocone dissipation tests was developed by Teh and Houlsby based on the combination of strain path method with the large strain finite element analysis *[12]*. They proposed the following equation to estimate $c_h(piezo)$:

$$c_h(piezo) = (T_{50}^* r_o^2 \sqrt{I_r}) / t_{50} \tag{17}$$

where, T^*_{50} is a modified time factor at 50 percent dissipation (T^*_{50} = 0.118 for the u_1 piezocone and 0.245 for the u_2 piezocone).

Since the dissipation of pore pressure occurs during the recompression range rather than in the normal consolidation range, Levadoux and Baligh suggested that the predicted $c_h(piezo) = c_h\,(overconsolidated)$ and proposed the following relation to transfer $c_h(piezo)$ to normally consolidated condition $c_h(NC)$*[1]*:

$$c_{h(NC)} = \frac{RR}{CR}\ c_h(piezo) \tag{18a}$$

where,

$$RR = \frac{c_r}{1+e_o} \qquad \text{and} \qquad CR = \frac{c_c}{1+e_o} \tag{18b}$$

where, *RR* and *CR* are the modified compression index and the modified recompression index, respectively; c_r is the swelling index; c_c is the compression index; and e_o is the initial void ratio of the soil. The vertical coefficient of consolidation (c_v) can then be calculated using the ratio of vertical to horizontal coefficients of hydraulic conductivity (k_v/k_h) using the following expression suggested by Levadoux and Baligh *[10]*:

$$c_{v(NC)} = \frac{k_v}{k_h} c_{h(NC)} \tag{19}$$

In this study, the interpretation method proposed by Teh and Houlsby was used to evaluate the coefficients of consolidation. In this method, the proper estimation of c_h depends on the selection of an appropriate value of the rigidity index (I_r) and, thus, the value of the shear modulus (*G*). In this study, the undrained shear strength was obtained from UU triaxial tests. The shear modulus was determined from the k_o-anisotropic consolidated undrained (k_o-*CU*) triaxial tests conducted on samples obtained adjacent to PCPT dissipation tests.

Interpretation of OCR

The OCR, which is defined as the ratio of the maximum past effective consolidation stress and the existing effective overburden stress, represents the stress history of the soil deposit. The value of the OCR has an important effect on strength, stress-deformation, and the compressibility characteristics of the soil. Profiling the OCR is essential for the proper selection of relevant soil parameters for geotechnical designs. The OCR is based on the estimation of preconsolidation pressure (σ'_p) from the oedometer consolidation tests on undisturbed samples.

Several methods are available in literature attempting to evaluate the OCR from the PCPT data. These methods are based either on the undrained shear strength (s_u) or directly on the PCPT profile *[5]*, *[15]*, *[16]*, *[25]*, *[26]*. In the previous study, three different methods were selected to predict the OCR from PCPT data. These methods are summarized below:

Schmertmann suggested estimating the OCR based on the undrained shear strength (s_u) as follows *[16]*:

(a) Estimate s_u from CPT data;

(b) Compute the ratio S = (s_u / σ'_{vo});

(c) Estimate the corresponding normally consolidated value S_n = $(s_u / \sigma'_{vo})_{NC}$ from the plasticity index, I_p, and;

(d) Estimate the OCR by using a correlation chart or using the following relation *[27]*:

$$OCR = (S / S_n)^{1.13 + 0.04(S/S_n)} \tag{20}$$

Kulhawy and Mayne related the OCR with the normalized net cone resistance $(q_t - \sigma_{vo}) / \sigma'_{vo}$ and suggested the following equation to estimate the OCR from the PCPT data *[5]*:

$$OCR = k_t \left[(q_t - \sigma_{vo}) / \sigma'_{vo} \right] \tag{21}$$

with k_t = 0.33. This is similar to the relation obtained by Powell et al. based on a study on some clays in the United Kingdom, with k_t = 0.2 to 0.24, and Cai et al. based on a study on quaternary clays in China, with k_t = 0.37 to 0.45 *[28], [29]*. The value of k_t seems to be soil type and site dependent.

Using the effective stress approach, Chen and Mayne suggested the following simplified relation to estimate OCR from piezocones with pore pressure element located at the tip, u_1 *[15]*:

$$OCR = k_1 \left[(q_t - u_1) / \sigma'_{vo} \right] \tag{22}$$

where, k_1 = 0.81. However, for piezocones with pore pressure elements located at the base, u_2, they proposed the following expression to estimate OCR:

$$OCR = k_2 \left[(q_t - u_2) / \sigma'_{vo} \right] \tag{23}$$

where, k_2 = 0.46. A similar expression was also suggested by Konrad and Law to estimate OCR with k_2 = 0.49 *[25]*.

In a previous LTRC study, the measured OCRs were compared with $(q_t - u_1) / \sigma'_{vo}$ and $(q_t - \sigma_{vo}) / \sigma'_{vo}$ ratios *[8]*. A linear correlation was obtained between OCR and $(q_t - u_1) / \sigma'_{vo}$ as follows:

$$OCR = 0.161\ (q_t - u_1) / \sigma'_{vo} \quad , \qquad \text{with } R^2 = 0.91 \tag{24}$$

And the following linear correlation was also obtained between OCR and $(q_t - \sigma_{vo}) / \sigma'_{vo}$ as:

$$OCR = 0.152\ (q_t - \sigma_{vo}) / \sigma'_{vo} \quad , \qquad \text{with } R^2 = 0.90 \tag{25}$$

The arithmetic mean and standard deviation of the predicted to measured OCR (OCR_p/OCR_m) are 0.98 and 0.44 for the first correlation and 1.05 and 0.56 for the second correlation. The second correlation is closer to the findings of Powell et al. for non-fissured clay (with k_t = 0.2 to 0.24) than any the other available correlation *[28]*.

Guidelines for Settlement Calculation using PCPT and Dissipation Tests

This section describes the step-by-step procedures of calculating the magnitude and time-rate of embankment settlement using PCPT and the dissipation test.

Procedures of Calculating Magnitude and Time-rate of Embankment Settlement

1. Draw a cross section of the embankment and the underlaid soil strata.
2. Determine the effective calculation depth.
3. Divide the new embankment base line, i.e., along the natural ground surface or old embankment surface, into several equal-spaced intervals.
4. According to the soil profile and dissipation test locations, divide the soils profile to several layers.
5. For each point along the base of new embankment, calculate the depth of the midpoint of each soil layer; calculate the induced stress by the embankment fill at the midpoint using the solutions provided by Poulos and Davis *[30]*. Calculate the overburden effective stress at the midpoint of each soil layer.
6. Determine constrained modulus and vertical coefficient of consolidation at the midpoint of each layer from PCPT and dissipation test data using the procedures listed in the following section. Determine the average constrained modulus using equation (3).
7. Calculate the total settlement of soil layer using equation (2) for each layer. Combine the total settlement of each soil layer to get the total settlement at a chosen point of the new

embankment base.

8. Calculate the degree of consolidation at the specified time intervals using the following equations:

$$T = \frac{c_v t}{H_{dr}^2} \tag{26}$$

$$T = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \qquad \text{for U < 60\%} \tag{27}$$

$$T = 1.781 - 0.933\log(100 - U\%) \qquad \text{for U > 60\%} \tag{28}$$

where, T is the time factor and t is the chosen time for settlement calculation. H_{dr} is the length of the longest drainage path. U is the degree of consolidation.

9. Compute the settlement at the chosen time for each layer using the following equation:

$$S(t) = S_{total}U(t) \tag{29}$$

where, *S(t)* is the settlement at the chosen time t; S_{total} is the total settlement determined at step 7; *U(t)* is the degree of consolidation at chosen time *t*. Combine the settlement for each layer to get the settlement at time *t* for the chosen point at the new embankment base.

Procedures of Interpreting PCPT for Constrained Modulus

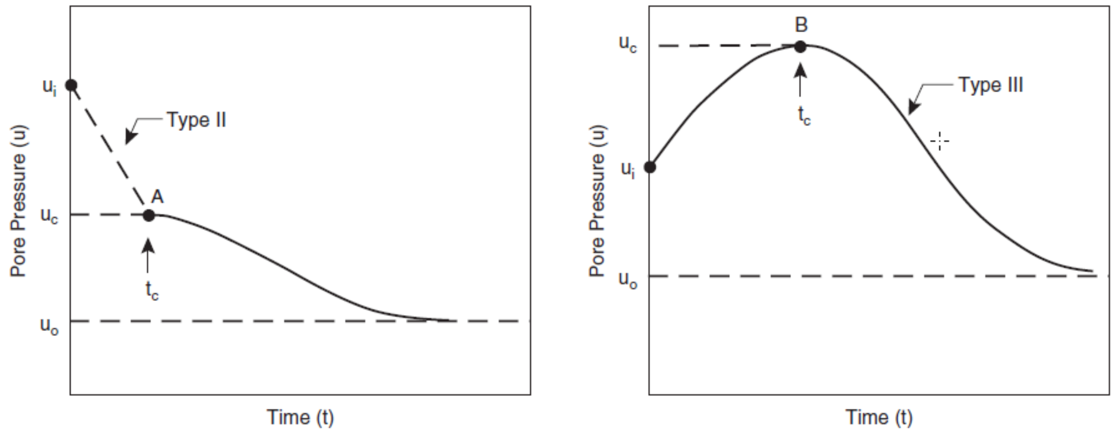
1. Compute the corrected cone tip resistance (q_t) using equation (5).
2. Compute the average q_t for each layer.
3. Compute the constrained modulus using equation (11) or (12) with q_t obtained in step 2.

Procedures of Interpreting Dissipation Tests for Vertical Coefficient of Consolidation

1. Normalize the excess pore water pressure in relative to the initial pore water pressure at the beginning of the dissipation (u_i) and equilibrium in-situ pore water pressure (u_o) as:

$$U = \frac{u_t - u_o}{u_i - u_o} \tag{30}$$

where, U is the normalized excess pore water pressure; and u_t is the excess pore water pressure at time *t*. Plot the normalized excess pore water pressure on a log time scale. According to the shape of the dissipation curve, correct the time and pore water pressure following the suggestions shown below in Figure 1. The time t_c is taken as the new 0 time, and the corresponding pore pressure is taken as the peak initial excess pore pressure for the dissipation curve.



(a) (b)

Figure 1

Correction of type II and III of dissipation curves (a: type II, b: type III)

2. From normalized dissipation curve, determine the 50 percent dissipation time t_{50}.
3. Determine the modified time factor (T^*_{50}) according to the type of pore water pressure measurement (T^*_{50} = 0.118 for the u_1 piezocone and 0.245 for the u_2 piezocone).
4. According to cone type, choose penetrometer radius r_o [r_o= 0.7 in. (1.784 cm) for the 1.55 in^2 (10 cm^2) piezocone penetrometer and r_o= 0.86 in. (2.185 cm) for the 2.33 in^2 (15 cm^2) piezocone penetrometer].
5. Determine the rigidity index ($I_r = G/s_u$) for each soil layer. The shear modulus and undrained shear strength can be estimated using equations (31) and (32), respectively.

$$G = \frac{M}{2}\frac{(1-2v)}{(1-v)} \tag{31}$$

$$s_u = \frac{q_t - \sigma_{vo}}{N_k} \tag{32}$$

where, M is the average constrained modulus estimated from PCPT for each soil layer, v is the Poisson's ratio, 0.4-0.5 is for saturated clay, 0.3 is for unsaturated clay, q_t is the average corrected tip resistance for each soil layer, σ_{vo} is the overburden soil pressure at the midpoint of soil layer, and N_k is empirical factor. A value of 15 is recommended for N_k for Louisiana soil.

6. Compute horizontal pizeocone coefficient of consolidation using equation (17) using the parameters estimated from the above steps.
7. Estimate the ratio of normal horizontal coefficient of consolidation to the piezocone coefficient of consolidation from the ratio of compression index to the recompression

index, which is usually in the range of 5-10.

8. Estimate the ratio of normal vertical coefficient of consolidation to horizontal coefficient of consolidation using equation (19). Jamiolkowski et al. suggested using k_v / k_h = 1 to 1.5 for no evidence of layering, k_v/k_h = 2 to 4 for slight layering, and k_v / k_h = 3 to 5 for varved clays and other deposits containing embedded and more or less continuous permeable layers *[31]*. From this ratio, compute the normal vertical coefficient of consolidation.

Laboratory and In-situ Tests

The main objective of this study was to evaluate the interpretation methods of a constrained modulus and a coefficient of consolidation proposed in the previous study for their ability to estimate the magnitude settlement and time-rate of consolidation settlement from the PCPT penetration and dissipation tests. To achieve this goal, two sites were selected: Juban Road I-12 Interchange Bridge site and Bayou Courtableau Bridge - LA 103 site. Laboratory tests and in-situ field tests were conducted to determine the consolidation parameters of subsurface fine-grained soils such as M, OCR, and c_v. This section describes the laboratory and field testing programs and the geotechnical conditions of the investigated sites.

Laboratory Tests

High quality 3-in. (7.6-cm) Shelby tube samples were retrieved from boreholes at different depths for comprehensive laboratory testing. Water content, unit weight, Atterberg limits, grain size distribution, and specific gravity were performed in accordance with ASTM standards D7263 - 09, D4318-10, D 422 – 63, and D854 – 10, respectively. One-dimensional consolidation tests were conducted in accordance with ASTM standard D 2435 – 04 to obtain the constrained modulus (M), vertical coefficient of consolidation (c_v), overconsolidation ratio (OCR), and the compression indices (c_c and c_r) of soils. UU and k_o-CU triaxial tests were also performed in accordance with ASTM standards D 2850 – 03a and D 4767 – 04, respectively, to estimate the undrained shear strength (s_u) and the shear modulus (G) of the soils. Some of the consolidation results for samples obtained from 12 ft. (3.66 m) to 15 ft. (4.57 m) deep at the Bayou Courtableau site are presented in Figures 2 through 4.

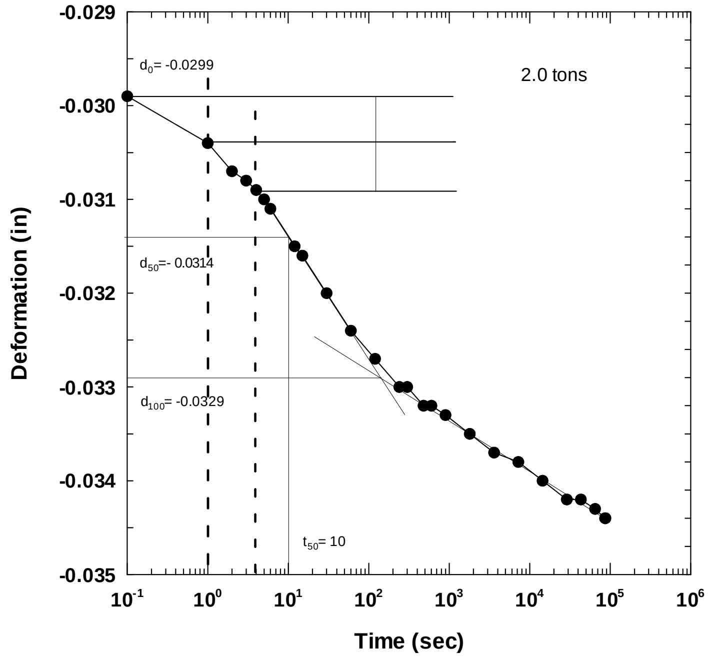


Figure 2

A sample result of one-dimensional consolidation test for east Bayou Courtableau Bridge - LA 103 site, depth = 12-15 ft. (3.66-4.57 m)

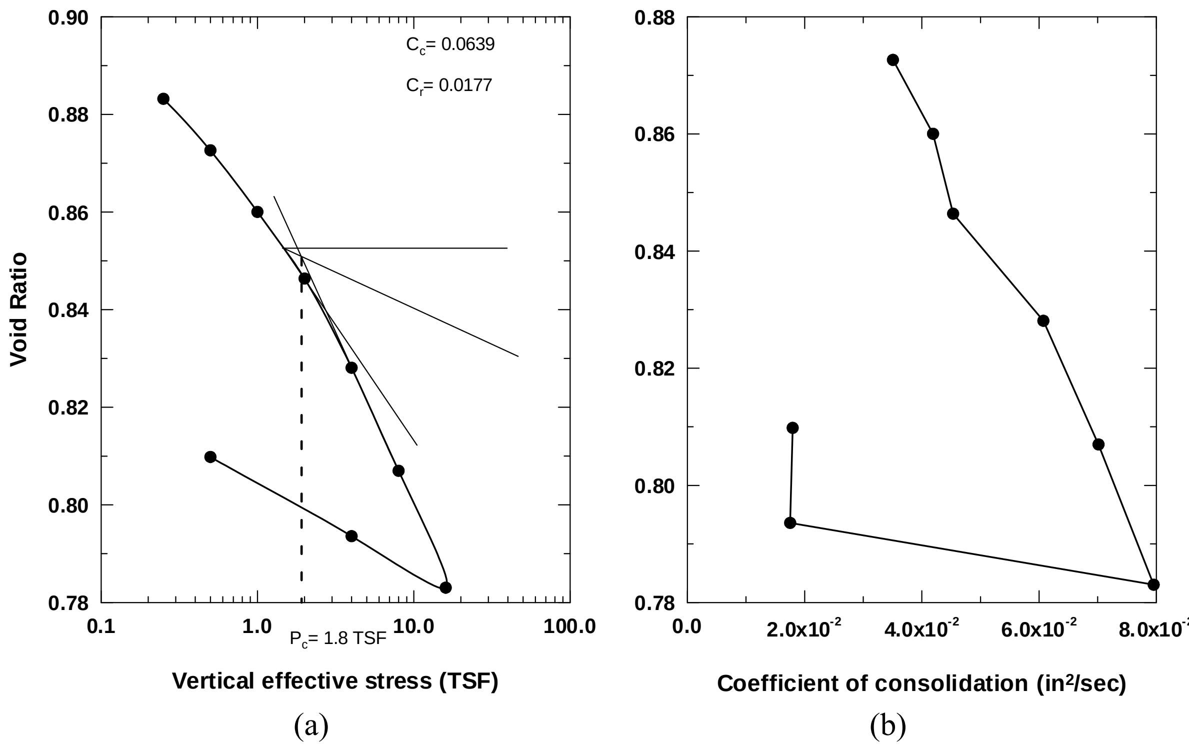
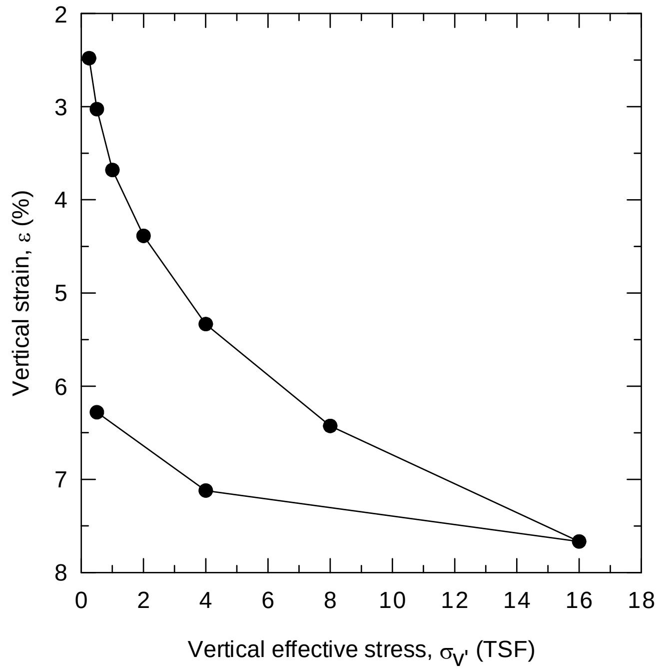
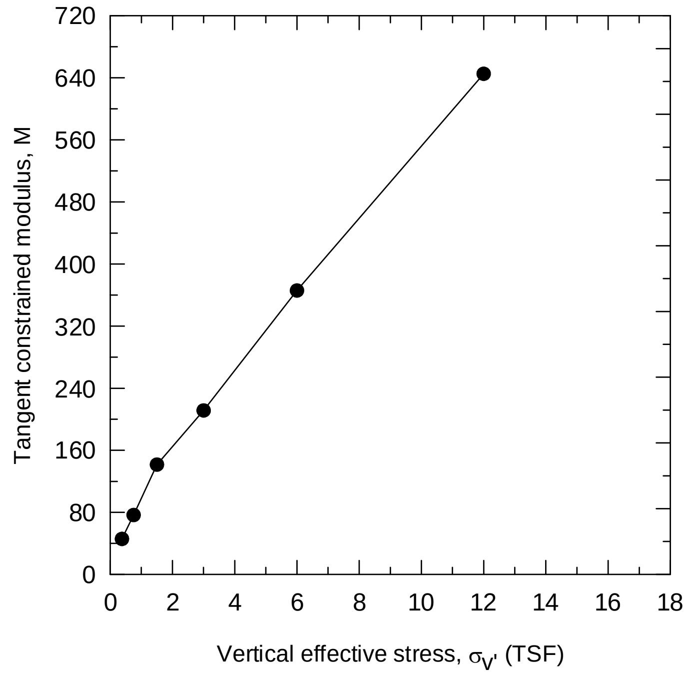


Figure 3

Sample consolidation test results for east Bayou Courtableau Bridge - LA 103 site, depth = 12-15 ft. (3.66-4.57 m); (a) vertical effective stress (σ_v') versus void ratio; (b) coefficient of consolidation (c_v) versus void ratio (e)



(a)



(b)

Figure 4
Consolidation test results for east Courtableau Bridge - LA 103 site, depth = 12-15 ft. (3.66-4.57 m); (a) vertical effective stress (σ_v') versus vertical strain; (b) σ_v' versus tangent constrained modulus (*M*)

In-situ Tests

The in-situ testing program included both PCPT and dissipation tests. The state-of-the-art cone penetration system, a 20-ton Research Vehicle for Geotechnical In-situ Testing and Support (REVEGITS) as shown by the left truck in Figure 5 was used to perform PCPT and dissipation tests. The REVEGITS is an in-situ test and support CPT system developed to acquire data for soil investigations, design, and analysis. The system consists of a hydraulic pushing and leveling system, 3.28-ft. (1-m) segmental rods, cone penetrometers [1.55 in^2 (10 cm^2) and 2.33 in^2 (15 cm^2)], and a data acquisition system.

Figure 5
Louisiana cone penetration systems: REVEGITS cone truck on the left and CIMCPT cone truck on the right

At each study site, several in-situ PCPT tests were performed around the drilled boreholes using the 1.55 in^2 (10 cm^2) and 2.33 in^2 (15 cm^2) piezocone penetrometers. The piezocones used in this study are Fugro subtraction-type cone penetrometers. The 1.55 in^2 (10 cm^2) piezocone has a sleeve area of 23.3 in^2 (150 cm^2) with a pore pressure transducer located 0.2 in. (5 mm) behind the base (u_2 configuration), while the 2.33 in^2 (15 cm^2) piezocone has a sleeve area of 31.0 in^2 (200 cm^2) with two pore pressure transducers located on the cone face and behind the sleeve (u_1 and u_3 configuration). The photos of the 1.55 in^2 (10 cm^2) and 2.33 in^2 (15 cm^2) piezocone penetrometers are depicted in Figure 6. During a PCPT test, the piezocone was pushed at the rate of 0.79 in/sec (2 cm/sec), and data was collected every 0.79 in. (2 cm). The 1.55 in^2 (10 cm^2) piezocone provided measurements of the cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure behind the base (u_2); while the 2.33 in^2 (15 cm^2) piezocone provided measurements of q_c, f_s, and pore water pressure at the cone tip (u_1). The profile of the PCPT tests was used to classify the soil using the probabilistic region estimation method to evaluate the undrained shear strength (s_u) and to evaluate the constrained modulus (M) using Abu-Farsakh et al. interpretation methods *[7], [8], [21]*.

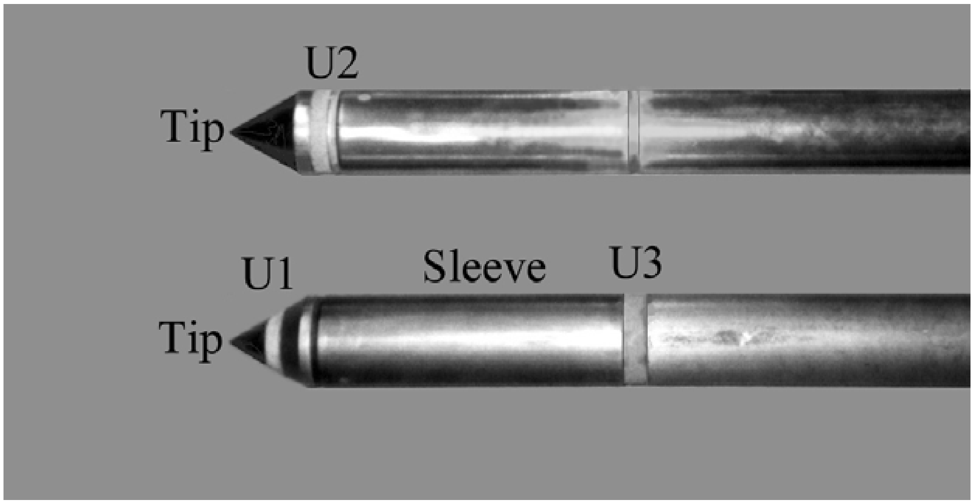


Figure 6
1.55 in^2 (10 cm^2) and 2.33 in^2 (15 cm^2)

The penetration of the piezocone was stopped at pre-specified penetration depths to perform dissipation tests with respect to time. The dissipation test curves were then used to estimate the horizontal and vertical coefficient of consolidation, c_h and c_v, respectively, based on the Teh and Houlsby interpretation method *[12]*.

Juban Road-I12 Embankment Site

Site Description and Instrumentation

The site includes two embankments, the north approach embankment and the south approach embankment, constructed for the Juban Road Interchange Bridge at Interstate I-12, located east of Baton Rouge in Livingston Parish. Figure 7 presents a typical cross section of the Juban Road embankment, which has a top width of 100 ft. (30.5 m) and a varied bottom width depending on the distances from the bridge end. The embankments in both sides of the bridge were instrumented with horizontal inclinometers and vertical extensometers to monitor the consolidation settlement with time.

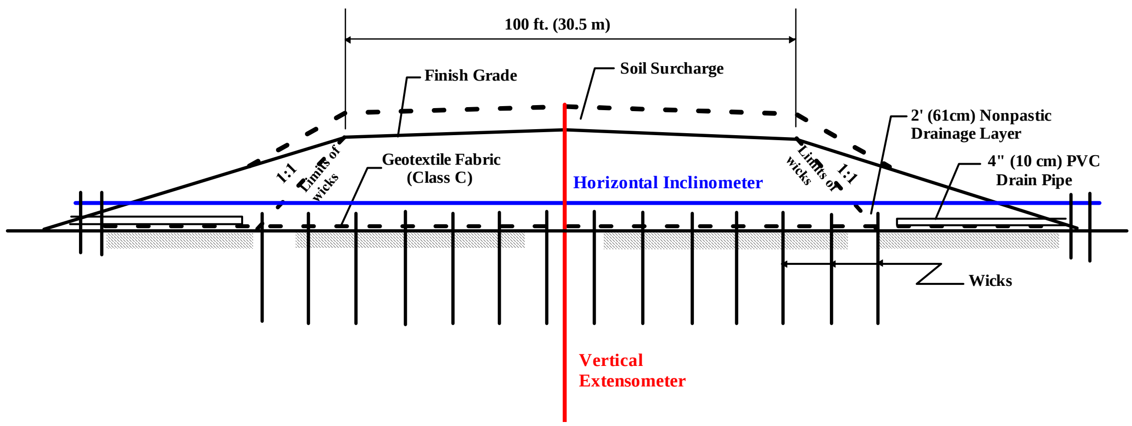


Figure 7
Typical embankment section at Juban Road – I12 Interchange

One horizontal inclinometer was installed in each embankment at a selected location to monitor the profile of consolidation settlement progress of the soil underneath each embankment. The digital horizontal inclinometer system was manufactured by RST Instruments Ltd. A 2 ft. (0.61 m) wide × 2 ft. (0.61 m) deep horizontal trench was first excavated across each embankment's width prior to the placement of any embankment fill. A 3.34-in. (85-mm) diameter inclinometer casing and a return pipe were installed along the trench (Figure 8). The trench was then filled with sand and compacted. Two wooden posts were used at each end of the inclinometer casings (extending 10 ft. (3 m) beyond the embankment) to secure its horizontal and vertical position. A settlement survey was conducted by drawing the inclinometer from one end of the casing to the other, halted in its travel at 2-ft. (0.61-m) intervals for inclination measurements. The elevations of posts relative to a fixed reference point were measured every time the inclinometer was used. The first survey was conducted after sand compaction to establish the initial profile of the casing (known as baseline survey). The subsequent surveys revealed the changes in the profile due to embankment settlement. The inclinometer readings were taken at specified time intervals until 12 months after the completion of the construction.

Figure 8
Horizontal inclinometer (left) and vertical magnet extensometers (right)

Juban Road North Embankment Site
The Juban Road – I-12 Bridge north embankment has an average top width of 100 ft. (30.5 m), bottom width of 316 ft. (96.3 m), and an average height of 29.4 ft. (8.96 m). A surcharge height of 3 ft. (0.91 m) and wick drains with a 6-ft. (1.83-m) triangular spacing and 41 ft. (12.5 m) deep were used to accelerate the consolidation settlement. The construction of north embankment fill including the surcharge was completed after 180 days. The surcharge was maintained for six months after construction.

Geotechnical Conditions. Two boreholes were drilled on the north embankment site and Shelby tube samples were recovered for laboratory testing. The subsurface soil stratigraphy as revealed from borings showed a top soil layer consisting of brown-gray, lean clay with traces of organics and/or sand down to a depth of 34.4 ft. (10.5 m). Soil below consisted mainly of sand interbedded with silty-clay layers to about 82 ft. (25 m). The groundwater level was about 6.56 ft. (2 m) below the ground surface at the time of the geotechnical exploration.

The results of laboratory tests on samples taken from the soil boring showed that the natural water content was close to the plastic limit with a mean value of about 25 percent. The undrained shear strength (s_u) varied from 2.5 psi (17 kPa) to 25.7 psi (177.5 kPa). The vertical coefficient of consolidation (c_v) determined from the one-dimensional consolidation tests was in the range of 2.1×10^{-5} in^2/sec (1.36×10^{-4} cm^2/sec) to 1.5×10^{-3} in^2/sec (9.6 x 10^{-}

[3] cm^2/sec). High OCR (>10) was observed in the top layer down to the depth of 6.56 ft. (2 m). Profiles of different soil properties including soil log, Atterberg limits, undrained shear strengths, constrained modulus, coefficients of consolidation, and OCRs are shown in Figure 9.

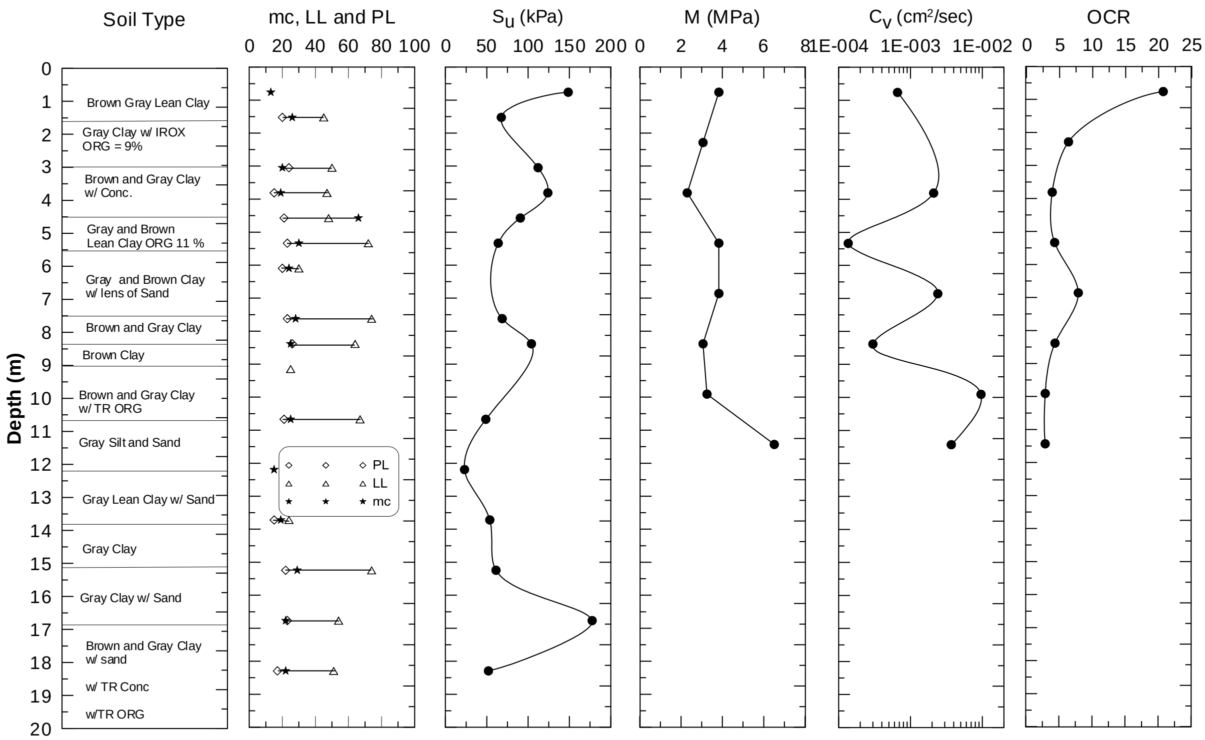


Figure 9
Soil boring profile for north embankment site

PCPT Tests. Four PCPT soundings with pore water pressure measurements using either u_1 or u_2 were conducted on the north embankment site down to a depth of 65.6 ft. (20 m). The profiles of PCPT test results and the corresponding CPT soil classification based on Zhang and Tumay method are presented in Figure 10 *[32]*. The soil profile consists of silty clay soils down to about 34.44 ft. (10.5 m) followed by sand soil down to the depth of penetration. Dissipation tests with u_1 measurements were conducted at depths of 6.99 ft. (2.13 m), 12.19 ft. (4.02 m), 19.81 ft. (6.04 m), 25.58 ft. (7.80 m), 25.94 ft. (7.91 m), 32.24 ft. (9.83 m), 35.45 ft. (10.81 m), and 36.11 ft. (11.01 m) below the ground surface. Figure 11(a) shows the results of dissipation tests for the Juban north embankment.

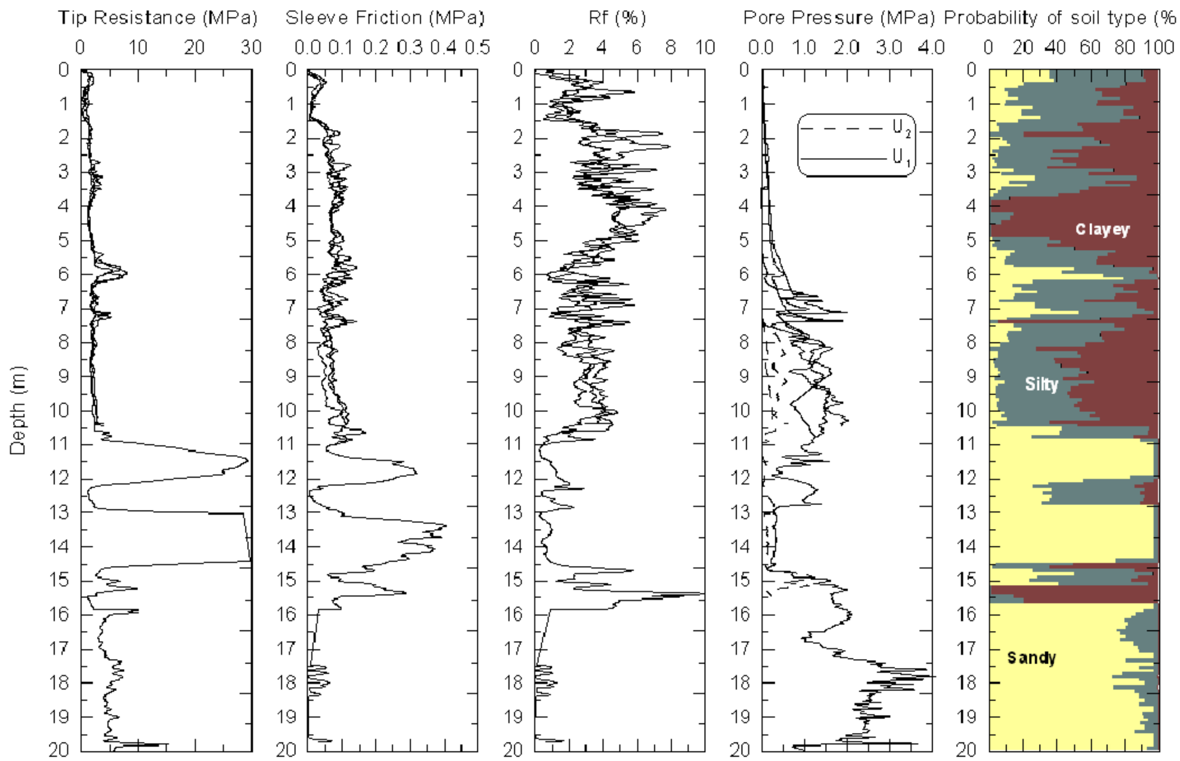
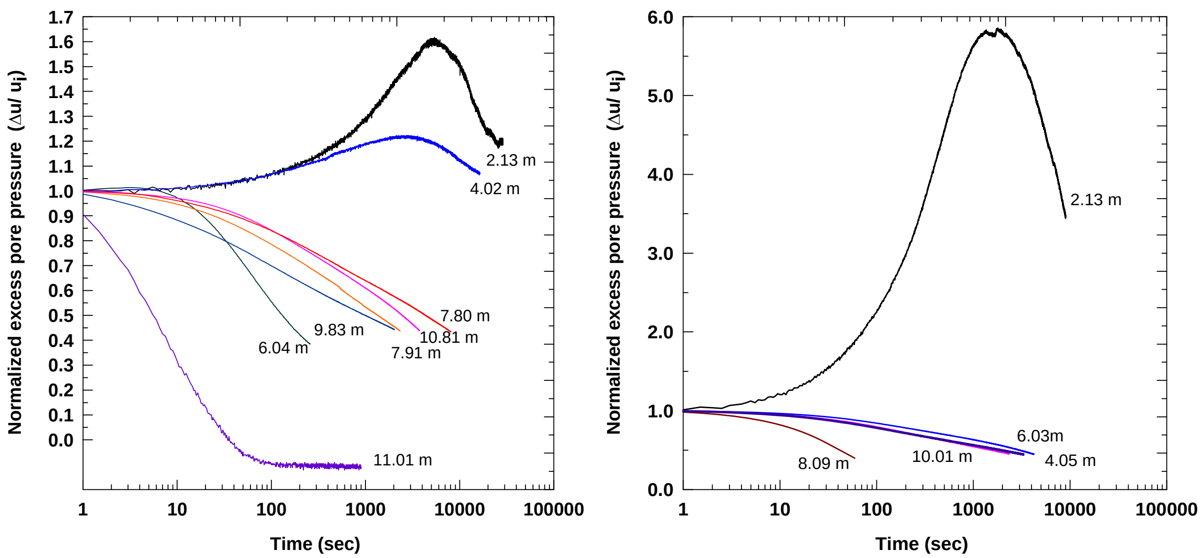


Figure 10
Soil boring profile for north embankment site



(a) North site (b) South site

Figure 11
Dissipation tests at Juban Road – I12 embankment sites

Juban Road South Embankment Site

The Juban Road - I-12 Bridge south embankment has the same top width of 100 ft. (30.5 m) as the north embankment. The bottom width is 274 ft. (83.52 m) with an average height of 26.5 ft. (8.08 m). A surcharge height of 3 ft. (0.91 m) and wick drains with a 6-ft. (1.83-m) triangular spacing and 41 ft. (12.5 m) deep were used to accelerate the consolidation settlement. The construction of south embankment fill including the surcharge took 140 days. The surcharge was maintained for six months after the completion of construction of the embankment fill.

Geotechnical Conditions. Two boreholes were also drilled at the south embankment site. The subsurface soil stratigraphy consists of grey or brown stiff lean clay down to 39.36 ft. (12 m) with occasional traces of organics. The soil between 39.36 ft. (12 m) and 55.76 ft. (17 m) depths consists of dense silty sand followed by brown or grey stiff clay and silty clay down to the depth of 65.6 ft. (20 m). The groundwater level was about 6.56 ft. (2 m) below the ground surface. The laboratory tests on the soil samples extracted from the site showed that the natural water content was close to the plastic limit with a mean value of 25 percent. The undrained shear strength, s_u, varied from 1.45 psi (10 kPa) to 19.87 psi (137 kPa). The vertical coefficient of consolidation as obtained from Oedometer test was in the range of 7.4 $\times 10^{-5}$ in^2/ sec (4.8 $\times 10^{-4}$ cm^2/ sec) to 5.0 $\times 10^{-4}$ in^2/ sec (3.2 $\times 10^{-3}$ cm^2/sec). High OCRs were also observed in the top layers down to the depth of 6.56 ft. (2 m). Figures 12 depicts profiles of soil description, Atterberg limits, undrained shear strength, constrained modulus, coefficients of consolidation, and OCRs.

PCPT Tests. Three PCPT tests were conducted at the south embankment site down to 65.6 ft. (20 m), two tests used u_1, and one test used u_2 measurements. The profiles of PCPT test results and the corresponding CPT soil classification are presented in Figure 13. Soil classification from the CPT measurements indicates that the soil profile consists of silty clay soils down to about 34.44 ft. (10.5 m) interbeded with thin sand layers. Two PCPT tests (with u_1 measurement) were selected for dissipation tests at different depths: 6.99 ft. (2.13 m), 13.28 ft. (4.05 m), 19.77 ft. (6.03 m), 26.54 ft. (8.09 m), and 32.83 ft. (10.01m), below the ground surface. Figure 11b depicts the results of dissipation tests.

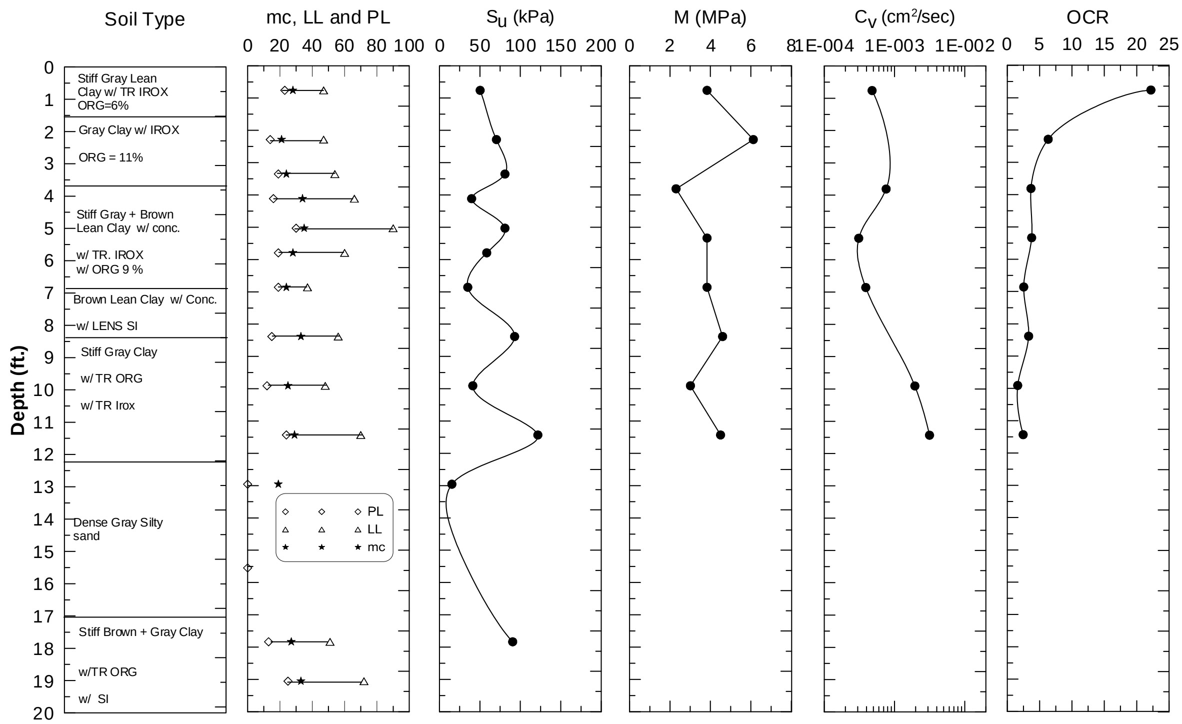


Figure 12
Soil boring profile for south embankment site

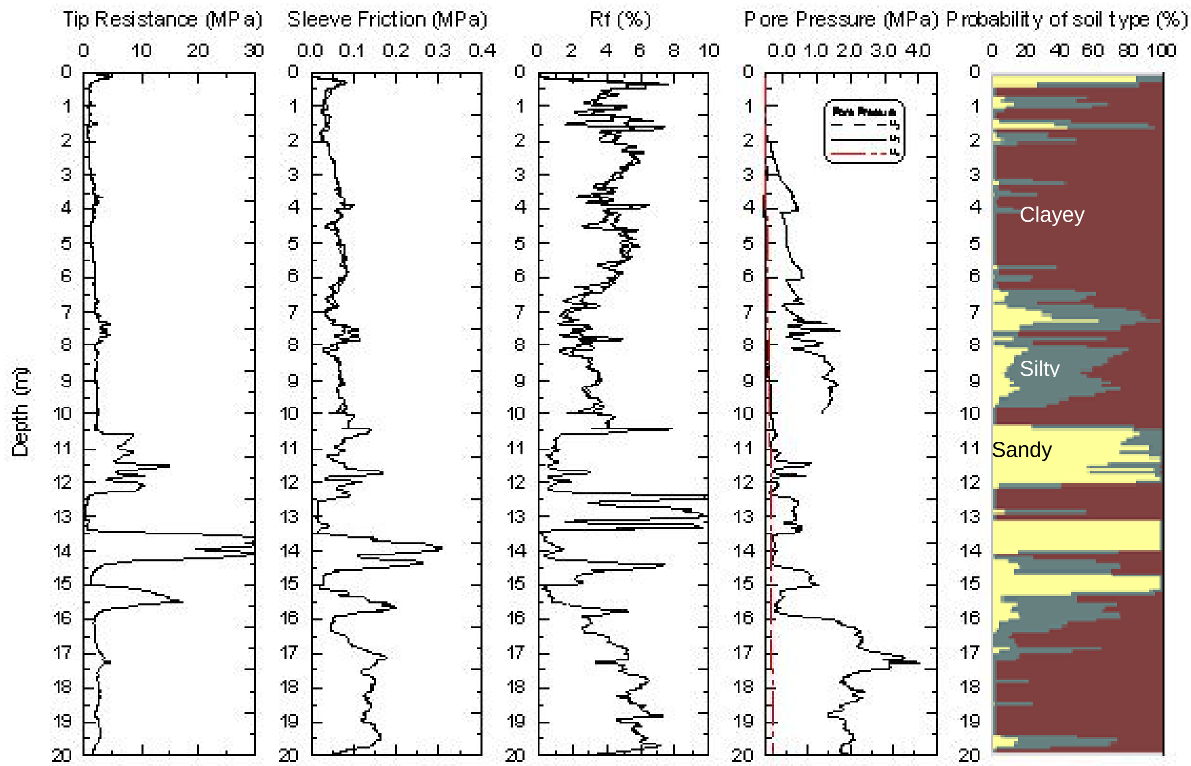


Figure 13
PCPT profiles and soil classification for south embankment site

Bayou Courtableau Bridge - LA 103 Site

Site Description and Instrumentation

LA DOTD decided to replace the old Bayou Courtableau Bridge - LA 103, as shown in Figure 14, located in St. Landry parish on LA route 103 about 8 miles north of Port Barre. The new bridge was built with precast prestressed concrete girders supported by concrete pile bents. One inclinometer pipe was installed at each of embankment at a selected location to monitor the profile of consolidation settlement of subgrade soil along the selected embankment cross section.

Figure 14
Old Bayou Courtableau Bridge - LA 103

The installation plan of inclinometers is shown in Figure 15. Similar to the Juban road site, a 2-ft. (0.6-m) wide and 2-ft. (0.6-m) deep trench as shown in Figure 16 was dug before construction of the first lift of embankment fill. An inclinometer casing with a diameter of 3.34 in. (85 mm) and a return pipe were then aligned on the bottom of the trench. The same inclinometer probe used in Juban road study was also used in this site. Each end of the inclinometer casing was extended about 10 ft. (3.05 m) beyond the embankment and was fastened to two wooden posts inserted into the natural ground to provide stable reference points for the later surveys. The first survey was conducted immediately after the trench backfill was compacted to obtain the baseline survey plus to ensure the function of the casing. The inclinometer probe was pulled through the casing twice with the probe in forward and backward position at a 2-ft. (0.6-m) interval, i.e., the length of probe. The two readings can eliminate possible instrument errors and provide an accurate measurement of the settlement profile. The inclinometer readings were taken at specified time intervals until 6 months after the completion of the construction. A survey of both pipe ends was taken at each site visit to provide corrections of elevation of the reference points.

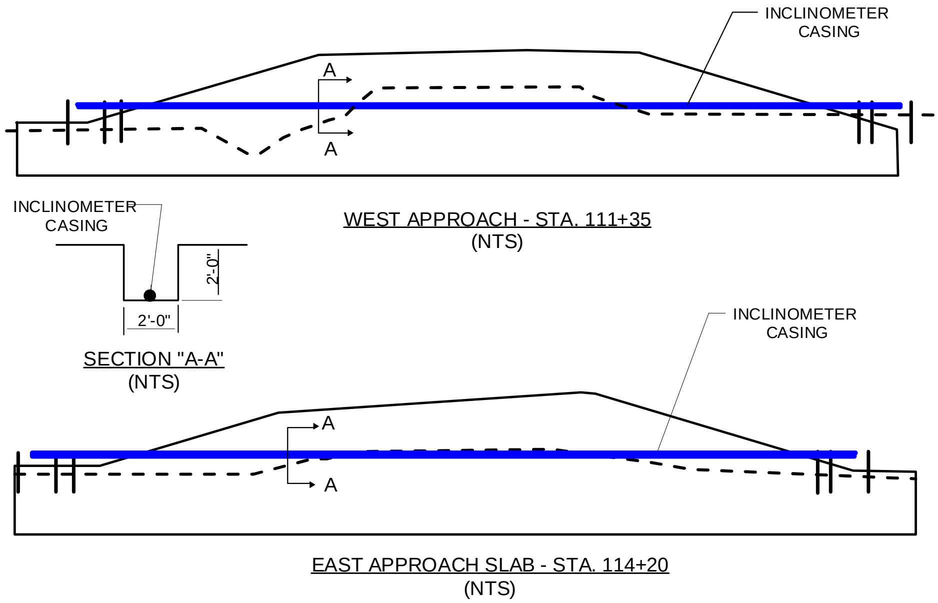


Figure 15
Installation plan of horizontal inclinometers

Figure 16
Installation of horizontal inclinometer casing and return pipe

Bayou Courtableau Bridge - LA 103, East Embankment

The newly constructed east approach embankment has a top width of 40 ft. (12.19 m), a height of 8 ft. (2.44 m) above the existing pavement, and a bottom width of 120 ft. (36.58 m). The construction of embankment fill was completed within one week.

Geotechnical Conditions. One borehole was drilled on the east embankment site. The subsurface soil stratigraphy as revealed from borings showed silty clay in the top 40 ft. (12.19 m) with interbedded sand layers. The results of one-dimensional consolidation tests along with soil strata from soil boring logs are presented in Figure 17. The soil water content at 49.2 ft. (15 m) to 6.56 ft. (20 m) was very close to the liquid limit water content. The undrained shear strength varied from 2.61 psi (18 kPa) to 25.67 psi (177 kPa). The vertical coefficient of consolidation was in the range of 4.5×10^{-5} in^2/ sec (2.92×10^{-4} cm^2/ sec) to 2.68×10^{-3} in^2/ sec (1.73×10^{-2} cm^2/sec). Low to moderate OCR was observed in the top 15 ft. (4.57 m). These parameters were used in the calculation of embankment settlement in a later section.

Selected sample stress-strain curves of UU and CU triaxial tests on retrieved Shelby tube samples at each site are shown in Figures 18 and 19.The confining pressures for UU tests and consolidation pressures for CU tests were the same as the in-situ stresses at which the soil samples were taken. The undrained shear strengths of subsurface soils were determined from the UU test results. The shear moduli of soils were estimated from k_o-CU test results.

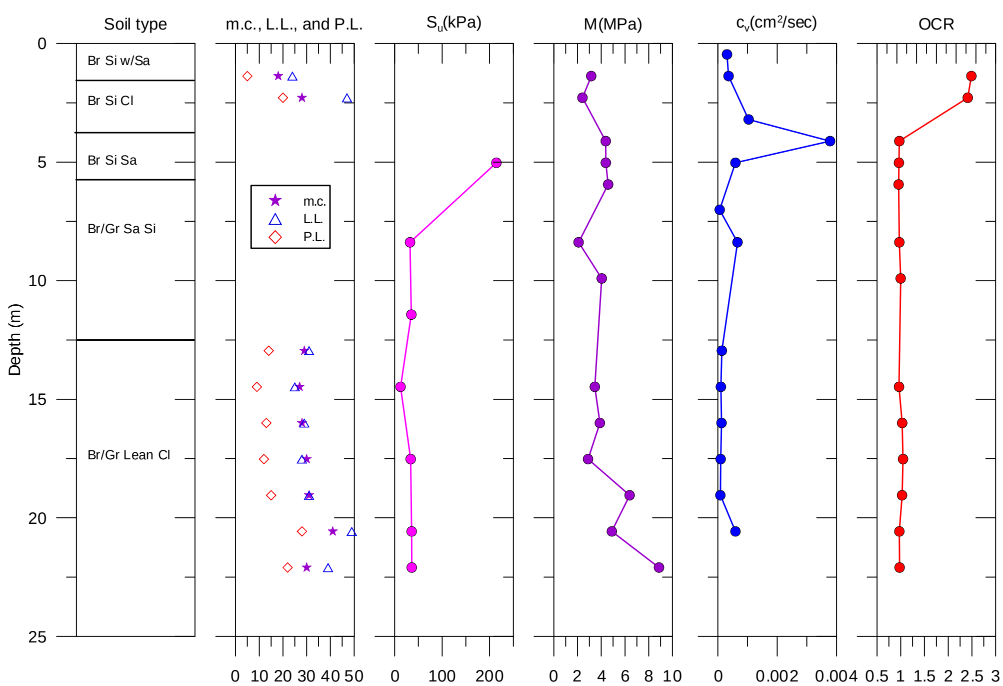


Figure 17
Profile of subsurface soil properties at east embankment side

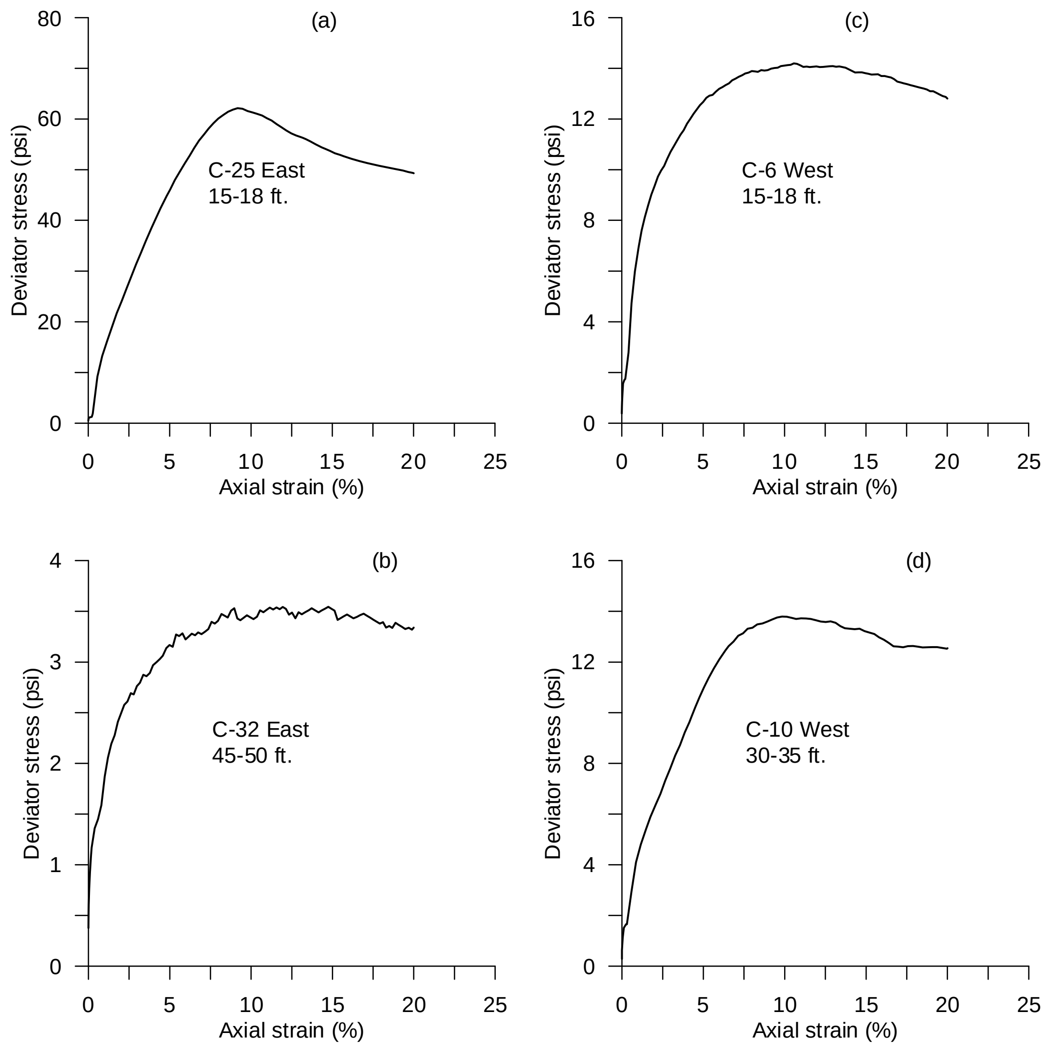


Figure 18
Undrained triaxial tests for Bayou Courtableau Bridge - LA 103, east and west sides

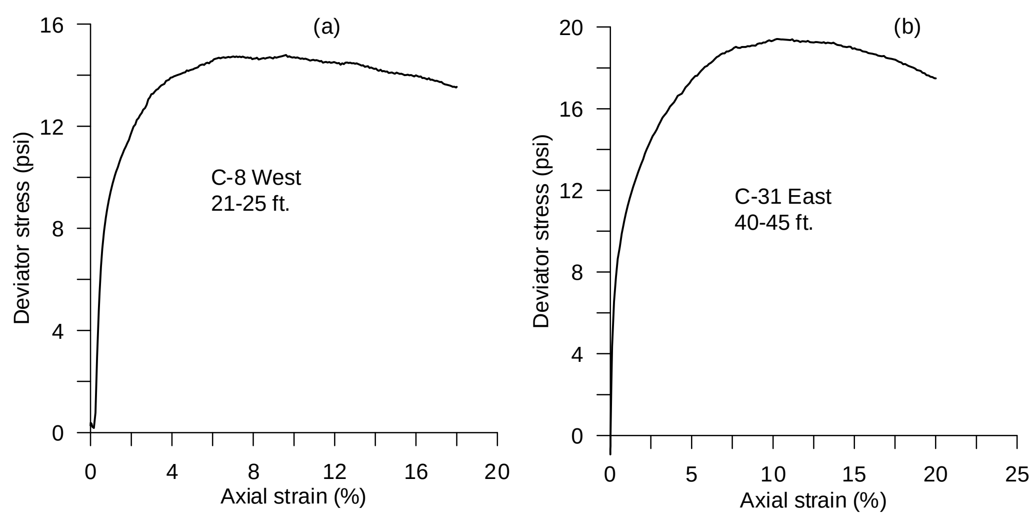


Figure 19
Consolidated undrained triaxial tests for Bayou Courtableau Bridge - LA 103

PCPT Tests. One PCPT test was conducted on the east embankment site of the bridge to profile the subsurface soils for estimation of the consolidation settlement. The profiles of CPT test results (q_t, f_s, and R_f) and the corresponding CPT soil classification using Zhang and Tumay method are presented in Figure 20 *[32]*. As shown by the CPT soil classification in the figures, the subsurface soil mainly consists of clayey silt in the upper 40 ft. (12.19 m) below ground surface. Compared to the west embankment site, the east site is sandier. Dissipation tests were conducted at each side of the bridge, as shown in Figure 21, to determine the coefficients of consolidation used for estimating the rate of consolidation settlement. The depths for the PCPT dissipation tests at the east side are: 28.26 ft. (8.62 m), 41.08 ft. (12.53 m), 55.81 ft. (17.01 m), 65.65 ft. (20.01 m), 68.97 ft. (21.03 m), 69.55 ft. (21.24 m), 71.35 ft. (21.75 m), and 72.63 ft. (22.14 m) below the old pavement surface.

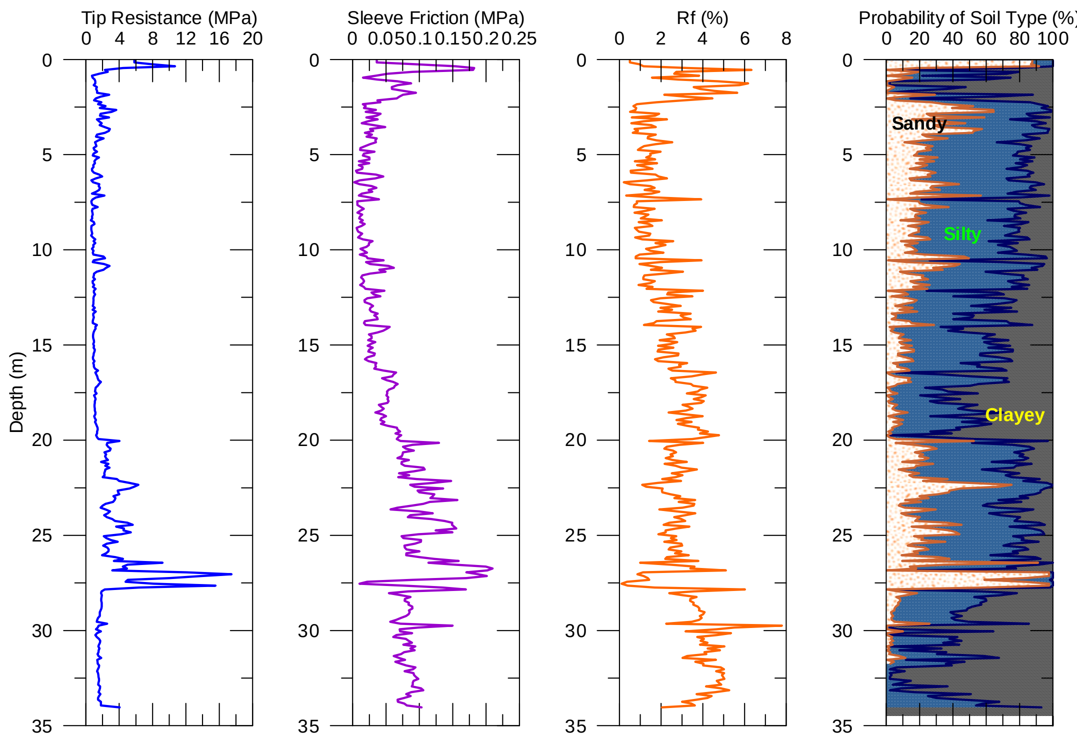


Figure 20
PCPT profiles and soil classification for Courtableau Bridge - LA 103 at east embankment side

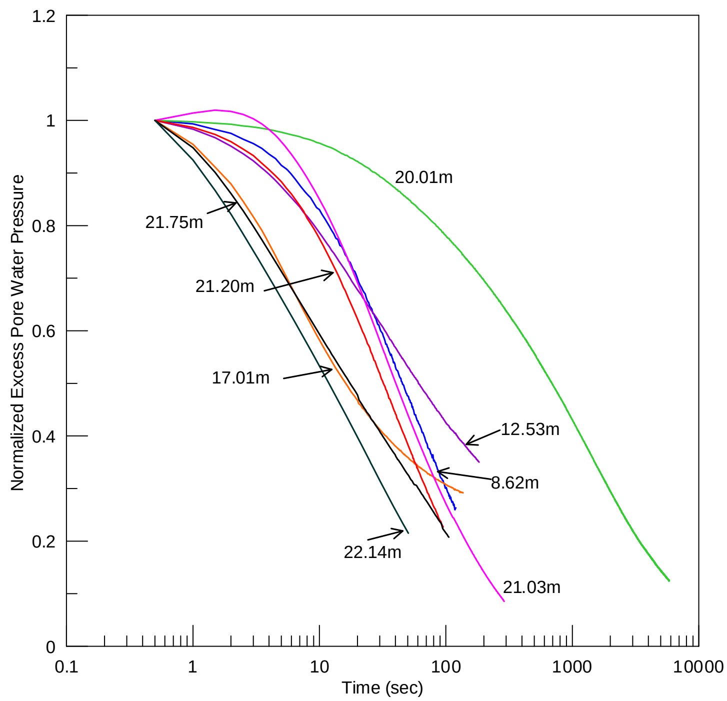


Figure 21
PCPT profiles and soil classification for Courtableau Bridge - LA 103 at east embankment side

Bayou Courtableau Bridge - LA 103, West Embankment
The newly constructed west approach embankment has a top width of 40 ft. (12.19 m), a height of 6 ft. (1.83 m) above the existing pavement, and a bottom width of 110 ft. (33.53 m). The construction of embankment fill was completed within one week. One inclinometer pipe was installed at the west embankment at a selected location to monitor the profile of the consolidation settlement of subsurface soil along the selected embankment cross section.

Geotechnical Conditions. One borehole was drilled on the west embankment site. The subsurface soil stratigraphy as revealed from borings showed silty clay in the top 80 ft. The results of one-dimensional consolidation tests along with soil strata from soil boring logs are presented in Figure 22. The undrained shear strength varied from 1.74 psi (12 kPa) to 31.03 psi (214 kPa). The vertical coefficient of consolidation was in the range of 2.67×10^{-5} in^2/ sec (1.72×10^{-4} cm^2/ sec) to 2.92×10^{-3} in^2/ sec (1.89×10^{-2} cm^2/sec). Low to moderate OCRs were observed in the top 15 ft. (4.57 m). These parameters were used in the calculation of embankment settlement in the later section.

The stress-strain curves of UU and CU triaxial tests on retrieved Shelby tube samples at each site are shown in Figures 18 and 19. The test results were used to determine the undrained shear strength and G_{50} of the soil, respectively.

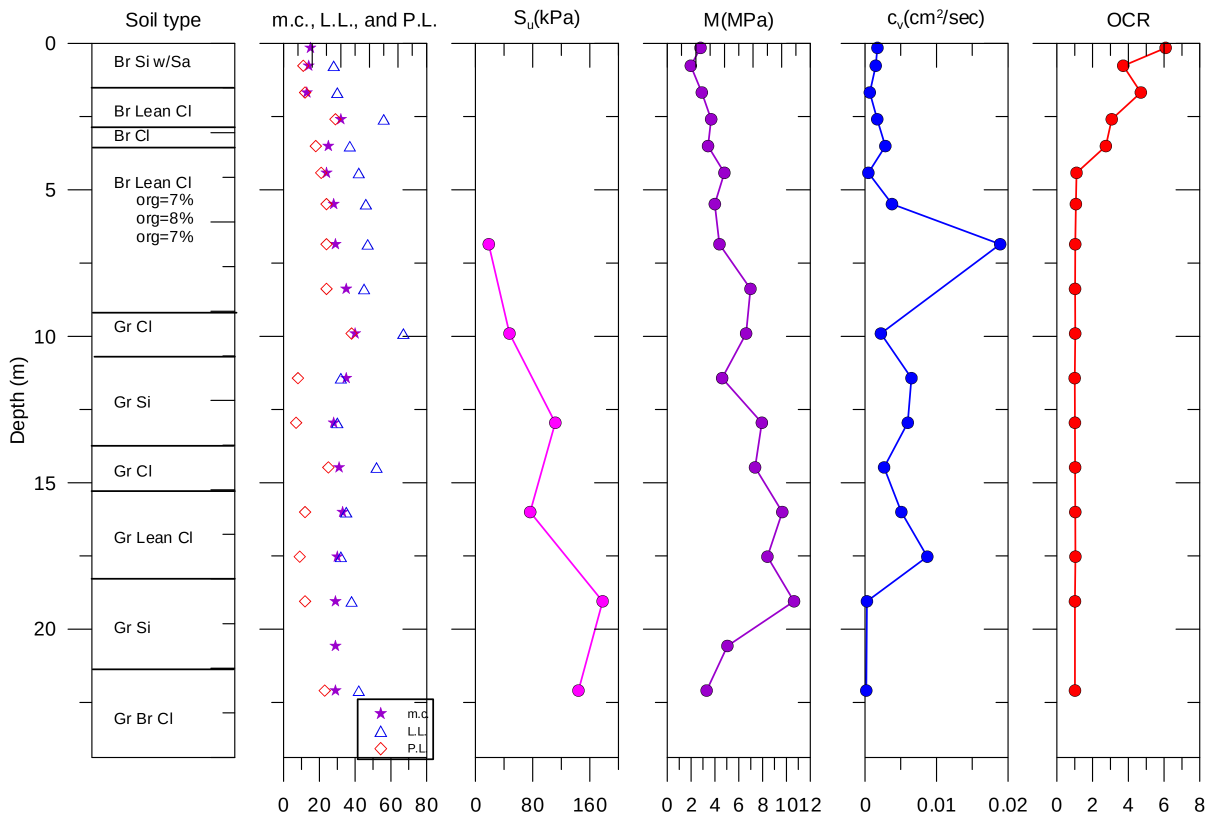


Figure 22
Profile of subsurface soil properties at west embankment side

PCPT Tests. One PCPT test was conducted on the west embankment site of the bridge to profile the subsurface soils for the estimation of the consolidation settlement. The profiles of CPT test results (q_t, f_s, and R_f) and the corresponding CPT soil classification using Zhang and Tumay method are presented in Figure 23 *[32]*. As shown by the CPT soil classification, the subsurface soil mainly consists of clayey silt in the upper 80 ft. (24.38 m) below ground surface. Dissipation tests were conducted at certain depths on the west side of the bridge, as shown in Figure 24, to estimate the coefficients of consolidation needed for evaluating the rate of consolidation settlement. The depths for the PCPT dissipation test at the west side are: 6.43 ft. (1.96 m), 8.66 ft. (2.64 m), 12.56 ft. (3.83 m), 24.93 ft. (7.6 m), 31.32 ft. (9.55 m), 35.49 ft. (10.82 m), 41.30 ft. (12.59 m), 45.53 ft. (13.88 m), 52.97 ft. (16.15 m), and 59.53 ft. (18.15 m) below the old pavement surface.

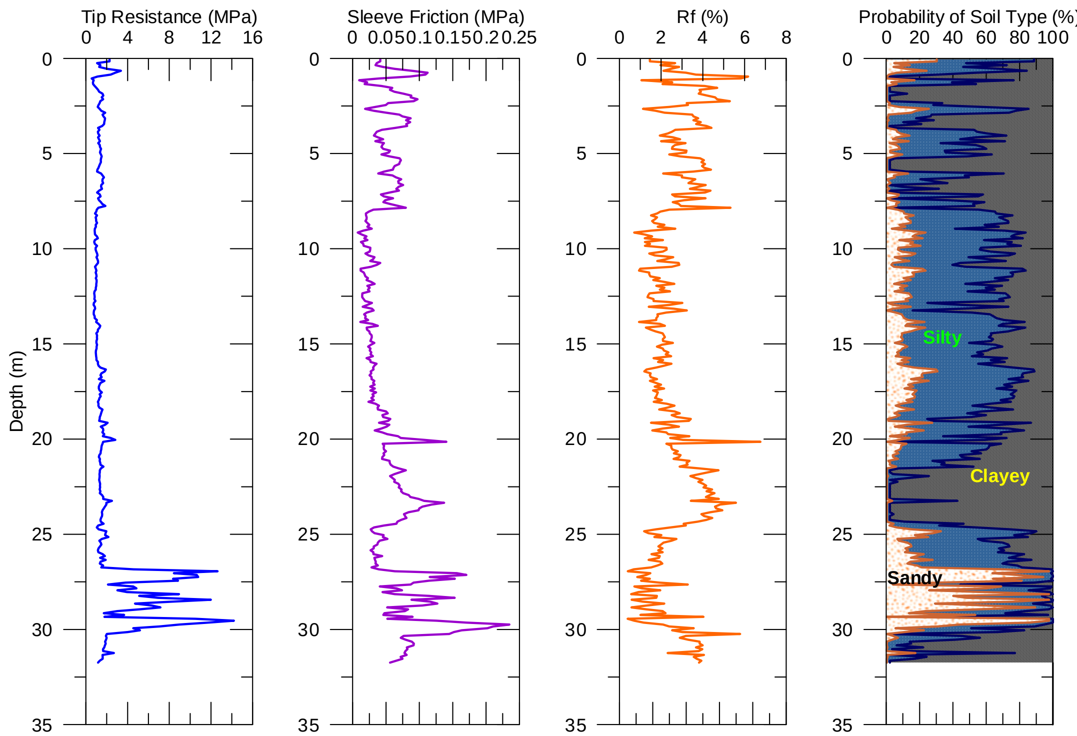


Figure 23
PCPT profiles and soil classification for Courtableau Bridge - LA 103 at west embankment side

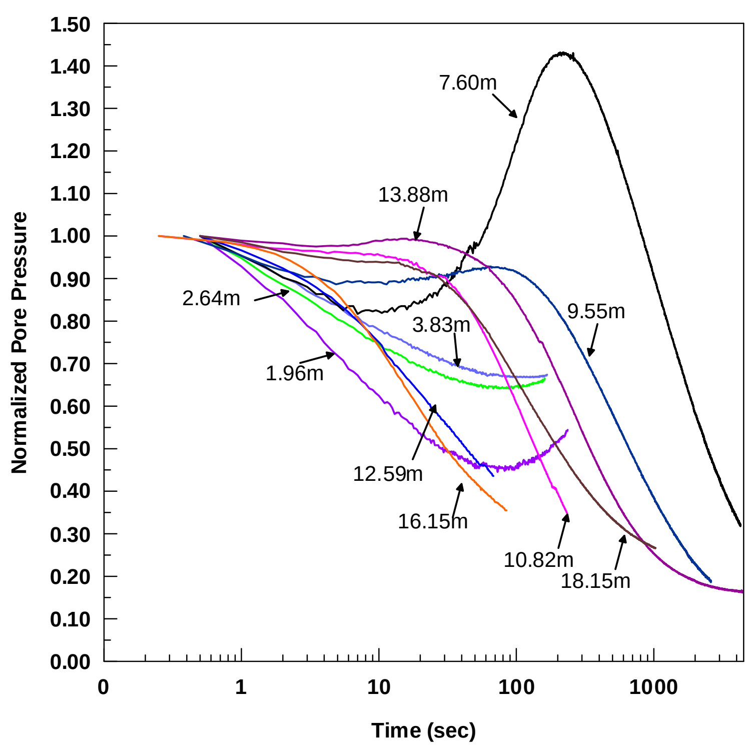


Figure 24
Dissipation test at Courtableau Bridge - LA 103 at west embankment side

DISCUSSION OF RESULTS

This section evaluates the settlements predicted using the PCPT interpretation methods proposed in the previous study. Data from PCPT penetration and dissipation tests were first utilized to evaluate the soils' consolidation parameters, i.e., the constrained modulus (*M*) and coefficients of consolidation (c_h and c_v) using different prediction methods. The PCPT predicted embankment settlements at Juban Road - I-12 Interchange Bridge and Bayou Courtableau Bridge - LA 103 sites were compared with the field settlements measured using horizontal inclinometers and vertical magnet extensometers.

Juban Road-I-12 Embankment Site

Comparison between PCPT and Laboratory Derived Parameters

The profiles of PCPT soundings and the results of piezocone dissipation tests were used to calculate the consolidation parameters: constrained modulus (*M*) and coefficients of consolidation (c_h and c_v) of subsurface soils for north and south embankment sides.

Constrained Modulus. The profiles of constrained modulus (*M*) at in-situ stresses were calculated from the PCPT using Abu-Farsakh methods based on q_t or q_n [equation (11) and (12)] *[8]*. The average value of total overburden pressure (σ_{vo}) needed for each soil layer to compute the q_n was estimated from the soil borings. The comparisons of predicted *M* values from PCPT versus the laboratory measured *M* are presented in Figure 25 for both north and south embankment sides. It is evident from the figure that the PCPT-*M* values for both sides are greater than the laboratory estimated *M* values. It is noted that this interpretation of constrained modulus only applies to cohesive soil. Therefore, sandy soil was left blank in the figure. Comparison with back-calculated values from field measurements for the south embankment side will be presented later.

Vertical Coefficient of Consolidation. The vertical coefficients of consolidation were calculated from the piezocone dissipation tests presented in Figure 11 using the Teh and Houlsby method *[12]*. This method requires the evaluation of t_{50} from the dissipation test curves [equations (17) and (19)]. To calculate the rigidity index ($I_r = G/s_u$), the undrained shear strength was estimated from the UU test, and the shear modulus (*G*) was determined from the k_o-*CU* triaxial tests.

The plots of c_v values estimated from dissipation tests and those derived from laboratory tests are presented in Figures 26 for the north and south embankments. Although the figure does not show good correlations between these two values, it is obvious that the subsurface cohesive soils at Juban Road site have a c_v value of about 1.6×10^{-4} in^2/sec (1×10^{-3} cm^2/sec).

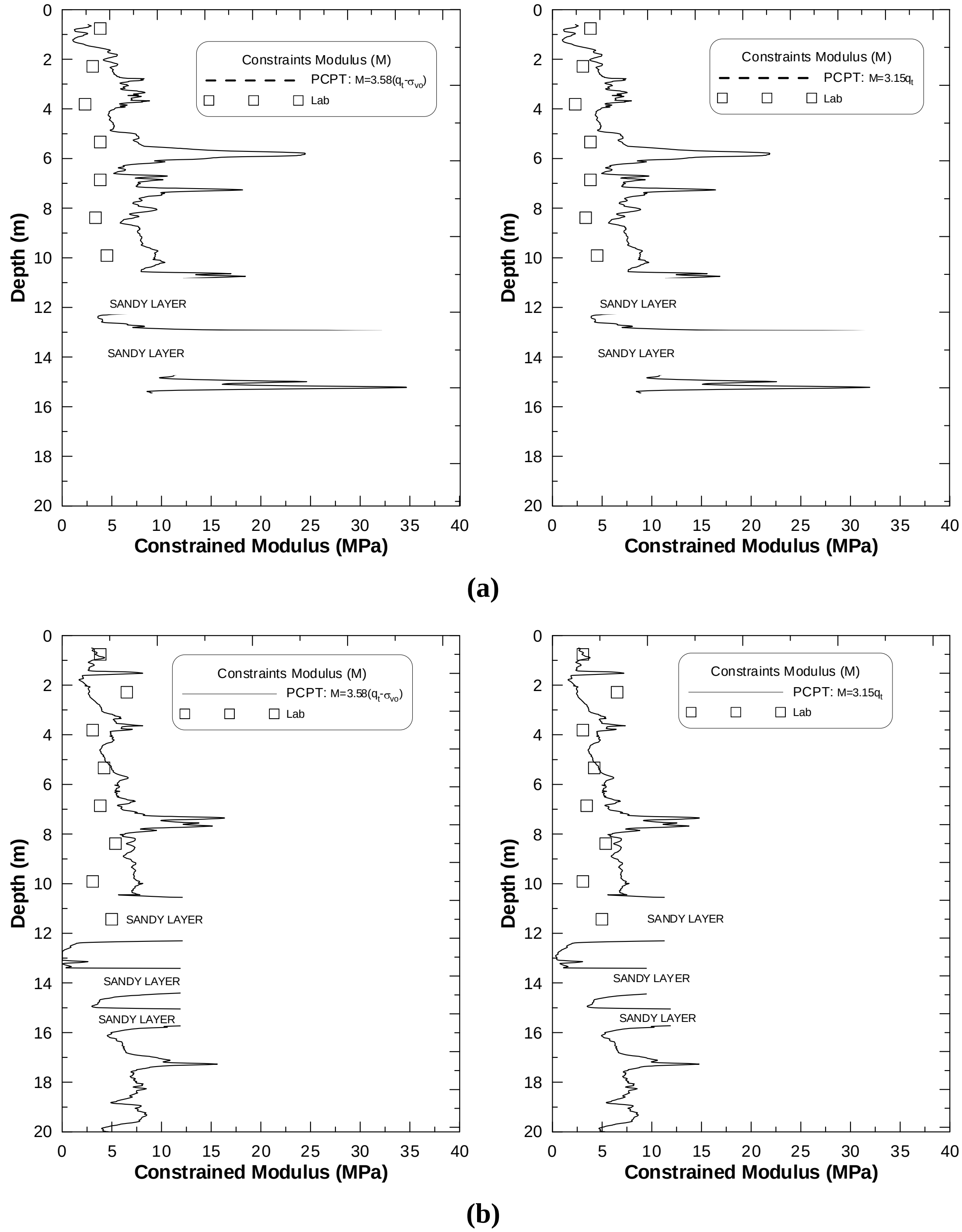


Figure 25
PCPT versus laboratory measured profiles of M (a) Juban North (b) Juban South

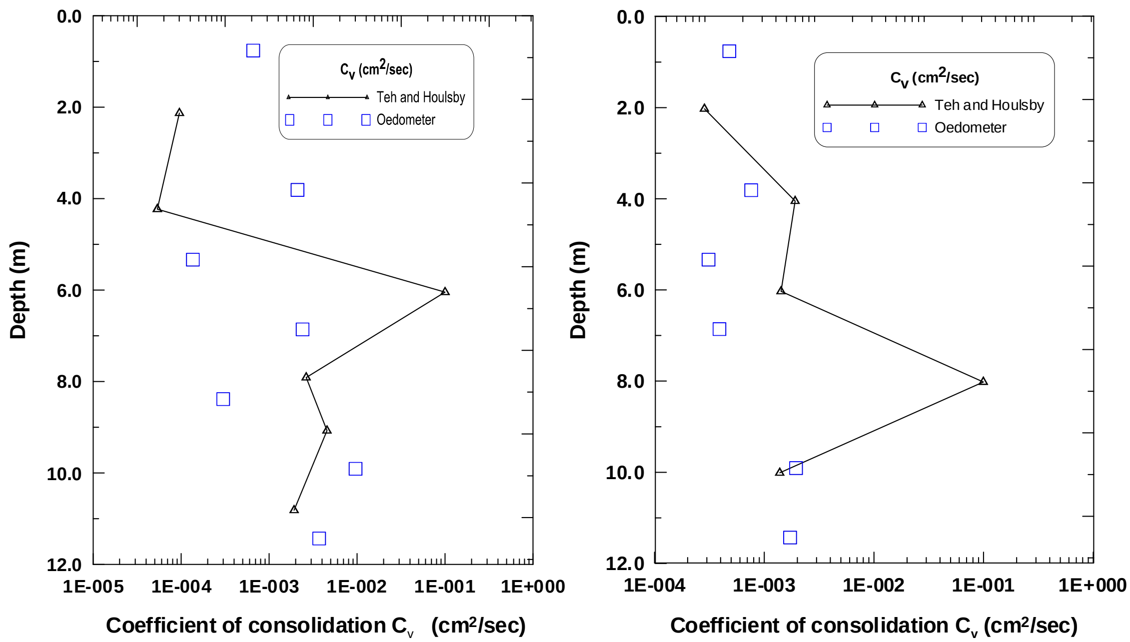


Figure 26
Measured versus predicted c_v for Juban Road Site (left: north, right: south)

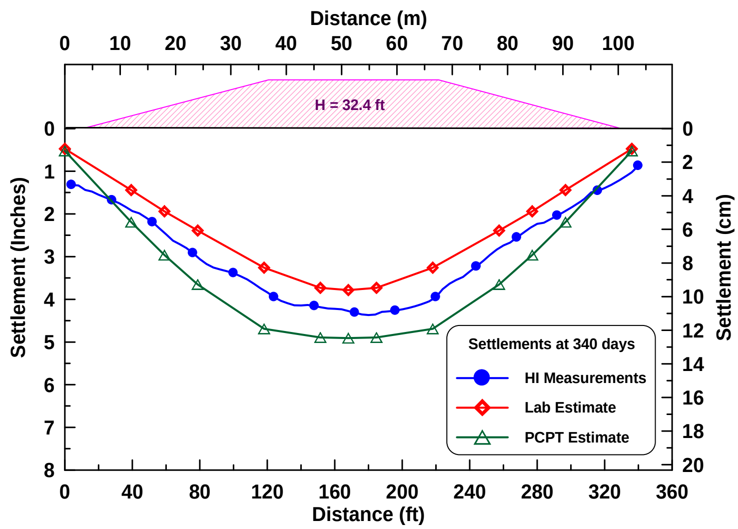
Comparison with Horizontal Inclinometer Measurements

The first horizontal inclinometer survey was conducted immediately after the compaction of a sand layer inside the trench, before placing any embankment fill, to establish the initial baseline profile. The subsequent survey measurements taken at different times were used to calculate the settlement profiles of the soil underneath the embankment. The settlement profiles were also calculated using the PCPT-interpreted parameters [equation (2)] as well as the laboratory-derived consolidation parameters. The subsurface soil properties and the results of in-situ PCPT and dissipation tests were presented earlier. The applied stress ($\Delta\sigma$) used to calculate the magnitude of settlement was due to embankment loading plus surcharge, which increased with construction time until it reached a maximum height. Due to the installation of wick drains, the excess pore water pressure can drain both vertically and radially. In this case, the average degree of consolidation (U) is given as follows:

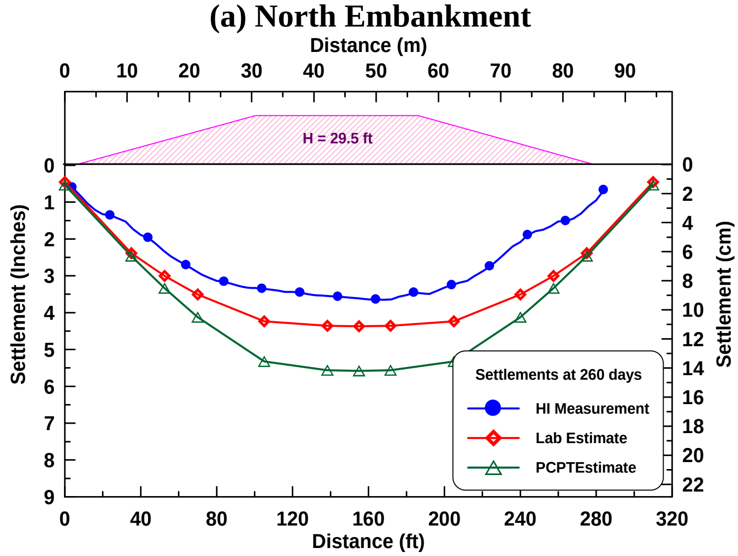
$$U = 1 - (1 - U_v)(1 - U_r) \tag{33}$$

where, U_v, U_r are the average degree of consolidation due to the vertical and radial (or horizontal) drainage, respectively.

The settlement profiles along the width of the embankment calculated using the PCPT interpretation method [equation (11)] and the laboratory-derived consolidation parameters and the settlement profiles measured using the horizontal inclinometers are presented in Figures 27a and 27b for north and south embankments, respectively. As seen in the figures, the predicted settlements from PCPT data and laboratory parameters agree fairly well with the field measurements. For the north embankment site, the PCPT predicted settlements over-estimated the measured (actual) settlements by about 12 percent, while the laboratory calculated settlements under-estimated the measured settlements by about 13 percent. For the south embankment site, both the PCPT and laboratory calculations over-estimated the field measurements (~20 percent and ~50 percent). These findings are in agreement with findings from the previous studies, which demonstrated the difficulty of predicting the actual field settlements using either PCPT or laboratory-derived parameters *[12], [18], [19], [21]*. However, being able to estimate the magnitude of settlement using PCPT data within the same range of accuracy as the laboratory calculations is an indication of the benefit of using the PCPT in predicting settlements.



(a) North Embankment

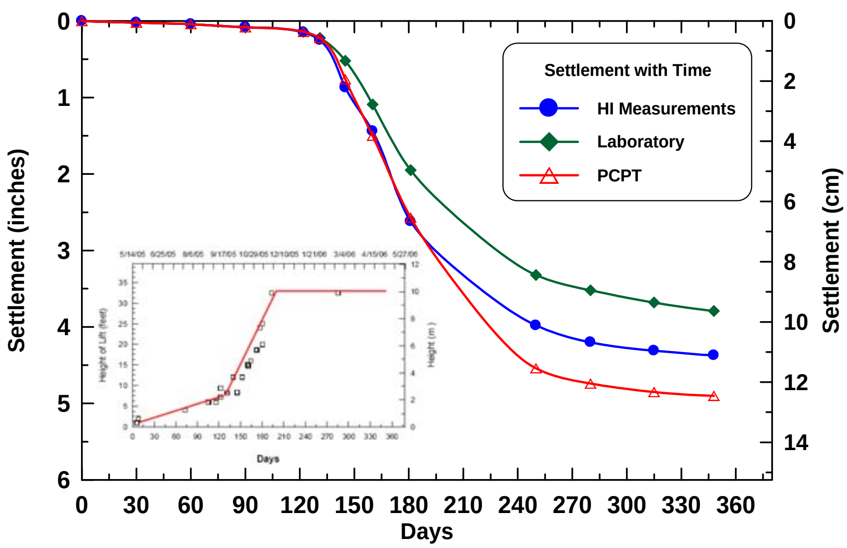


(b) South Embankment

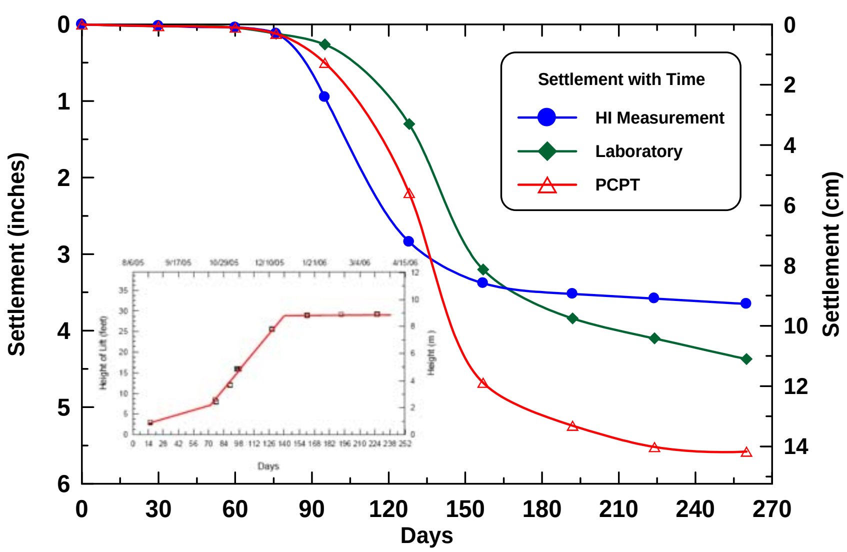
Figure 27

Comparison of predicted settlement profiles with field measurements

The rate of consolidation settlement predicted from the laboratory parameters and PCPT dissipation tests using Teh and Houlsby interpretation method are presented in Figures 28a and 28b for the north and south embankments, respectively. Although the predicted magnitudes of settlement vary from the actual field measurements, the figures clearly indicate that the PCPT estimated rate of consolidation settlement from dissipation tests matches fairly well with the field monitoring. It is worth mentioning that geotechnical engineers, in certain cases, are more interested in estimating the rate of embankment settlement than the magnitude of settlement for better planning the extent of a preloading period needed to overcome the majority of settlement.



(a) North Embankment



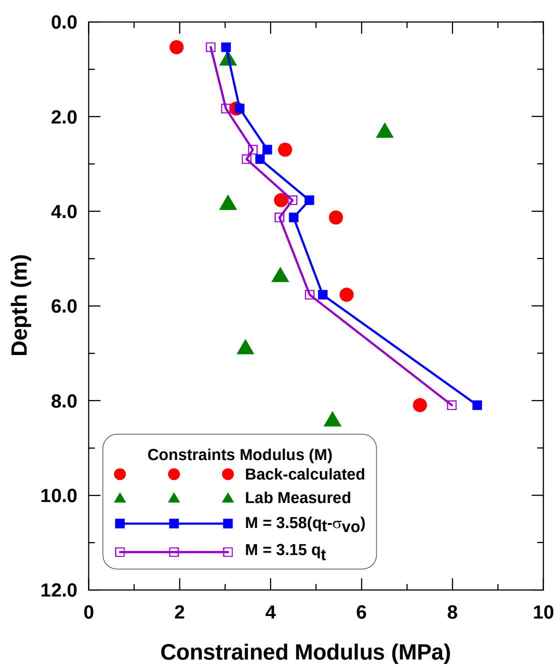
(b) South Embankment

Figure 28

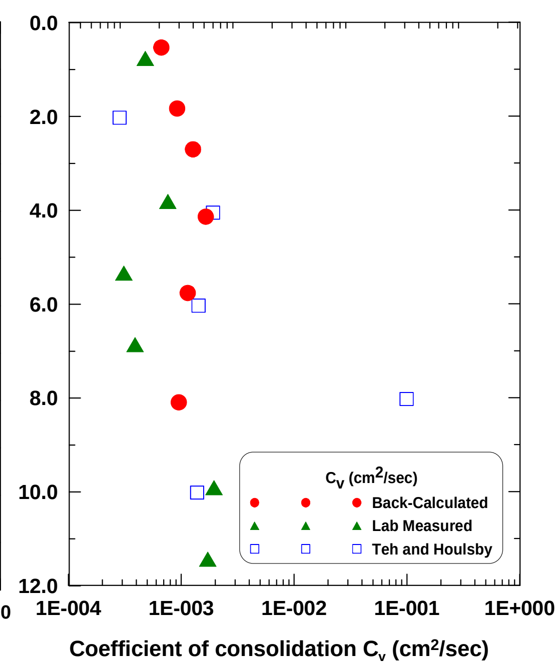
Time-rate of settlement

Back-calculation of Consolidation Parameters from Vertical Extensometer

Measurements from the vertical magnet extensometer were used to back-calculate the consolidation parameters (*M* and c_v) of the subsurface soil layers for the south embankment site. The vertical extensometer for the north site was damaged during the construction. By recording the relative movement of spider magnets, the corresponding vertical settlement of each layer was calculated for each incremental stress ($\Delta\sigma_i$). The end of primary consolidation was estimated using the rectangular hyperbola curve fitting method as proposed by Sridharan and Sreepada Rao *[33]*. The total consolidation settlement for each layer was used to back-calculate its constrained modulus (*M*) and the results are presented in Figure 29a. The figure also compares the PCPT, laboratory, and back-calculated *M* values with depth, which shows the PCPT estimated *M* values are in good agreement with the back-calculated values. Statistical analysis performed on the collected data showed that α_1 and α_2 values [M= $\alpha_{□}$ q_t, and M= α_2 ($q_t - \sigma_{vo}$)] for the south embankment site have means of 3.01 and 3.16 and standard deviations of 0.59 and 0.65. Figure 29b presents the comparison of the back-calculated c_v values, the PCPT predicted c_v values using Teh and Houlsby, and the laboratory measured c_v values. As seen in the figure, although there is some scattering, but most of the values fall within one log cycle. Most importantly, the parameters predicted using the PCPT measurements match fairly well to the range of average measured c_v values.



(a) Constrained modulus



(b) Coefficient of consolidation

Figure 29
Comparison between PCPT, laboratory and back-calculated values

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Courtableau Bridge - LA 103 Site

Comparison between PCPT and Laboratory Derived Parameters

The profiles of PCPT soundings and the results of the piezocone dissipation tests were used to calculate the consolidation parameters: constrained modulus (M) and coefficients of consolidation (c_h and c_v) of subsurface soils for the east and west embankment sides.

Constrained Modulus. Similar to the Juban road site, the predicted M values from the PCPT data are also compared with the laboratory calculated M values as shown in Figures 30a and 30b for east and west embankment sides, respectively. It is evident from the figures that the PCPT-M values for both embankment sides are greater than the laboratory estimated M values. On the average, the laboratory measured values agree with the PCPT prediction at similar depths.

Vertical Coefficient of Consolidation. The plots of piezocone estimated versus laboratory-calculated c_v values are compared in Figures 31(a) and 31(b) for the east and west embankments, respectively. The coefficient of consolidation is difficult to determine accurately in nature as shown in the Juban Road case. The PCPT estimated vertical coefficients of consolidation, in general, agree well with the laboratory measured values.

Comparison with Horizontal Inclinometer Measurements

The settlement calculation of Bayou Courtableau Bridge - LA 103 embankments was made using consolidation parameters determined based on laboratory tests on retrieved boring samples and in-situ PCPT penetration and dissipation tests. The embankment load was obtained as the height difference between the new embankment and the existing old embankment as indicated in Figure 15. The stress induced by the applied embankment load ($\Delta\sigma$) was calculated using a MatLab code based on the concept of vertical stress distribution due to embankment loading *[30]*. The results of laboratory consolidation tests were used to calculate the embankments' settlements as indicated by "-Lab" as shown in Figures 32 and 33. Settlements based on laboratory tests were calculated as the primary consolidation settlements. The properties of subsurface soils and the results of PCPT penetration and dissipation tests were presented earlier. The constrained modulus (M) for each layer was predicted using the Sanglerat method and the correlations proposed by Abu-Farsakh [$M = 3.58(q_t - \sigma_{vo})$, $M = 3.15q_t$], with q_t representing the average q_t values of the soil layer. The vertical coefficient of consolidation (c_v) predicted from piezocone dissipation tests using the Teh and Houlsby method was used to determine the time-rate of consolidation *[12]*.

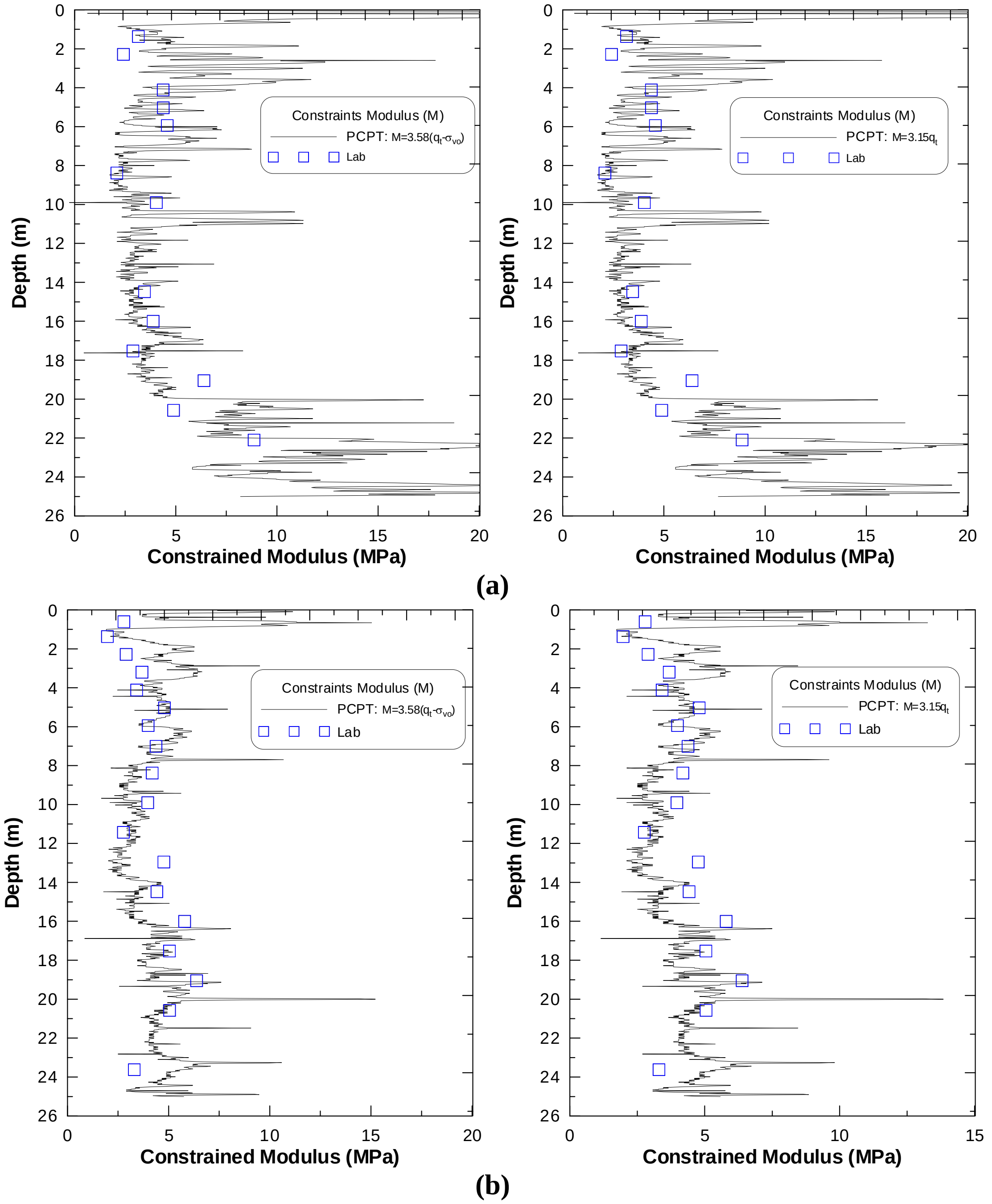


Figure 30
PCPT versus laboratory measured profiles of M (a) Courtableau east embankment, (b) Courtableau west embankment

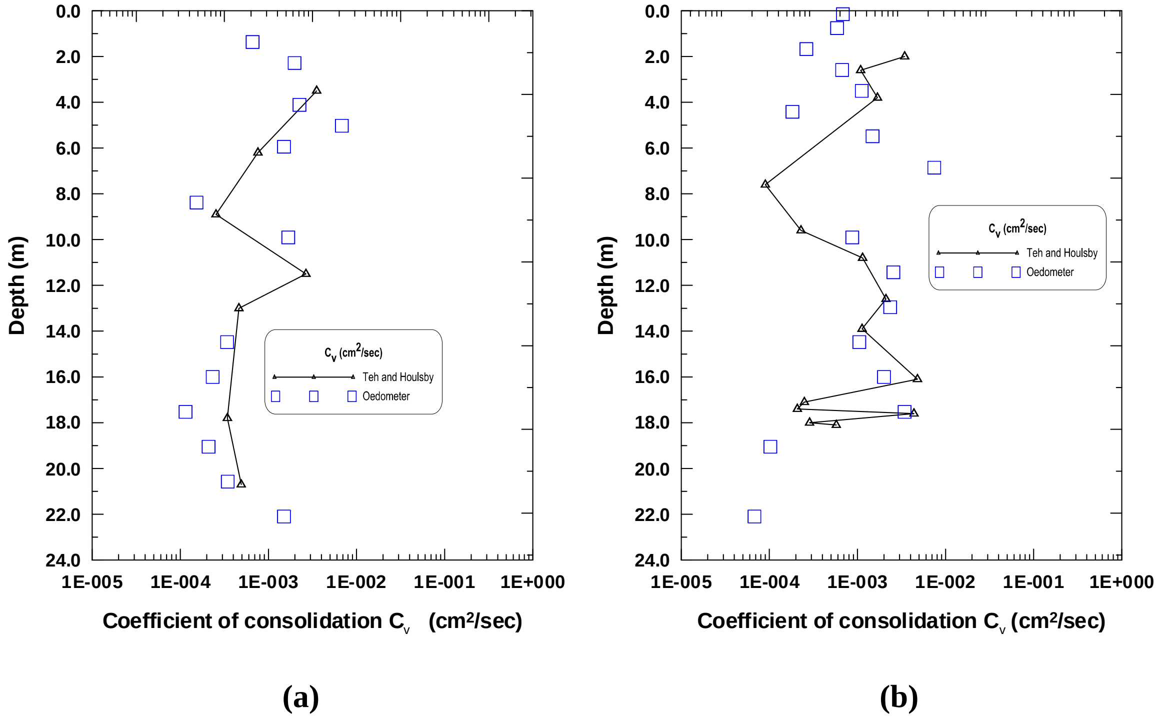


Figure 31
Measured versus predicted c_v for Courtableau Bridge - LA 103 Site (a) east embankment (b) west embankment

The PCPT-predicted settlements (using Sanglerat and Abu-Farsakh's correlations of *M*) were compared with the laboratory-calculated settlements and field-measured settlements using the horizontal inclinometer as shown in Figure 32 and 33 for the east and west embankments, respectively. Upon general observation, the settlement calculation based on laboratory tests tends to over predict the measured settlement. The proposed PCPT interpretation equations gave a better prediction of the total consolidation settlements than the Sanglerat PCPT method and the laboratory estimations, with the settlements calculated from laboratory tests having the largest settlements. The predicted settlements using the Sanglerat method are slightly larger than the settlement predicted by the proposed equations. The settlement rates with time is shown in Figures 34(a) and 34(b) for the east and west embankments, respectively. In the first month, the laboratory calculation has the largest settlement rate; while in the following months, all three methods have almost similar settlement rates.

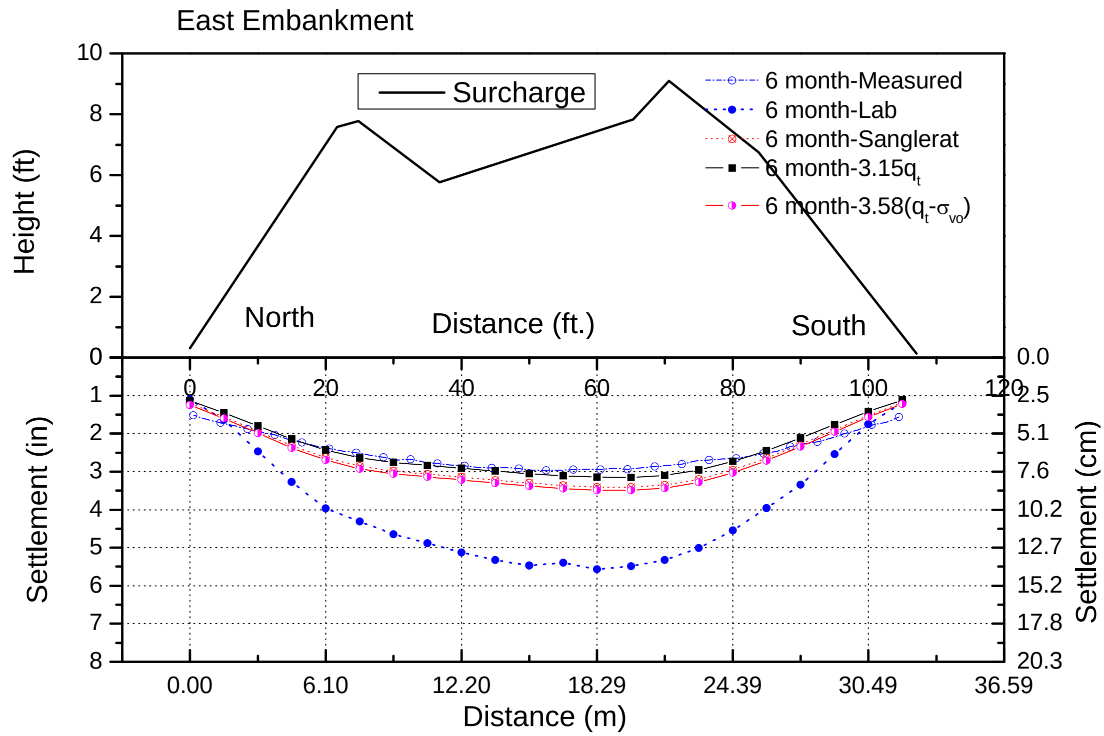


Figure 32
Comparison of predicted settlement profiles with field measurements for Bayou Courtableau Bridge - LA 103, east embankment

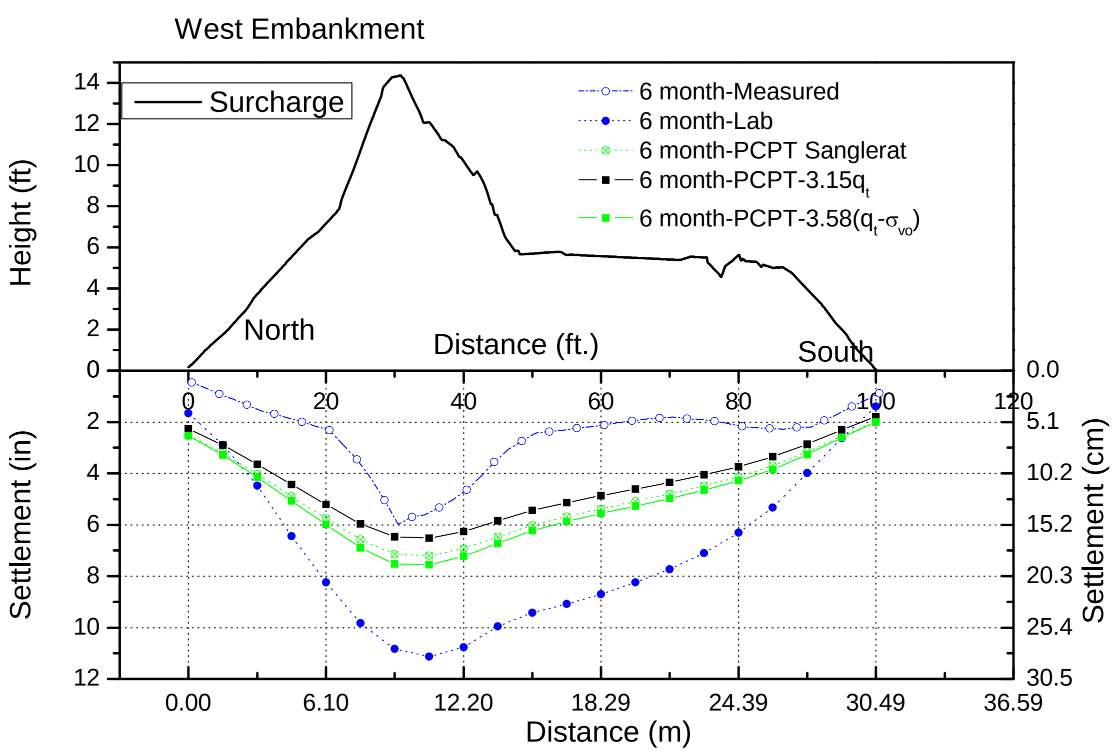
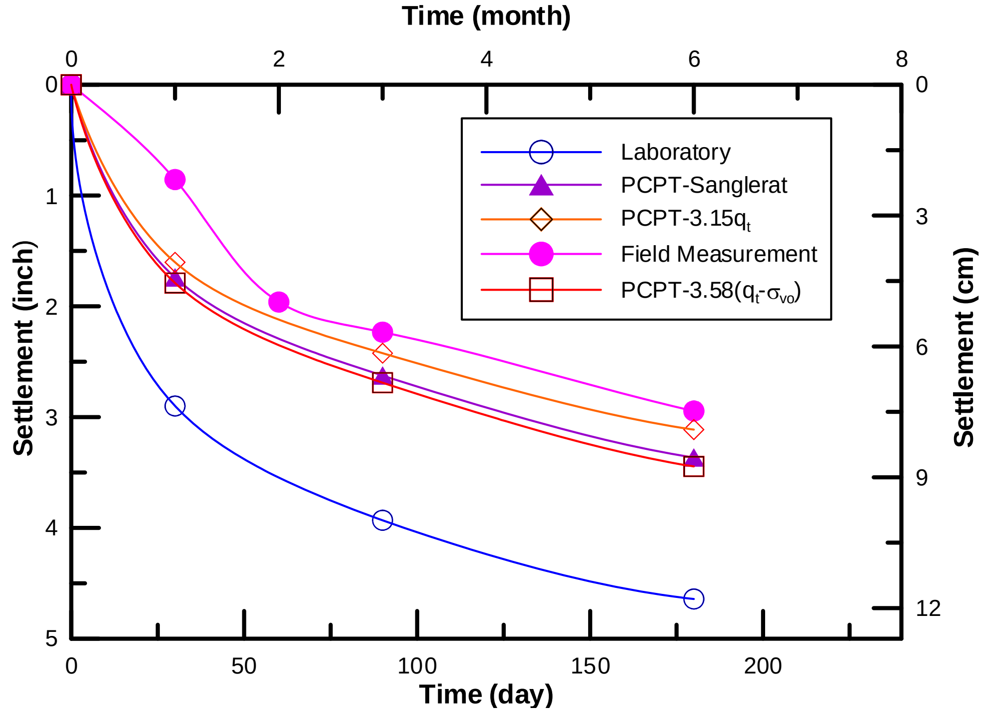
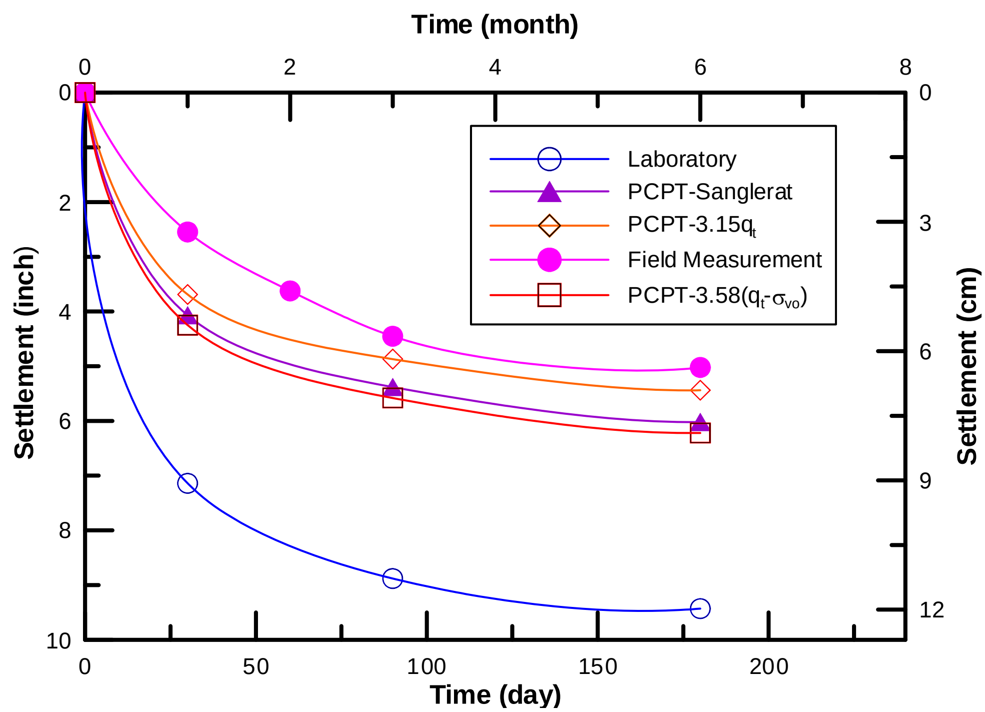


Figure 33
Comparison of predicted settlement profiles with field measurements for Bayou Courtableau Bridge - LA 103, west embankment



(a) east embankment



(b) west embankment

Figure 34
Time-rate of settlement

Development of Computer Software
Louisiana Embankment Settlement Prediction Program from PCPT (LESPP-PCPT)

This software application was developed to classify and calculate settlement under the embankment loading. This section gives an overview of the software package developed and describes the main features available.

Introduction
This windows application package (Figure 35) was developed using Visual Basic 6.0 and ChartFX software version 5.1 to facilitate the estimate of magnitude and time-rate of embankment settlement in the field using PCPT parameters. This application can be installed on any personal computer or laptop with the Windows operating system. In the following sections, the available features in this application are described in detail.

Startup Windows and Input Files
This software provides an interactive Graphical User Interface (GUI). The user can enter inputs, browse files, and navigate using buttons and other user friendly GUI controls. The main screen requests the user to provide the location of the PCPT files and the dissipation files, the unit system of the data stored in these files, and the ground water table depth. These files should be in a text format. Once the files are correctly read, the data read from the files are analyzed by the program to calculate other related parameters and displayed in the form of graphs. If type 2 cone tests (with u_2 measurement) are available and uploaded, this program also corrects cone tip resistance for pore water pressure. There is a text area to preview the contents of the file before uploading. If there is an error in reading the file, appropriate error messages are provided to guide the user to solve the issue as easily as possible. This feature helps verify the type of cone used, check units, as well as other remarks (Figure 36).

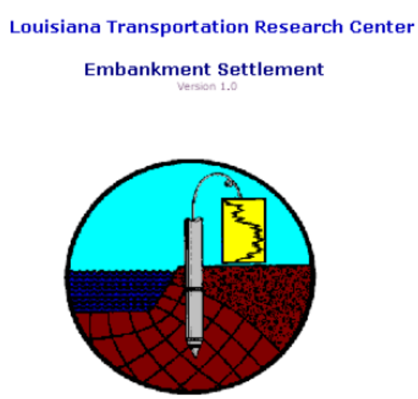


Figure 35
Embankment settlement program logo

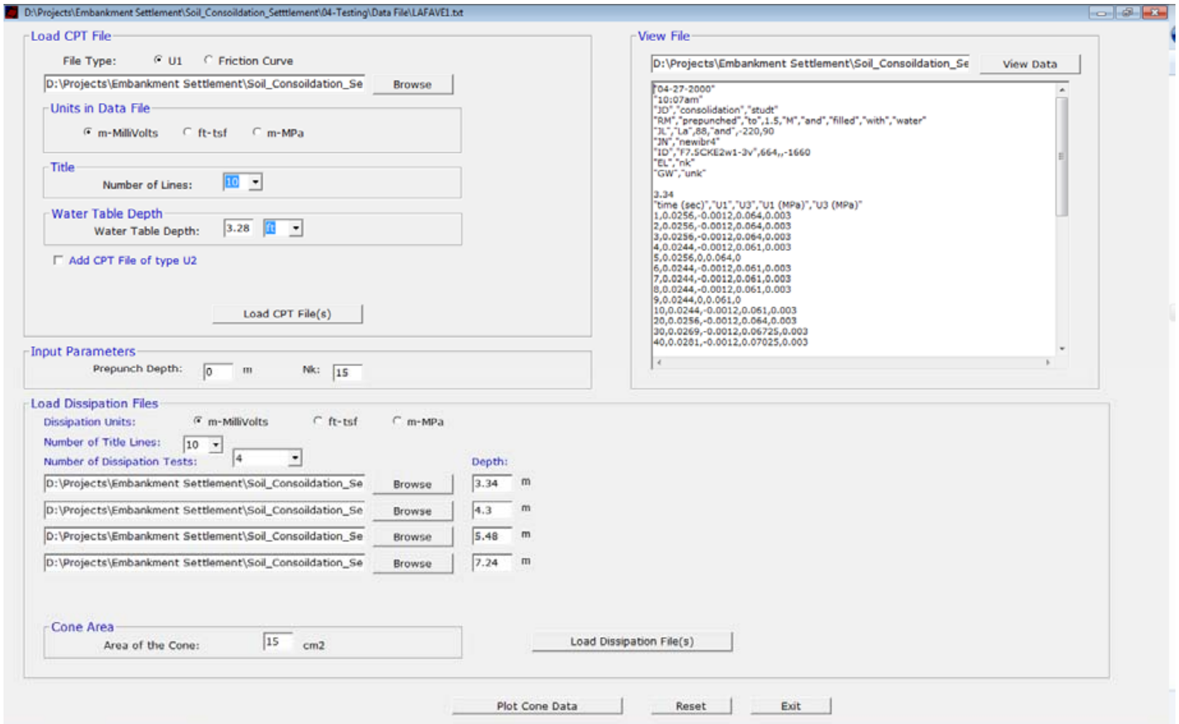


Figure 36

Opening window with navigation links and input parameters

Project Information

Once data are loaded into the program, there is another input screen displayed for the user to enter project related information. Project information and other remarks are displayed in a pop-up window as shown in Figure 37. This information is meant for project identification and data display purposes only.

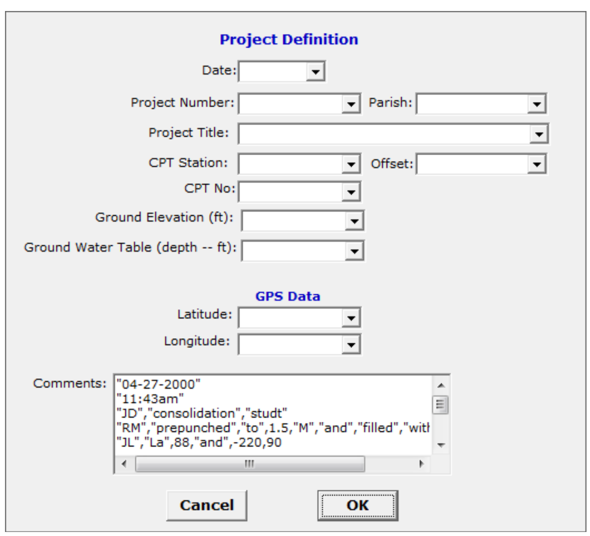


Figure 37

Project information window

Plot of PCPT Profile and Soil Classification

The first two graphs in this window display q_t and f_s profiles with depth. Other two graphs display pore pressure measurements or soil classification depending upon the user's selection. The main functionalities available in this window are described in the following sections.

Classify Soil. The soil classification profile for the test is plotted using the probabilistic region estimating soil classification method (Zhang and Tumay) as shown in Figure 38 [30]. The classification profile is displayed along with tip resistance, sleeve resistance, and sleeve tip ratio to aid analysis. The charts are aligned together for readability purposes. As it can be gleaned from the figure below, the project description entered in Figure 37 is visible at the bottom of the form displayed in the figure below.

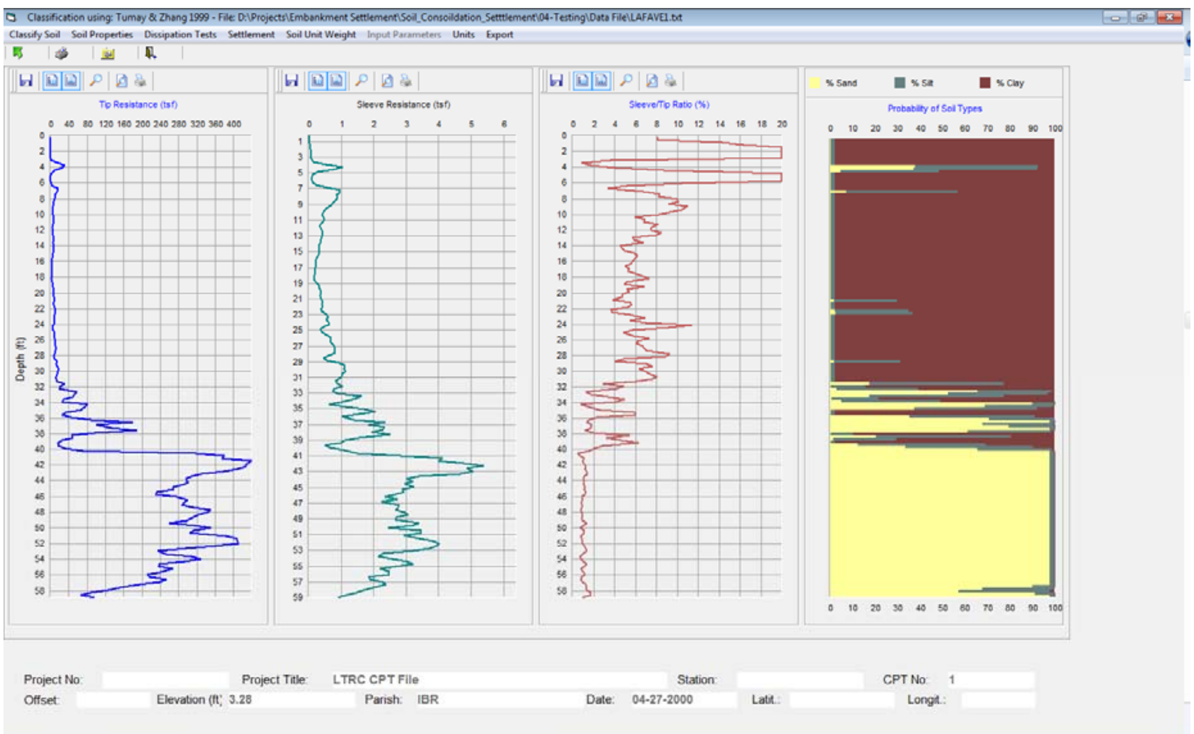


Figure 38
Plot of PCPT profile and soil classification at test site

Soil Unit Weight. In order to calculate the soil unit weight, the program provides the user with options to either select one average value for soil unit weight or enter soil unit weights for different layers from the borehole information as shown in Figure 39. In addition, the unit weight for each soil layer can be estimated using CPT measurements *[34]*. As overburden stress is used in several PCPT correlations, several features in the other windows are disabled until a selection is made in this menu.

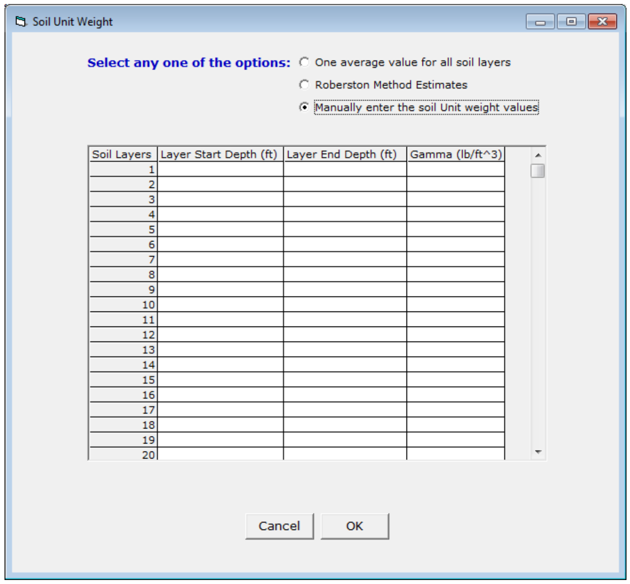


Figure 39
Soil unit weight input weight input window

Soil Properties. This menu allows the selection of the display of the profile of undrained shear strength (S_u), constrained modulus (M), and OCR with depth, estimated using PCPT correlations.

Dissipation. An option is available to display dissipation curves (opens in separate window) or the profile of the c_v or c_h value estimated using Teh and Houlsby method (Figure 40) *[12]*. Also the calculated values of c_v and c_h can be exported in text formats.

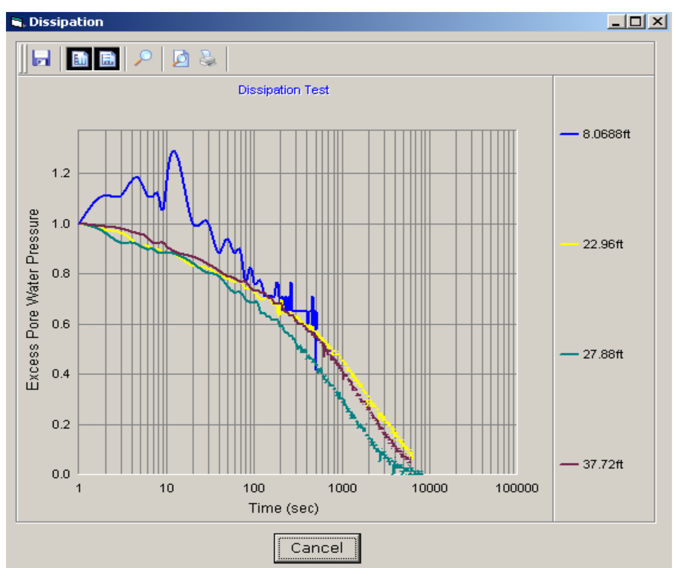


Figure 40
Normalized dissipation curves for different depths

Units. The program provides the user an option to choose English (ft-TSF) or metric (m-MPa) units as per their convenience.

Settlement. Analysis of the settlement under embankment loading can be done for both symmetric and asymmetric embankments. Once the basic design parameters are entered, the program estimates the magnitude of settlement under embankment loading based on PCPT estimated consolidation parameters. An input window for this feature is shown in Figure 41.

Settlement at any point under the embankment can be displayed by choosing the coordinate (x) of point from the origin (from the left hand side of embankment). Also display options for the settlement profile along the embankment width with respect to time (Figure 42 and 43) and time-rate of settlement at the ID point (maximum settlement) as shown in Figure 44.

Summary of Input Parameters. This summary of input parameters screen gives the summary of estimated consolidation parameters, soil classification, and location of drainage layers as used for settlement calculation (Figure 45). Also, at this point, users can manually change or add the information based on experience, engineering judgment, or other additional information such as results from close borehole drill. These edited parameters are automatically updated by the program for its calculation and used to furnish a new settlement profile. The application can also be used to predict the settlement profile for laboratory estimated parameters by replacing CPT parameters in the table by laboratory estimates.

Provision for Design of Surcharge Height and PVD Installation. In order to expedite the time-rate settlement in the field, sometimes additional temporary fill, known as surcharge, is used. In some cases, surcharge alone may not be sufficient and, in that case, vertical drains such as sand drains or PVD are used to accelerate the dissipation of excess pore water pressure and, hence, time of settlement. This application can also be used to estimate the height of surcharge and to design PVD parameters during the early design stage. Users can manipulate different values of surcharge height and/or add a PVD option to determine the optimum condition to get the desired value of embankment settlement within the given time frame. The settlement profiles displayed in Figure 42 and Figure 43 are for total settlements and settlement with expansion, respectively.

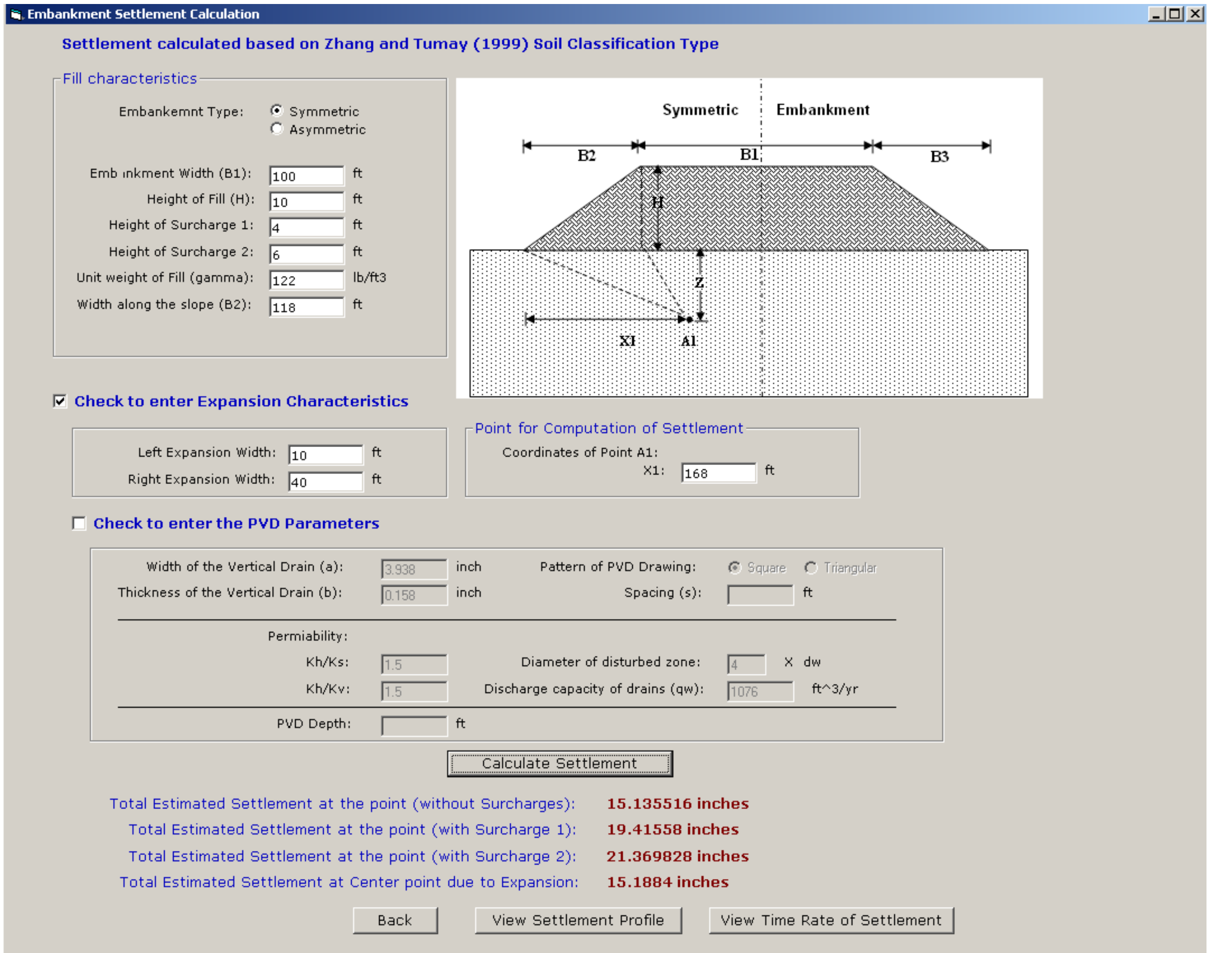


Figure 41

Input window for embankment dimension, fill characteristics, and PVD design

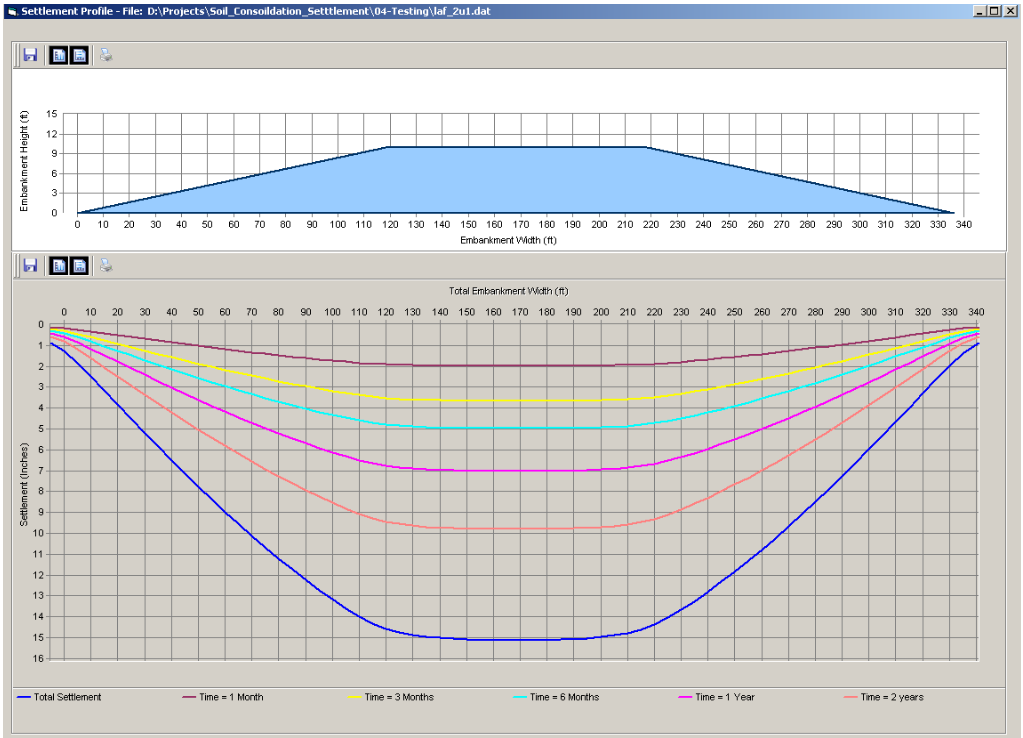


Figure 42

Progress of settlement profile along the width of embankment with time

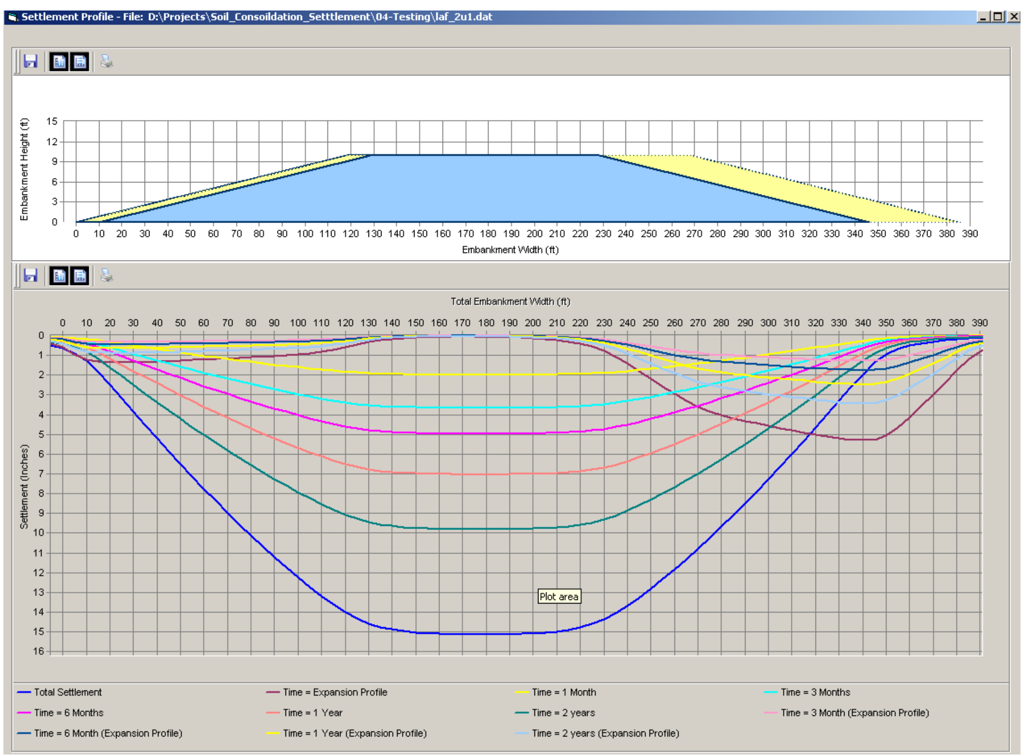


Figure 43

Progress of settlement profile with expansion along the width of embankment with time

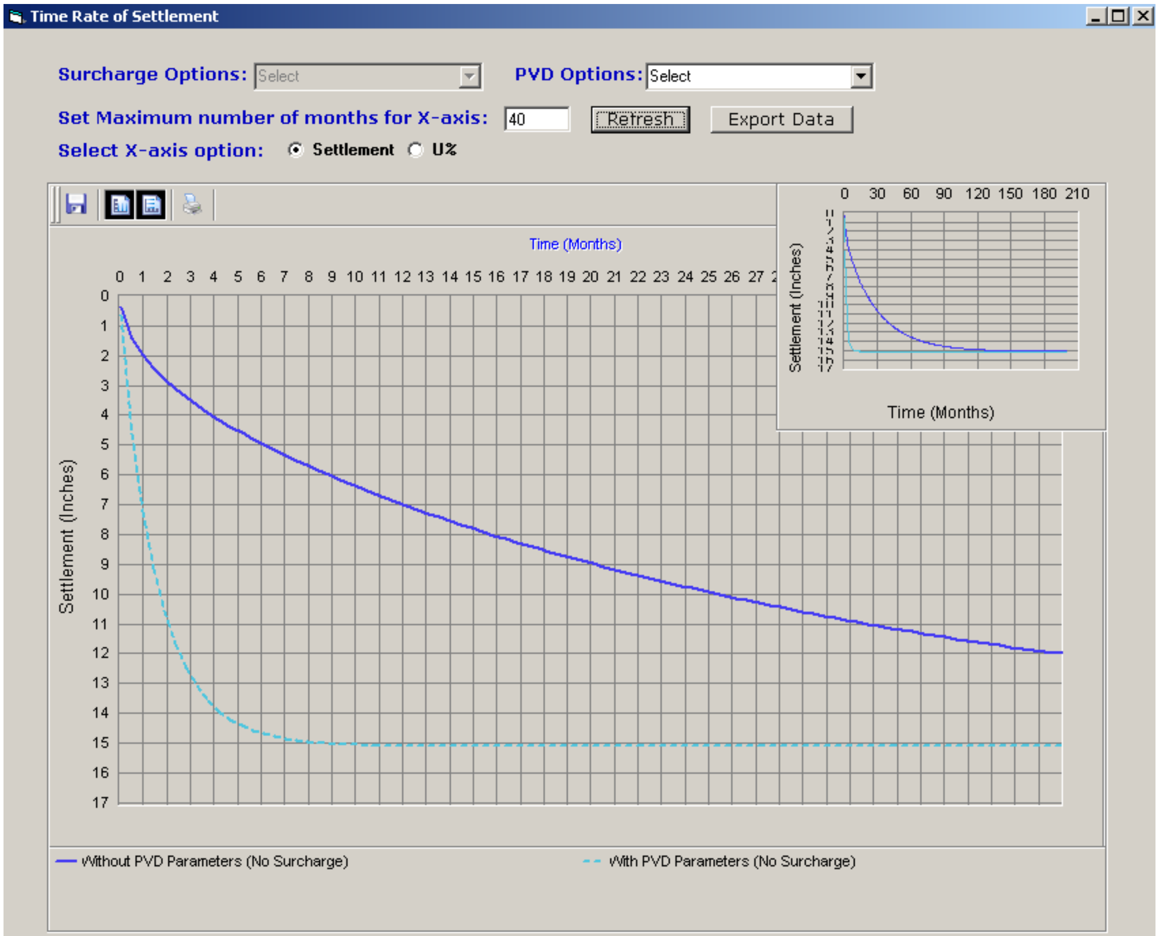


Figure 44
Comparison of time-rate of settlement curve at the center for with and without surcharge condition

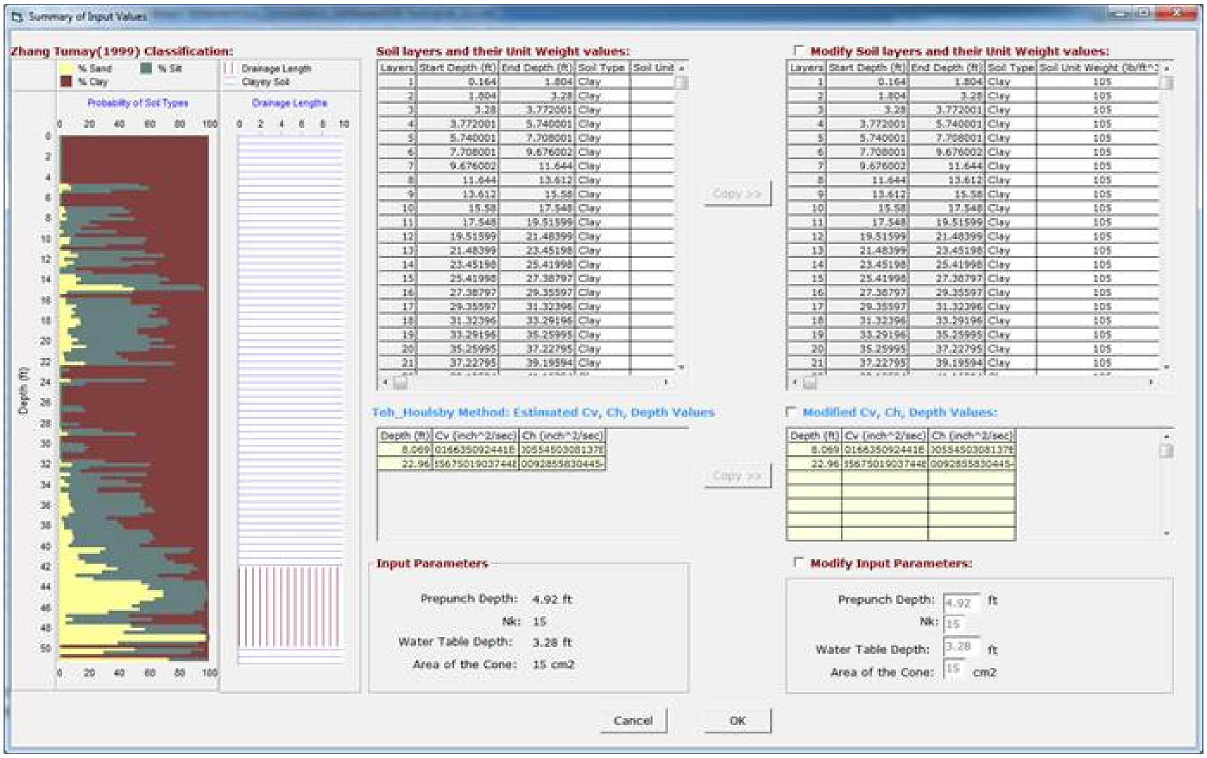


Figure 45
Summary tables for design parameters used in calculation

CONCLUSIONS

This report presented verification of previously proposed PCPT methods for estimating the magnitude and time-rate of consolidation settlement of embankments over fine-grained soils *[8]*. For this purpose, two bridge construction sites, with two embankments each, were selected: the Juban Road - I-12 Interchange Bridge and the Bayou Courtableau Bridge - LA 103 to verify the PCPT methods. On each embankment site, embankment settlements were monitored using horizontal inclinometers and/or vertical magnet extensometers. Laboratory tests, PCPT, and piezocone dissipation tests were conducted on each site to determine the consolidation parameters needed for settlement calculation. For each embankment site, the magnitude and time-rate of settlement predicted from PCPT and laboratory derived consolidation parameters were compared with the field measured settlements. Based on results from these two bridge sites, the following conclusions can be made:

- The results showed that the PCPT methods utilizing piezocone penetration and dissipation data can be used to estimate the magnitude and time-rate of consolidation settlement of embankments within the same range of accuracy as of the traditional laboratory settlement calculation method.

- The PCPT interpreted consolidation parameters were also compared with laboratory measured values as part of the verification program. The PCPT derived constrained modulus (M) using the Abu-Farsakh equations of $M = 3.15\ q_t$ and $M = 3.58(q_t - \sigma_{vo})$ had fairly good estimation of the constrained modulus *[8]*. These two correlation equations had a closer and slightly better performance than the Sanglerat method *[2]*.

- The PCPT estimation of the coefficients of consolidation (c_v, c_h) from dissipation test data using Teh and Houlsby interpretation method gave better estimation of the rate of consolidation settlement with time than the laboratory calculations *[12]*.

- The back-calculated constrained modulus, M, from vertical magnet extensometers were compared with the PCPT and laboratory derived M values. The results showed that the PCPT estimated M values are in good agreement with back-calculated values, while the laboratory estimated M values are lower.

- The comparison of the c_v values estimated from the dissipation tests using the Teh and Houlsby method with the back-calculated and laboratory measured c_v values showed reasonable agreement among all *[12]*.

- The PCPT methods have the advantage of providing continuous profiles of the constrained modulus and coefficient of consolidation with depth. In addition, performing the PCPT tests is much faster compared to the sampling and subsequent laboratory testing of soil samples, thus the use of PCPT will help in speeding up the field construction. Hence, the PCPT based settlement calculation can substitute the traditional settlement calculation based on laboratory tests that require time-consuming laboratory testing.

RECOMMENDATIONS

The increasing use of the cone penetration soundings at LADOTD will eventually make the estimation of the magnitude and time-rate of settlement easier, faster, cheaper, and more reliable compared to the expensive and time-consuming sampling and the subsequent laboratory testing of soil samples. In addition, in-situ PCPT tests can provide the data needed to estimate the parameters of soils that are difficult or near impossible to obtain using normal means. Based on the results of this study, it is recommended that LADOTD implement the PCPT technology to estimate the consolidation settlement of fine-grained soils, in conjunction with the traditional laboratory calculation of settlements.

Researchers recommend that LADOTD engineers continue comparing the consolidation settlements predicted from the PCPT data, the calculated settlements from the laboratory consolidation parameters, and the field measured settlements to gain experience and confidence when using PCPT for settlement estimation purposes. With increasing confidence and experience, LADOTD engineers can gradually move toward replacing the conventional subsurface exploration with piezocone penetration and dissipation tests for the estimation of consolidation settlement.

It is recommended to study the possibility and limitations of estimating the rate of consolidation settlement for unsaturated soils from PCPT data.

It is also recommended to extend the study to develop correlations between the coefficient of secondary compression (c_{α}) and the PCPT data/classification charts to evaluate the secondary consolidation of the soils.

Researchers recommend extending the Louisiana Embankment Settlement Prediction Program from PCPT to include settlement calculations of other structural foundations by using either PCPT or laboratory input soil consolidation properties.

Finally, it is recommended to develop a training manual and workshop in order to train LADOTD engineers on how to use PCPT methods for evaluating the consolidation parameters needed for estimating the total and time-rate of settlements of fine-grained soils.

ACRONYMS, ABBREVIATIONS, AND SYMBOLS

CPT	Cone Penetration Test
CU	Consolidated Undrained
GUI	Graphical User Interface
LADOTD	Louisiana Department of Transportation and Development
LESPP	Louisiana Embankment Settlement Prediction Program
LTRC	Louisiana Transportation Research Center
OCR	Overconsolidation Ratio
PCPT	Piezocone Penetration Test
REVEGITS	Research Vehicle for Geotechnical In situ Testing and Support
UU	Undrained Unconsolidated

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