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16. Abstract

The piezocone penetration test (PCPT) has gained wide popularity and acknowledgement as a preferred in-situ device for subsurface investigation and soil characterization. The PCPT measurements can be utilized for soil identification and the evaluation of different soil parameters. Different interpretation methods have been proposed to evaluate the strength and consolidation parameters of cohesive soils utilizing the piezocone penetration and dissipation test data. This report presents the evaluation of the capability of the current PCPT interpretation methods to reasonably predict the consolidation parameters needed to predict the total and time rate of settlement of cohesive soils. Seven sites in Louisiana were selected for this study. In each site, in-situ PCPT tests were performed and soundings of cone tip resistance (q_c), sleeve friction (f_s) and pore pressures (u_1 and u_2) were recorded. Dissipation tests were also conducted at different penetration depths. High quality shelby tube samples were collected close to the PCPT tests and used to carry out a comprehensive laboratory testing program including unconfined compression test, triaxial test and one-dimensional oedometer consolidation test. The tangent constrained modulus (M), overconsolidation ratio (OCR) and the vertical coefficient of consolidation (c_v), predicted using the different interpretation methods, were compared with the reference values determined from the laboratory consolidation tests. Results of this study showed that the consolidation parameters of soils can be reasonably predicted from the piezocone penetration and dissipation test data, and hence provide a continuous profile of these parameters with depth. The results of this study were verified by comparing the predicted settlements from PCPT methods with the laboratory calculated and field measured settlements in three selected sites.

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EVALUATION OF CONSOLIDATION CHARACTERISTICS OF COHESIVE SOILS FROM PIEZOCONE PENETRATION TESTS

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ABSTRACT

The piezocone penetration test (PCPT) is widely acknowledged as a preferred in-situ device for subsurface investigation and soil characterization. The PCPT measurements can be used for soil identification and the evaluation of different soil parameters. Different interpretation methods have been proposed to evaluate the strength and consolidation parameters of cohesive soils using the piezocone penetration and dissipation test data. This report presents the evaluation of the current PCPT interpretation methods' capability to reasonably predict the consolidation parameters necessary to calculate the total cohesive soil settlement and time rate. Seven sites in Louisiana were selected for this study. At each site, in-situ PCPT tests were performed, and soundings of cone tip resistance (q_c) sleeve friction (f_s), and pore pressures (u_1 and u_2) were recorded. Dissipation tests were also conducted at different penetration depths. High quality shelby tube samples were collected close to the PCPT tests and used to carry out a comprehensive laboratory testing program including the unconfined compression test, triaxial test, and one-dimensional oedometer consolidation test. The tangent constrained modulus (M), overconsolidation ratio (OCR), and the vertical coefficient of consolidation (c_v), predicted using the different interpretation methods, were compared with the reference values determined from the laboratory consolidation tests. Results of this study showed that the consolidation parameters of soils can be reasonably predicted from the piezocone penetration and dissipation test data, and thus provide a continuous profile of these parameters with depth. The results of this study were verified by comparing the predicted settlements from PCPT methods with the laboratory-calculated and field-measured settlements from three selected sites.

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IMPLEMENTATION STATEMENT

The results of this study demonstrated that the consolidation parameters—constrained modulus (*M*), overconsolidation ratio (*OCR*), and vertical coefficient of consolidation (c_v)—can be reasonably predicted from the results of piezocone penetration and dissipation tests. These parameters can be used to predict the magnitude and time rate of consolidation settlement for normally and lightly overconsolidated cohesive soils. However, the predicted total settlements from PCPT data were more reliable and accurate than the predicted rate of settlements. The proposed linear correlations exhibited better performance than the other interpretation methods, and therefore are the recommended means to estimate the constrained modulus (*M*) and the overconsolidation ratio (*OCR*). The Teh and Houlsby method provided good prediction and is recommended for the evaluation of the vertical coefficient of consolidation (c_v) [1].

The availability of the cone penetration test systems at LA DOTD 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 subsequent laboratory testing of soil samples. In addition, in-situ PCPT tests can provide the data needed to estimate the soil parameters in soils where it is impossible to obtain adequate sampling. Therefore, based on the results of this study, it is recommended that LA DOTD engineers gradually start implementing the PCPT technology, particularly to estimate the consolidation settlement of fine-grained soils, in conjunction with the traditional laboratory calculation of settlements. LA DOTD engineers should continue to compare the consolidation settlements predicted from the PCPT data, the calculated settlements from laboratory consolidation parameters, and the field measured settlements until they build enough confidence in the PCPT interpretation methods. With increasing confidence and experience, LA DOTD 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, in the long run, will result in a cost benefit and an improvement in settlement prediction.

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INTRODUCTION

Saturated fine-grained soils, when loaded, can undergo large consolidation settlements over a long period of time. The presence of this type of soil deposit is very common in southern Louisiana. Therefore, the construction of embankments, bridges, and other structures on soft Louisiana soils requires a reasonable estimate of the magnitude and time rate of consolidation settlement of the soil in order to conduct a rational and safe foundation analysis and design. A reliable estimate of the settlement of structures on soft soil deposits requires correct evaluation of the consolidation parameters of foundation soils.

The strength and consolidation characteristics of cohesive soils can be estimated either from laboratory or from in-situ tests. The laboratory tests such as the oedometer consolidation test are usually conducted on small, presumably undisturbed, intact samples. However, almost all recovered samples have a certain degree of disturbance. Because of a small sample size and the unknown degree of disturbance, the laboratory-derived strength and consolidation parameters may not be entirely representative of the in-situ soil conditions. For example, estimating the vertical coefficient of consolidation (c_v) from laboratory measurements can under-predict the in-situ values by several orders of magnitudes [2, 3, and 4], leading to an error by a factor of two to ten [5]. For soils that are interbedded or have some fabric, such as fissures or layering, the laboratory testing on small intact samples can be misleading. In addition, profiling the consolidation characteristics from laboratory tests conducted on samples taken from different depths can easily miss significant thin drainage layers [6].

In-situ tests can provide more accurate and reliable results than laboratory tests in assessing the actual in-situ strength and consolidation performance of soils. The use of conventional field tests (such as borehole permeameter, self-boring permeameter, pre-inserted porous probes) to measure total and time rate of consolidation, are expensive, time-consuming, and require high skill and experience; hence, it is not always possible to perform enough tests to achieve satisfactory results. The piezocone penetration test (PCPT) is gaining acknowledgement as a preferred device for subsurface investigation, soil characterization, and evaluation of geomedia. The PCPT is a robust, simple, fast, and economical test that can provide continuous soundings of subsurface soil. Capable of distinguishing between different drainage conditions during penetration, the PCPT test is basically conducted by advancing a cylindrical rod with cone tip down into the soil. The piezocone penetrometer can measure the cone tip resistance (q_c), sleeve friction (f_s), and pore pressures at different locations, depending on the location of the pressure transducer (at the cone face (u_1), behind the base (u_2), or behind the sleeve (u_3)). These measurements can be effectively used for soil stratification and identification, and to evaluate different soil properties such as the strength and consolidation characteristics of the soil. This makes the PCPT valuable for a wide range of geotechnical engineering applications.

The total consolidation settlement of fine-grained soils can be estimated from deformation moduli such as the constrained or tangent modulus (M), while the time rate of settlement is estimated using the vertical coefficient of consolidation (c_v). Different interpretation methods have been proposed to estimate the constrained modulus (M) from the piezocone penetration tests (PCPT) [7, 8, 9, and 10]. The available proposed correlations were determined by relating the PCPT test data (mainly q_c) to the laboratory measured constrained modulus obtained from one-dimensional oedometer tests. Several empirical, semi-empirical, analytical, and finite element interpretation methods have been developed by researchers to estimate the horizontal coefficient of consolidation (c_h) of cohesive soils from the piezocone dissipation tests [e.g., 1, 10, 11, 12, 13, 14, and 15]. Some of these methods are based on estimating the time for 50 percent dissipation (t_{50}) [e.g., 11, 13], some based on evaluating the gradient of initial linear dissipation [e.g., 14], and others based on the rate of dissipation at a given dissipation level [e.g., 12]. The rigidity of the soil (I_r) was included in some methods. The vertical coefficient of consolidation (c_v) can then be calculated using the relation suggested by Levadoux and Baligh, which is based on the ratio of the vertical to horizontal coefficients of hydraulic conductivity of the soils [13].

Deformation and compressibility characteristics of soils are highly dependent on the stress history represented by the overconsolidation ratio (*OCR*). Therefore, for a proper selection of the relevant soil parameters to estimate the total settlement, it is necessary to profile the *OCR* with depth. The correct evaluation of the *OCR* is very critical in estimating the total consolidation settlement of overconsolidated cohesive soils, since the deformation characteristics of the soil changes as the applied load exceeds the pre-consolidation pressure (P_c). Several investigators used the PCPT test that can provide continuous measurements with depth to estimate the *OCR*. Several correlation methods, mostly empirical, were proposed to evaluate the *OCR* from the PCPT data. These methods are based either on undrained shear strength (s_u) [e.g., 16], or directly from the PCPT profile using either tip resistance (q_c) or pore pressure (u) [e.g., 9, 17, 18, and 19]. Therefore, part of this study evaluated the reliability of the existing interpretation methods for estimating the *OCR* from the PCPT data and/or developing a new method based on the collected PCPT data.

The main objective of this study was to evaluate the current interpretation methods for their capability to reasonably predict the consolidation parameters needed to calculate the magnitude and time rate of consolidation settlement of cohesive soils as well as the *OCR*. Seven sites in southern Louisiana were included in this study. In each site, in-situ PCPT tests were performed and dissipation tests were conducted at different penetration depths. A comprehensive laboratory-testing program was conducted to calculate the reference soil parameters. 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 results of this study were verified by comparing the predicted settlements from the proposed PCPT method with the field measured settlements at three selected sites.

OBJECTIVE

This research was aimed at utilizing the piezocone penetration tests (PCPT) and dissipation tests to evaluate the magnitude and time rate of consolidation settlement of fine-grained soils in Louisiana. This was achieved through the evaluation of the different deformation parameters. Therefore, this study focused on the following objectives:

- Evaluate the current interpretation methods for estimating the magnitude settlement of fine-gained soils through the determination of the compression or constrained modulus (M) from PCPT test data,

- Evaluate the applicability of the existing interpretation methods for estimating the overconsolidation ratio (OCR) from the PCPT test data,

- Evaluate the applicability of the existing interpretation methods for determining the vertical coefficient of consolidation (c_v) of cohesive soils utilizing the PCPT dissipation tests, by comparing the derived values of coefficient of consolidation from PCPT dissipation tests with the laboratory reference measured values.

- Verify the results of this study for estimating the magnitude of consolidation settlement by comparing the settlements predicted from PCPT tests with the field measurements from selected sites.

SCOPE

This research project focused on predicting the magnitude and time rate of consolidation settlement for normally consolidated soils through the evaluation of strength and deformation parameters utilizing the PCPT test results. The PCPT tests were conducted using the 60^{o} Fugro piezocones of cross-sectional areas of 10 and 15 cm^2 with pore pressure measurements at the cone tip (u_1) and the base (u_2). The PCPT tests were conducted at a penetration rate of 2 cm/sec. All the dissipation tests were conducted using the u_1 measurements. The u_2 measurements were used only to correct the cone tip resistance (q_t). The average PCPT measurements (q_c, u_1, 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 different PCPT interpretation methods. The dissipation tests (with u_1) were used to predict the vertical coefficient of consolidation (c_v) at different penetration depths. The results of this study were based on the comparison between the predicted consolidation parameters obtained from different interpretation methods and the laboratory measured parameters obtained from one-dimensional oedometer consolidation tests. However, the verification of the research findings was based on comparison between the predicted consolidation settlement and field settlements measured using settlement plates.

METHODOLOGY

Background

PCPT Measurements and Corrections

During the piezocone penetration test (PCPT), the cone tip resistance (q_c), sleeve friction (f_s), and pore water pressures measured at different locations (at the cone tip (u_1), behind the base(u_2), and behind friction sleeve (u_3)), are continuously recorded with depth. These measurements can be used for soil identification and the evaluation of different geotechnical soil properties. Due to the geometric design of the piezocone, the pore water pressures will act on the shoulder behind the base and at the both ends of friction sleeve, as shown in figure 1. This will influence the total stress measured from the cone tip and the friction sleeve. Therefore, the measured cone tip resistance and sleeve friction need to be corrected to account for the pore water pressure.

The corrected cone resistance, q_t, is given as:

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

where

$a = A_n/A_c$ is the effective area ratio of the cone,

A_n = cross-sectional area of the load cell,

A_c = projected area of the cone,

For the piezocones used in this study, a = 0.59.

The corrected sleeve friction, f_t, can be given as:

$$f_t = f_s - \frac{(A_{sb}u_2 - A_{st}u_3)}{A_s} \tag{2}$$

where

A_{sb} = bottom cross-sectional area of the friction sleeve,

A_{st} = top cross-sectional area of the friction sleeve,

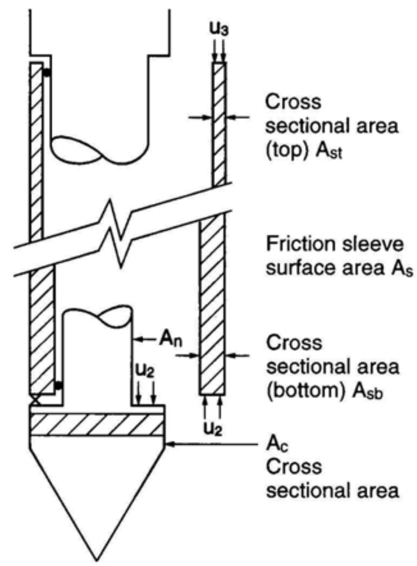


Figure 1
Effect of pore water pressure on cone tip resistance (q_c)
and sleeve friction (f_s)

A_s = surface area of friction sleeve

Since the pore water pressure behind the sleeve (u_3) is rarely measured, the correction to the sleeve friction can be made assuming equal pore pressures at each end of the sleeve. The ratio of the corrected to the measured sleeve friction usually ranges within ± 20 percent. However, the magnitude of correction can be reduced significantly if the end areas of the sleeve are equal.

Consolidation Characteristics

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

$$M = \frac{\partial \sigma'}{\partial \varepsilon} = \frac{2.3(1+e)\sigma'_v}{C_c} = \frac{1}{m_v} \tag{3}$$

where C_c is the compression index, e is the void ratio, and m_v is the coefficient of volume compressibility.

The total consolidation settlement (S_c) of fine-grained soils can be estimated utilizing the piezocone penetration test data through the evaluation of the constrained modulus (M) using the following equation:

$$S_c = \mathrm{H}\frac{\Delta\sigma}{M} \tag{4}$$

where H is the thickness of the compressible soil layer, and $\Delta\sigma$ is the applied stress.

The rate of consolidation can be calculated using the vertical coefficient of consolidation, c_v, that can be evaluated from the piezocone dissipation tests, as will be discussed in the following sections.

Constrained Modulus

The compressibility of the soil can be expressed by the constrained modulus (M), which varies with the effective stress (σ'_v) in different ways for various soil type. However, all variables are accounted for in the following general expression [20]:

$$M = mp_a\left(\frac{\sigma'_v}{p_a}\right)^{1-a} \tag{5}$$

Where m = dimensionless modulus number, p_a = reference stress (100 kPa), and a = stress exponent. For the preconsolidation stress range, a = 1, while a = 0 for normally consolidated stress range.

Several correlations have been developed to relate the laboratory measured constrained modulus (M) obtained from oedometer test, to the cone tip resistance (q_c). The general relationship can be expressed as follows:

$$M = \alpha \,.\, q_c \tag{6}$$

where q_c is the measured cone tip resistance.

Sanglerat developed 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, as shown in table 1 [7]. Jones and Rust found out that for South African alluvial clay, a value of $\alpha = 2.75 \pm 0.55$ can provide good correlation with M [10].

Senneset et al. [8] conducted correlation between the constrained modulus, M, and corrected cone tip resistance, q_t, as presented in figure 2 [8]. For silty soils, they obtained a linear correlation between q_t and constrained modulus (M) and they suggested the following equations:

$$M = 2\, q_t \qquad \text{for} \quad q_t < 2.5 \text{ MPa} \tag{7}$$

And

$$M = 4\, q_t - 5 \qquad \text{for} \quad 2.5 < q_t < 5 \text{ MPa} \tag{8}$$

Senneset et al. related the constrained modulus (M) by a linear interpretation of the net cone tip resistance (q_n) [8]. For the pre-consolidation range, they proposed the following relation:

$$M_p = \alpha_p \,.\, q_n = \alpha_p \,.\, (q_t - \sigma_{vo}) \tag{9}$$

Where α_p ranges between 5 and 15, σ_{vo} is the total overburden stress, and q_t is the corrected cone tip resistance.

Table 1
Estimation of constrained modulus, *M*, for clayey soils [7]

q_c (MPa)	$M = 1/m_v = \alpha .\, q_c$	
$q_c < 0.7$ $0.7 < q_c < 2.0$ $q_c > 2.0$	$3 < \alpha < 8$ $2 < \alpha < 5$ $1 < \alpha < 2.5$	Clay of low plasticity (CL)
$q_c > 2.0$ $q_c < 2.0$	$3 < \alpha < 6$ $1 < \alpha < 3$	Silts of low plasticity (ML)
$q_c < 2.0$	$2 < \alpha < 6$	High plastic silts and Clays (MH, CH)
$q_c < 1.2$	$2 < \alpha < 8$	Organic silts (OL)
$q_c < 0.7$ $50 < w < 100$ $100 < w < 200$ $w > 200$ w = water content	$1.5 < \alpha < 4$ $1 < \alpha < 1.5$ $0.4 < \alpha < 1.0$	Peat and organic clays (P_t, OH)

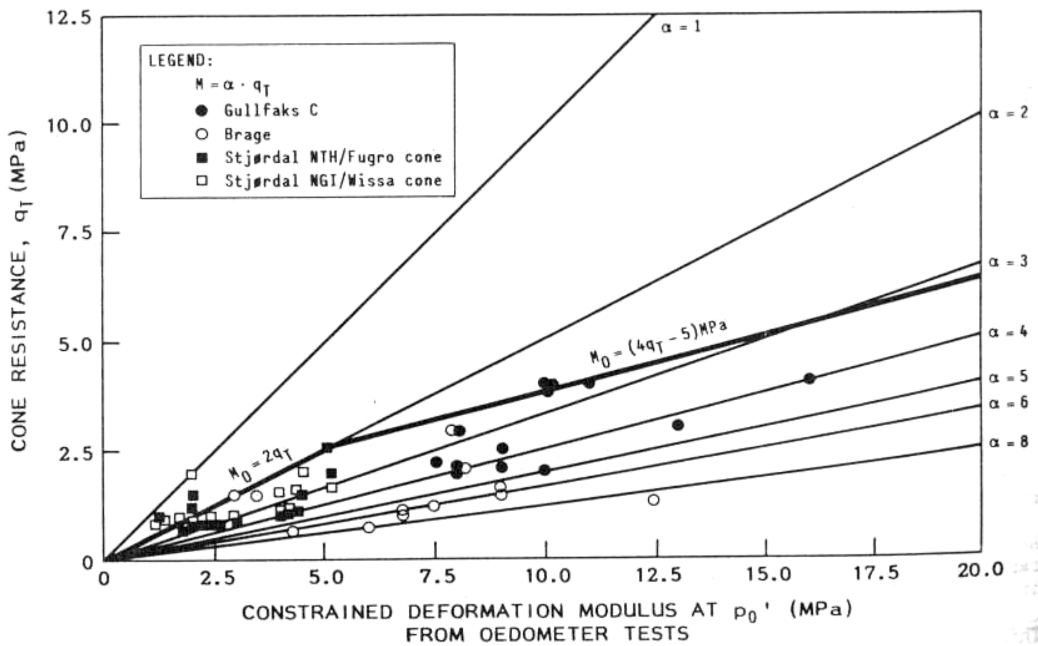


Figure 2
Constrained modulus (*M*) versus corrected tip resistance (q_t) [8]

Senneset et al. also propose the following relation for normally consolidated range [8]:

$$M_n = \alpha_n \cdot q_n = \alpha_n \cdot (q_t - \sigma_{vo}) \quad (10)$$

where $\alpha_n = 6 \pm 2$ for most clays.

For the Glava clays, Senneset et al. found that for the pre-consolidation range, the constrained modulus, M_p, compared well with the average interpretation of 10 q_n with a variation range of ± $5q_n$ as shown in figure 3 [8]. However, for the normal consolidation range, the constrained modulus, M_n, compared well with the upper limit of 8 q_n, as shown in figure 4. These examples demonstrate that compression moduli for clays can be predicted from semi-empirical relationships using CPT data.

Kulhawy and Mayne studied the relationship between the constrained modulus, *M*, and the net cone tip resistance (q_t - σ_{vo}) for different soils and suggested the following relation [9]:

$$M = 8.25 \cdot (q_t - \sigma_{vo}) \quad (11)$$

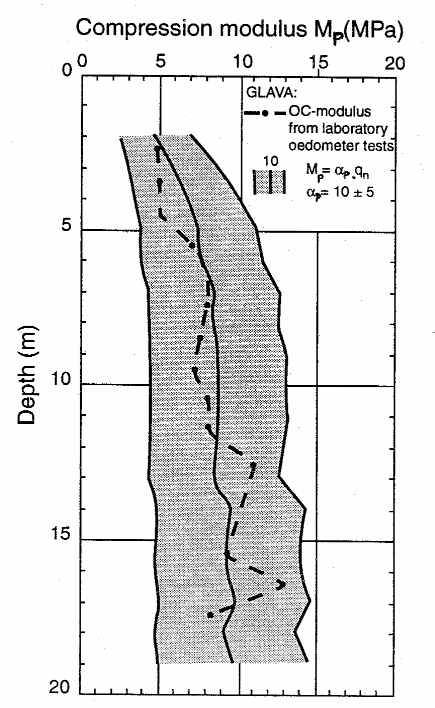


Figure 3
Comparison of modulus (M_p) for Glava clay [8]

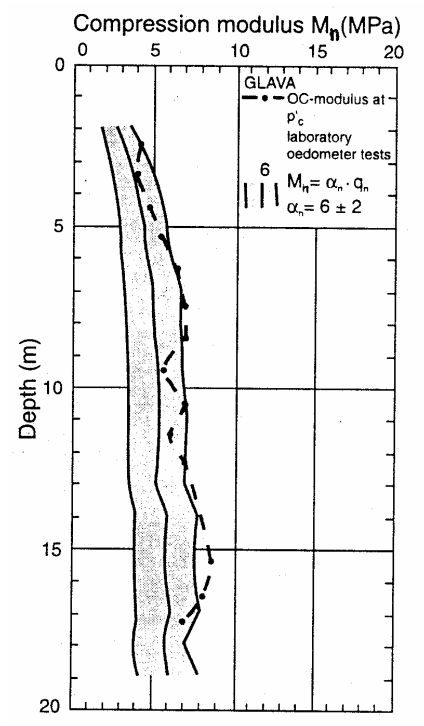


Figure 4
Comparison of modulus (M_n) for Glava clay [8]

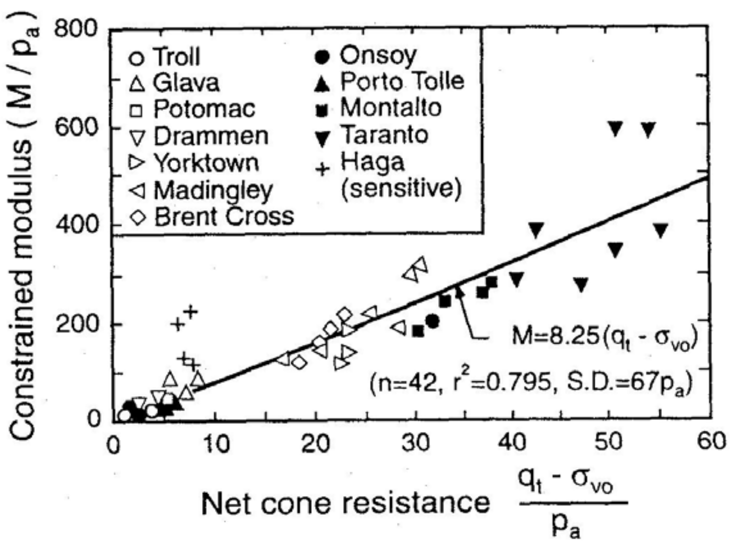


Figure 5
Relationship between constrained modulus and net cone resistance [9]

Figure 5 presents the general relationship between constrained modulus and net cone resistance as reported by Kulhawy and Mayne [9].

Even though these relations correlate well in some cases, local experience is essential to develop better correlation between cone tip resistance (q_c) and the constrained modulus (M) for different soil types with greater reliability.

For a stress range of $\sigma'_{vo} + \Delta\sigma'_v$, Senneset et al. suggested using the following relation to calculate the average constrained modulus, M_{av} [8]:

$$M_{av} = M\sqrt{\frac{\sigma'_{vo} + \Delta\sigma'_v / 2}{\sigma'_{vo}}} \qquad (12)$$

Coefficient of Consolidation

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

$$c_v = k\frac{M}{\gamma_w} \qquad (13)$$

The coefficient of consolidation (c) that is used to calculate the rate of soil settlement can be evaluated from the piezocone dissipation tests. The PCPT dissipation test consists of stopping the cone penetration and recording the dissipation of excess pore pressure (Δu) with time. The excess pore pressure is defined as the difference between the penetration pore pressure (u) and the static equilibrium pore pressure (u_o).

Several empirical, semi-empirical, and analytical methods have been developed to evaluate the consolidation characteristics of soils from the dissipation tests using the PCPT, based on the cavity expansion theories [e.g., 11, 21], the strain path method [e.g., 13], and the combination of the strain path method with the finite element technique [1].

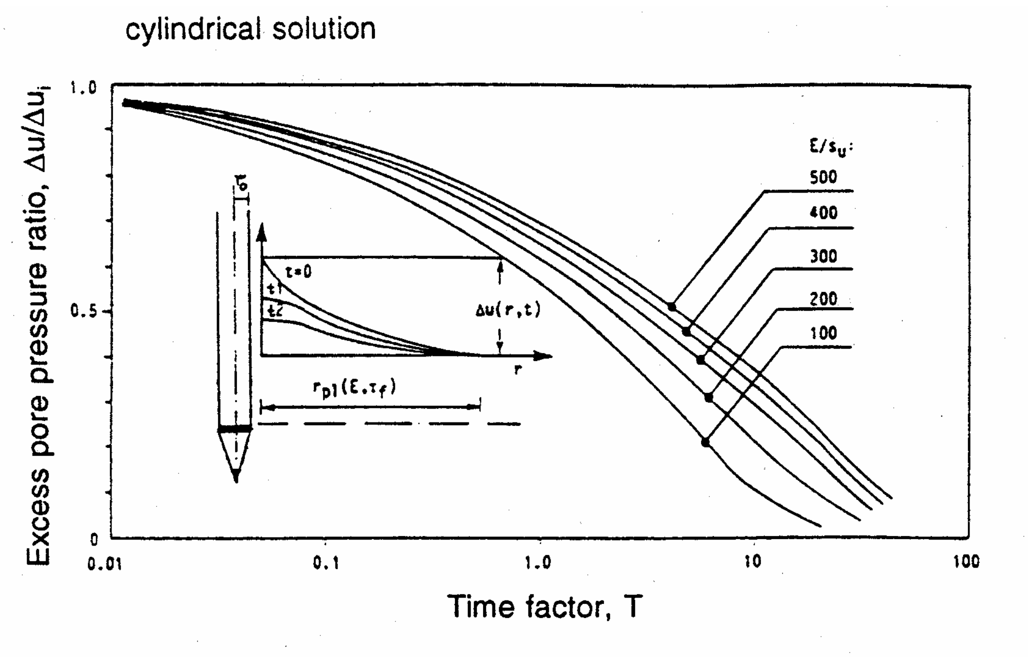
Cavity Expansion Method

Several investigators have used the cavity expansion theories (cylindrical and spherical) to model the piezocone penetration tests [e.g., 11, 21, 22]. The interpretation model developed by Torstensson assumes an elasto-plastic soil model and cylindrical or spherical cavity expansion theory to compute the initial excess pore pressure distribution [11, 21]. During the PCPT process, the soil along the penetrometer shaft is modeled using a cylindrical cavity expansion from zero to the piezocone radius, and at the cone tip the soil is modeled using a spherical cavity expansion from zero to the equivalent radius. Torstensson used a linear uncoupled one-dimensional consolidation to compute the dissipation of excess pore pressures [11 and 21]. Torstensson suggested that the coefficient of consolidation should be interpreted at 50 percent dissipation, and he proposed the following relation for the interpretation of the horizontal coefficient of consolidation (c_h) from piezocone dissipation tests:

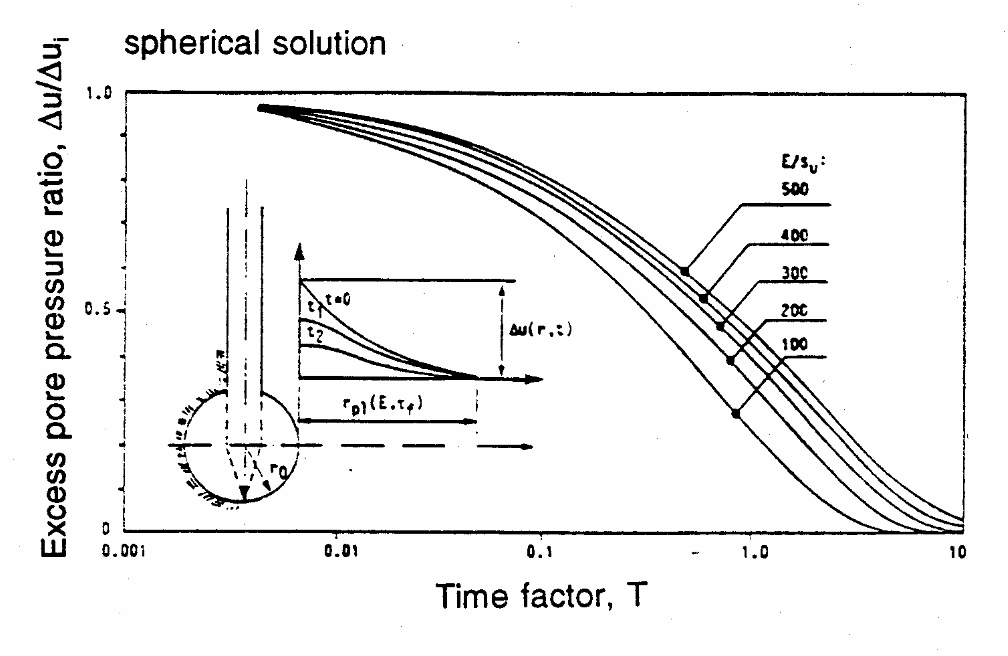
$$c_h(piez) = \frac{T_{50} r_o^2}{t_{50}} \tag{14}$$

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. The interpretation curves of the time factor (T) proposed by Torstensson for both cylindrical and spherical solutions are presented in figure 6 [11 and 21].

A similar equation was proposed by Senneset et al. [12] (method-a). The chart for time factor (T) is shown in figure 7. The time factor is a function of soil properties and degree of pore pressure dissipation, $\Delta u_t/\Delta u_i$; where $\Delta u_t = u_t - u_o$, and u_t is the pore pressure at a given time t.



(a)



(b)

Figure 6
Time factor for Torstensson's model: (a) cylindrical solution; (b) spherical solution

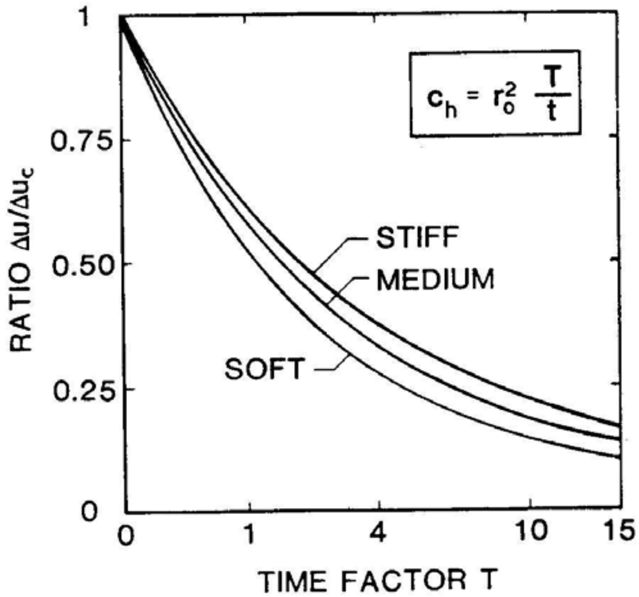


Figure 7
Interpretation of time factor, *T*, [12]

Teh and Houlsby Method

Teh and Houlsby developed a model to analyze the PCPT based on the combination of the strain path method with the large strain finite element analysis using an elastic-perfectly plastic material model of the Von Mises [1]. The strain path method was used to compute the initial distribution of excess pore pressures. The finite difference is used for the analysis of the dissipation excess pore pressure using the Terzaghi-Rendulic uncoupled consolidation theory. To include the effect of the soil stiffness (*I*), Teh and Houlsby introduced the modified time factor (*T**) as given in table 2 [1].The normalized dissipation curves at the cone face and cone shoulder are shown in figure 8. Teh and Houlsby proposed the following interpretation expression for the prediction of the horizontal coefficient of consolidation (c_h) [1]:

$$c_h(piez) = \frac{T^*_{50} r_o^2}{t_{50}} \sqrt{I_r} \tag{15}$$

where $I_r = G/s_u$ is the rigidity index, G is the shear modulus, and s_u is the undrained shear strength.

Table 2
Modified time factor (T^*) [1]

Degree of Consolidation	Location			
	Cone tip (u_1)	**Cone base (u_2)**	**5 radii above cone base**	**10 radii above cone base**
20	0.014	0.038	0.294	0.378
30	0.032	0.078	0.503	0.662
40	0.063	0.142	0.756	0.995
50	0.118	0.245	1.11	1.458
60	0.226	0.439	1.65	2.139
70	0.463	0.804	2.43	3.238
80	1.04	1.60	4.10	5.24

Table 3
Gradient of dissipation curve (M_G), root-time plot [14]

Filter Location	Cone tip (u_1)	Cone base (u_2)	5 radii above cone base
Dissipation curve gradient (M_G)	1.63	1.15	0.62

Teh Method

Teh proposed a method to interpret the coefficient of consolidation from the plot of pore pressure dissipation on square-root time and calculate the gradient of the initial linear section (m) as shown in figure 9 [14]. The horizontal coefficient of consolidation (c_h) can then be estimated using the follows equation:

$$c_h(piez) = (m/M_G)^2 \sqrt{I_r}\; r_o^2 \qquad (16)$$

Where M_G is a gradient of theoretical dissipation curve for a given penetrometer geometry and filter location as shown in table 3.

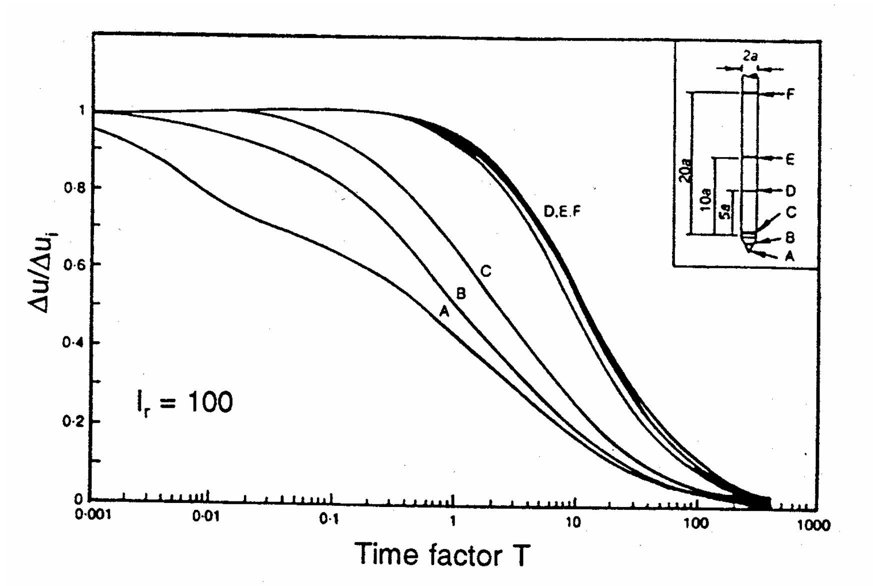


Figure 8a
Dissipation curves at different locations of a 60° cone penetrometer [1]

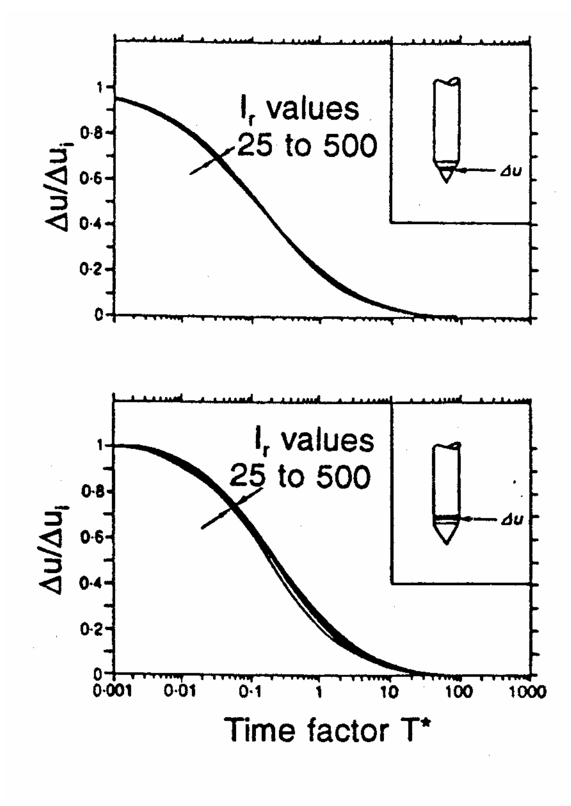


Figure 8b
Normalized dissipation curves plotted against *T [1]**

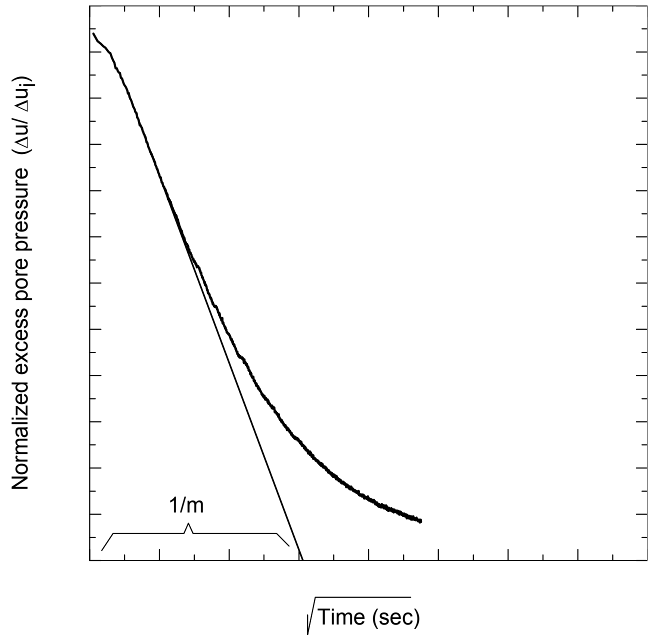


Figure 9
Calculating the gradient of initial linear section (*m*) (after Teh [1])

Senneset et al. Method-b

Senneset et al. [12] suggested an equation to predict $c_h(piezo)$ from the dissipation rate diagram as follows:

$$c_h(piezo) = \lambda_c r_o^2 \left| \Delta \dot{u}_t / \Delta u_i \right| \tag{17}$$

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 $t = 0$. Figure 10 describes the terminology for interpretation. The value for the rate factor λ_c can be obtained from figure 11. The rate factor is a function of soil properties and degree of pore pressure dissipation, $\Delta u_t/\Delta u_i$.

Since the dissipation of pore pressure occurs during recompression range (unloading) rather than in the normal consolidation range, Baligh and Levadoux suggested that the predicted $c_h(piezo) = c_h(overconsolidated)$ and they proposed the following relation to transfer $c_h(piezo)$ to normally consolidated condition $c_h(NC)$ [6]:

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

where

$$RR = \frac{c_r}{1+e_o} \qquad \text{and} \qquad CR = \frac{c_c}{1+e_o} \tag{19}$$

Where RR and CR are the recompression and compression ratios, respectively; c_r is the swelling index, c_c is the compression index, and e_o is the initial void ratio of soil.

Because of soil anisotropy, soil deposits typically have a greater horizontal hydraulic conductivity (k_h) than vertical hydraulic conductivity (k_v) and therefore in most cases the horizontal coefficient of consolidation is generally higher than the vertical coefficient of consolidation (i.e., $c_h > c_v$). It was indicated that c_h governs the consolidation process around the piezocone. The vertical coefficient of consolidation (c_v) can be calculated using the follows expression suggested by Levadoux and Baligh [13]:

$$c_{v(NC)} = \frac{k_v}{k_h} c_{h(NC)} \tag{20}$$

An estimation of the in-situ anisotropy of fine-grained soils (k_v/k_h) is difficult to obtain from laboratory tests because of the effects of sample size, sample disturbance, the presence of fissures and cracks, etc. [3]. Therefore, to estimate c_v a rough estimate of (k_v/k_h) can be used from suggested ranges of values of (k_v/k_h) for various soil types (Table 4) [6, 23, 24].

Table 4
Range of anisotropic hydraulic conductivity (k_h/k_v) of clays [23]

Nature of clay	**k_h/k_v**
No evidence of layering	1 to 1.5
Slight layering, e.g., sedimentary clays with occasional discontinuous lenses and layers of more permeable material	2 to 4
Varved clays and other deposits containing embedded and more or less continuous permeable layer	3 to 5

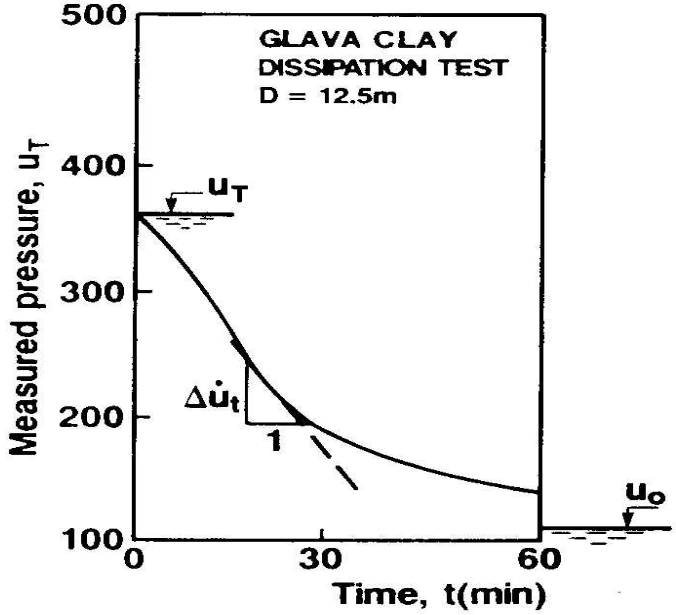


Figure 10
Terminology for interpretation of dissipation tests [12]

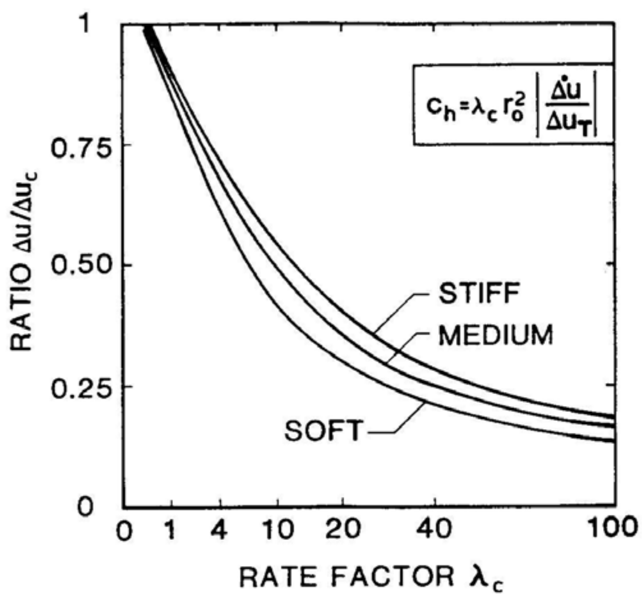


Figure 11
Interpretation of rate factor (λ_c) [12]

Overconsolidation Ratio

The overconsolidation ratio (*OCR*) and preconsolidation pressure (σ'_p) are considered fundamental characteristics of clayey soils that are needed for geotechnical engineering design to determine the deformation behavior of the soil under structures. They represent the stress history of the soil deposit. The *OCR* is defined as the ratio of the maximum past effective consolidation stress (preconsolidation pressure, σ'_p) and the existing effective overburden stress (σ'_{vo}). So, the *OCR* and the σ'_p are related to each other as follows:

$$\sigma'_p = OCR \cdot \sigma'_{vo} \qquad \text{or} \qquad OCR = \sigma'_p \,/\, \sigma'_{vo} \tag{21}$$

By knowing the *OCR*, the σ'_p can then be evaluated using the above equation.

The values of *OCR* and σ'_p have an important effect on the strength, stress-deformation and the compressibility characteristics of the soil. Hence, profiling the *OCR* or σ'_p is essential for the proper selection of relevant soil parameters for geotechnical design. The OCR and σ'_p are usually obtained from laboratory oedometer tests on soil samples obtained from the field.

The PCPT, which provides continuous measurements of q_c, f_s, and pore pressures (u_1 and u_2), can be a promising tool for estimating *OCR*. Several correlation methods, mostly empirical, are available in the literature attempting to evaluate the *OCR* from the PCPT data. These methods are based either on the undrained shear strength (s_u) [e.g., 16], or directly from the PCPT profile [e.g., 9, 17, 18, 19, and 25] using either tip resistance (q_c) or pore pressure (u_1 and u_2). These methods are described below:

I. Estimation of OCR from undrained shear strength

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

(1) Estimate s_u from CPT/PCPT data.

(2) Estimate the effective vertical pressure (σ'_{vo}) from soil profile

(3) Compute the ratio $S = (s_u / \sigma'_{vo}) = \left(\dfrac{q_t - \sigma_{vo}}{N_{kt}\ \sigma'_{vo}} \right)$, N_{kt} is the cone factor

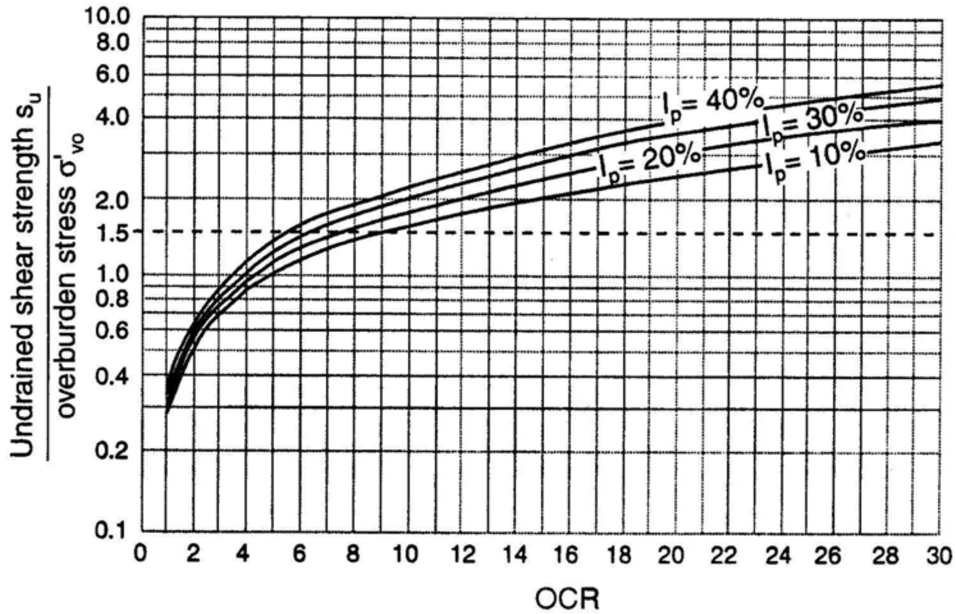


Figure 12
Relationship between s_u/σ'_{vo} and *OCR* [27]

(4) Estimate the corresponding normally consolidated value $S_n = (s_u/\sigma'_{vo})_{NC}$ from the plasticity index (I_p) using Skempton relation [26]:

$$S_n = (S_u/\sigma'_{vo})_{NC} = 0.11 + 0.0037\, I_p \tag{22}$$

If plasticity index is not known, an average value of $s_u/\sigma'_{vo} = 0.3$ for NC soil can be used.

(5) Estimate the *OCR* using a correlation chart shown in figure 12 [27] or using the following relation:

$$OCR = \left(\frac{S}{S_n}\right)^{1.13 + 0.04\,(S/S_n)} \tag{23}$$

II. Estimation of OCR from PCPT data

Several methods have been proposed to interpret the OCR of clays from PCPT data. These methods are based on cone tip resistance (q_c), pore pressure measurements (u_1, u_2), or a combination of both.

Mayne and Holts suggested correlating OCR and the normalized cone tip resistance with respect to effective stress [$(q_c - \sigma_{vo})/\sigma'_{vo}$] and obtained the following relationship [28]:

$$\mathrm{OCR} = 0.4\left(\frac{q_c - \sigma_{vo}}{\sigma'_{vo}}\right) \tag{24}$$

Kulhawy and Mayne [9], Chen and Mayne [18], Powell et al. [29], and Leroueil et al. [30], related the *OCR* with the normalized net tip resistance [$(q_t - \sigma_{vo})/\sigma'_{vo}$] and suggested the following equation to estimate the *OCR* from the PCPT data:

$$\mathrm{OCR} = k_t\left(\frac{q_t - \sigma_{vo}}{\sigma'_{vo}}\right) \tag{25}$$

Where the value of k_t seems to be soil type and site dependent. Based on a study on some clayey sites in United Kingdom, Powell et al. found that k_t ranges from 0.2 to 0.24 for non-fissured clay, and ranges from 0.91 to 2.22 for heavily overconsolidated fissured clay [29]. Lutenegger and Kabir, however, obtained a mean value of 0.30 for postglacial marine clays in New York [31]. Kulhawy and Mayne found a good correlation with $k_t = 0.33$ [9]. Sugawara suggested that for soils with fine content ≥ 80 percent the value of $k_t = 0.33$ represents an upper limit of OCR [32]. For eastern Canadian clays, Leroueil et al. proposed a value of $k_t = 0.28$ [30]. Chen and Mayne obtained a value of $k_t = 0.32$ with scattered results and low coefficient of determination (R^2 = 0.67) [18].

Using the effective stress approach, Chen and Mayne suggested the following simplified relation to estimate *OCR* from piezocones with pore pressure element at the tip (u_1) [18]:

$$\mathrm{OCR} = k_1\left(\frac{q_t - u_1}{\sigma'_{vo}}\right) \tag{26}$$

where $k_1 = 0.81$. For piezocones with pore pressure element at the base, u_2, Chen and Mayne proposed the following expression to estimate *OCR* [18]:

$$\mathrm{OCR} = k_2\left(\frac{q_t - u_2}{\sigma'_{vo}}\right) \tag{27}$$

where $k_2 = 0.46$. Similar expression was also suggested by Konrad and Law to estimate OCR with $k_2 = 0.49$ [17].

In piezocones where pore pressures are measured on both the cone tip and behind the base, Sully, et al. proposed the use of the normalized pore pressure difference, *PPD*, to estimate the *OCR* and suggested the following relation [19]:

$$OCR = 0.66 + 1.43\ (PPD) \qquad (28)$$

Where *PPD* is the pore pressure difference defined as:

$$PPD = \frac{u_1 - u_2}{u_o} \qquad (29)$$

Where u_1 is the pore pressure measured near the tip, and u_2 is the pore pressure measured just behind the base. This relation does not seem to be valid for *OCR* less than 10 [33]. However, Sully, et al. suggested that a better correlation can be obtained by using u_3 instead of u_2 [19]. Mayne et al. tried to find a correlation between *OCR* and the ratio u_2/u_1, but the results were not encouraging [34].

Other researchers [9, 35, 36] examined a correlation between *OCR* and the excess pore pressure normalized with respect to effective overburden pressure, σ'_{vo}, as follows:

$$OCR = k_u \frac{u - u_o}{\sigma'_{vo}} \qquad (30)$$

Where $u = u_1$ *or* u_2. The use of u_1 is most widely used because of the fact that u_2 values can be zero or negative in very stiff and heavily fissured clays [37]. For inorganic Scandinavian clays, Larsson and Mulabdic obtained a value of $k_u = 0.29$ for u_1 measurement, and k_u value of 0.3 to 0.4 for u_2 measurement in their correlation [35]. Based on correlation using u_2 measurement, Kulhawy and Mayne obtained a value of 0.54 for k_u [9], while Mayne and Kulhawy [36] proposed a value of 0.4 for k_u [36].

Due to similarities between the pore pressure responses in the undrained triaxial test and in the piezocone test, Wroth introduced the pore pressure ratio B_q, similar to Henkel's *a* parameter defined as [38]:

$$B_q = \frac{\Delta u}{q_t - \sigma_{vo}} \qquad (31)$$

Where $\Delta u = u - u_o$. The B_q ratio is highly dependent on the drainage characteristic and the stress history of the soil. Therefore, B_q can be a useful index to estimate the OCR [38]. The following expression was suggested:

$$OCR = \frac{2.3\ B_q}{(3.7B_q\ \text{-}1)} \tag{32}$$

Demers and Leroueil pointed out that the correlation of OCR using pore pressure data usually gives more scattered results than those using cone tip resistance [33].

Based on the combination of cavity expansion and critical state theory, Mayne suggested the following expression for estimating OCR [39]:

$$OCR = 2\left[\frac{1}{1.95M_c + 1}\left(\frac{q_t - u_2}{\sigma'_{vo}}\right)\right]^{1.33} \tag{33}$$

$$OCR = 2\left[\frac{1}{1.95M_c}\left(\frac{q_t - u_1}{\sigma'_{vo}} + 1\right)\right]^{1.33} \tag{34}$$

Where M_c is the slope of the critical state line given as

$$M_c = \frac{6\sin\varphi'}{3 - \sin\varphi'} \tag{35}$$

Undrained Shear Strength

The undrained shear strength, s_u, can be estimated from the CPT data using the following equation:

$$s_u = \frac{q_t - \sigma_{vo}}{N_k} \tag{36}$$

Where q_t is the corrected cone tip resistance, σ_{vo} is the total overburden pressure, and N_k is an empirical cone factor. Rad and Lunne showed that the N_k factor varies from 8 to 29 [40].

The wide range of the N_k factor requires that local correlation have to be well established for better prediction of s_u. The N_k factor is usually determined by calibration to a reference value of s_u obtained either from field or laboratory tests.

CPT Soil Classification

Several methods have been used to classify soil utilizing CPT data [e.g., 37, 41, 42, 43, 44]. The probabilistic region estimation method developed by Zhang and Tumay was used in this study to classify the soil based on CPT data [44]. This method is similar to the classic soil classification methods since it is based on soil composition. In this method, a conformal transformation is used to determine the soil classification index (U) from the CPT sounding parameters, the cone tip resistance, q_c, and friction ratio, R_f. A statistical correlation was established between the U index and the compositional soil type given by the Unified Soil Classification System (USCS). A normal distribution of U was established for each reference USCS soil type (GP, SP, SM, SC, ML, CL and CH). Each U value corresponds to several soil types with different probabilities. Soil types were further rearranged into three groups: sandy and gravelly soils (GP, SP, and SM), silty soils (SC and ML), and clayey soils (CL and CH).

The Zhang and Tumay method provides a profile of the probability or chance of having each soil type group (sandy, silty and clayey) with depth; this shows the chance of misclassifying the soils, similar to other CPT soil classification methods [44]. This method was implemented into a FORTRAN code as well as a Visual Basic code to facilitate its use.

Laboratory and In-Situ Tests

The main objective of this study was to evaluate the current interpretation methods for their ability to estimate the total settlement and time rate of settlement from piezocone penetration and dissipation tests. To achieve this goal, several sites were selected in Louisiana to conduct in-situ field and laboratory tests. Seven of these sites were used to evaluate the different PCPT interpretation methods, possibly develop new correlations, and estimate the consolidation parameters of fine-grained soils (M, OCR, c_v). The other three embankments sites were used for verification by comparing the predicted settlements with the field measured settlements. Figure 13 depicts a map of Louisiana with approximate locations of the sites selected for this study. This section describes the laboratory and field testing program and the soil profiles of the investigated sites.

Laboratory Tests

In each of the investigated sites, boreholes were drilled and high quality 7.6 cm (3 in.) shelby tube samples were recovered at different depths for comprehensive laboratory testing. The laboratory testing program included basic soil characterization tests such as water content, unit weight, Atterberg limits, grain size distribution including hydrometer tests, and the specific gravity. One-dimensional oedometer consolidation tests were also performed on undisturbed samples oriented in both vertical and horizontal directions. These tests evaluated the reference consolidation parameters of the soil in both directions and the ratio of vertical to horizontal coefficient of consolidation (c_v/c_h) for the different sites. The reference soil parameters include the vertical and horizontal coefficient of consolidation, c_v and c_h, the tangent constrained modulus, M, the overconsolidation ratio (OCR) and the compression indices, c_c, c_r. The unconfined compression tests and the k_o-consolidated undrained triaxial tests (Ck_oU) were also performed to estimate the undrained shear strength (s_u) and the shear modulus (G) of the soil. Table 5 presents a summary of the geotechnical properties of the subsurface soil obtained from the laboratory tests at the different investigated sites. Some of the consolidation test results for samples obtained from 4.5m to 6.0m deep at the Evangeline site are presented in figures 14 through 16.

In-Situ Tests

The in-situ testing program included performing both PCPT and piezocone dissipation tests. Two state-of-the-art cone penetration systems are available at the Louisiana Transportation

Research Center (LTRC). These systems are the 20-ton Research Vehicle for Geotechnical In-situ Testing and Support (REVEGITS) [45], and the Continuous Intrusion Miniature Cone Penetration Test (CIMCPT) system [46]. Figure 17 presents a photograph of REVEGITS and CIMCPT systems. 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, 1m segmental rods, cone penetrometers (10 and 15 cm^2), and a data acquisition system. The REVEGITS system was used in this study for in-situ piezocone penetration and dissipation testing.

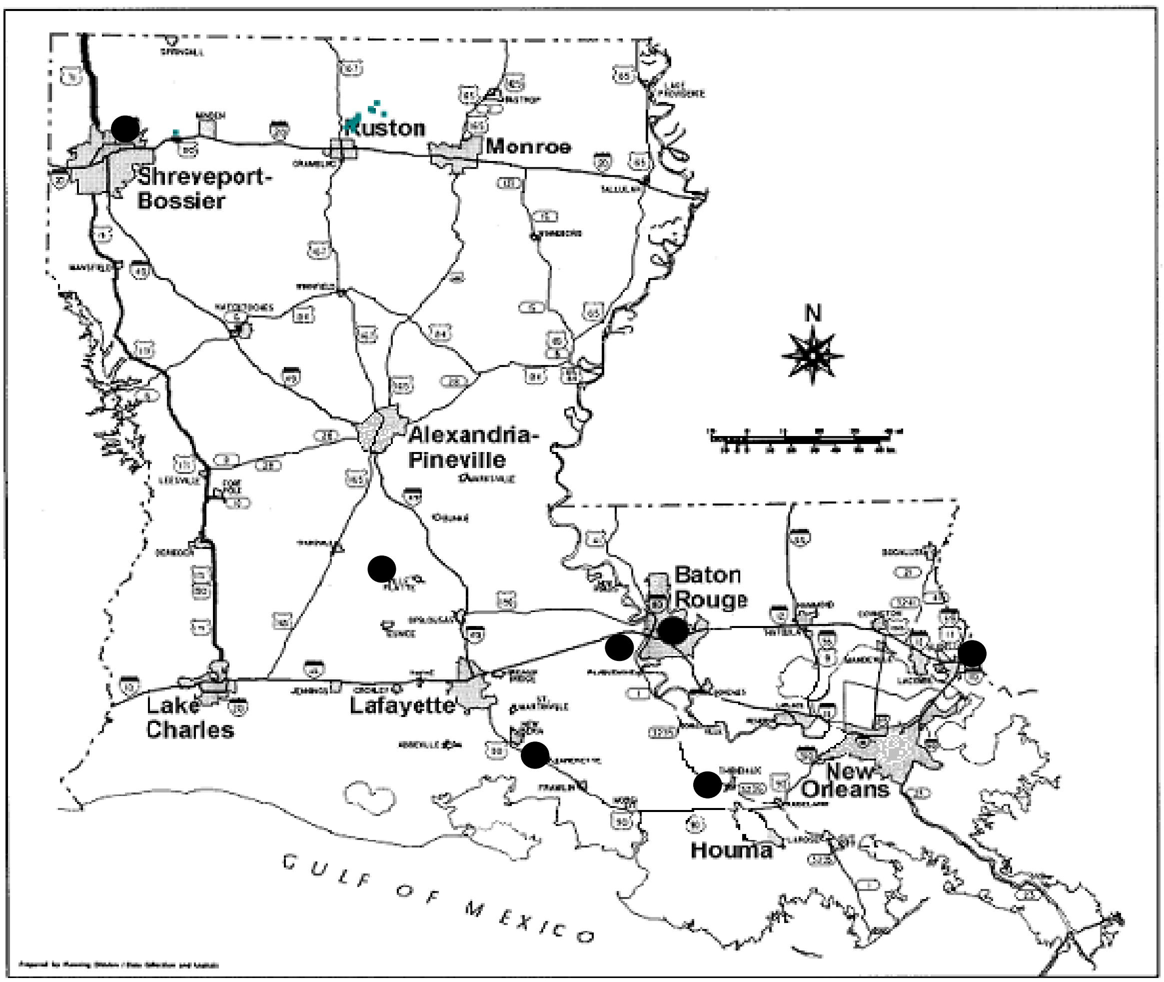


Figure 13
Louisiana state map with approximate locations of the tested sites

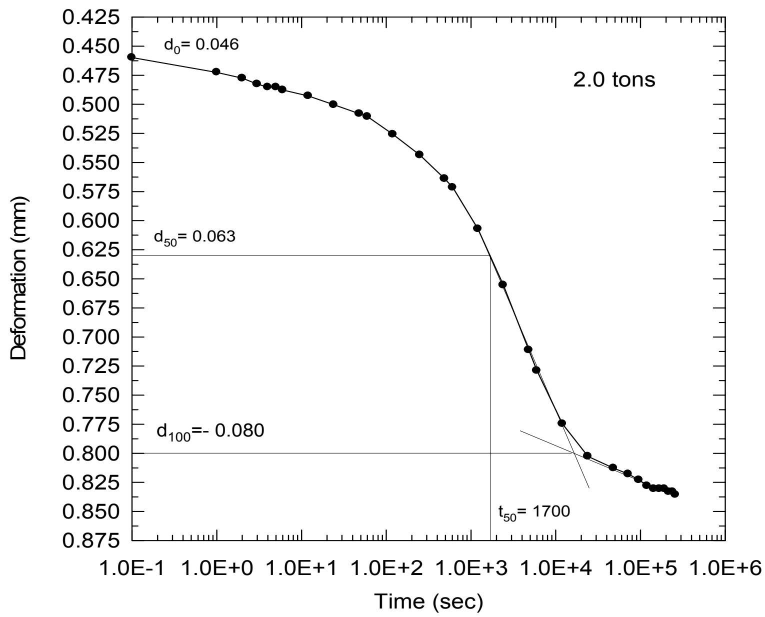
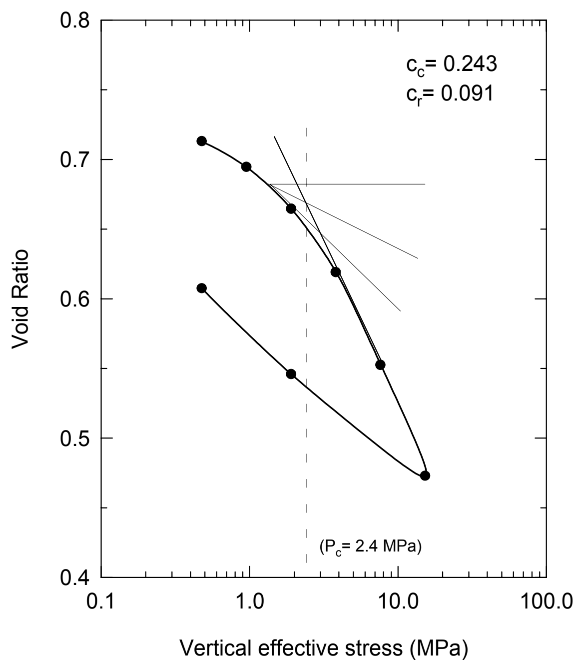
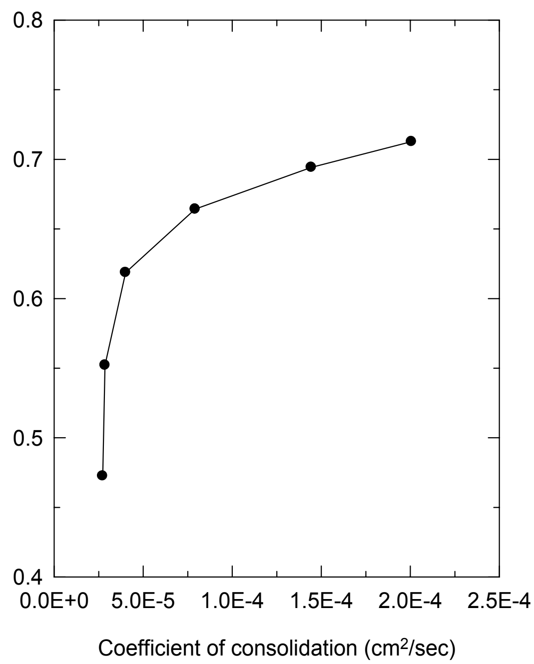


Figure 14
Results of one-dimensional consolidation test for Evangeline site, depth = 4.5-6 m



(a)



(b)

Figure 15
Consolidation test results for Evangeline site, depth = 4.5-6 m; (a) vertical effective stress (σ_v') versus void ratio; (b) coefficient of consolidation (c_v) versus void ratio (e)

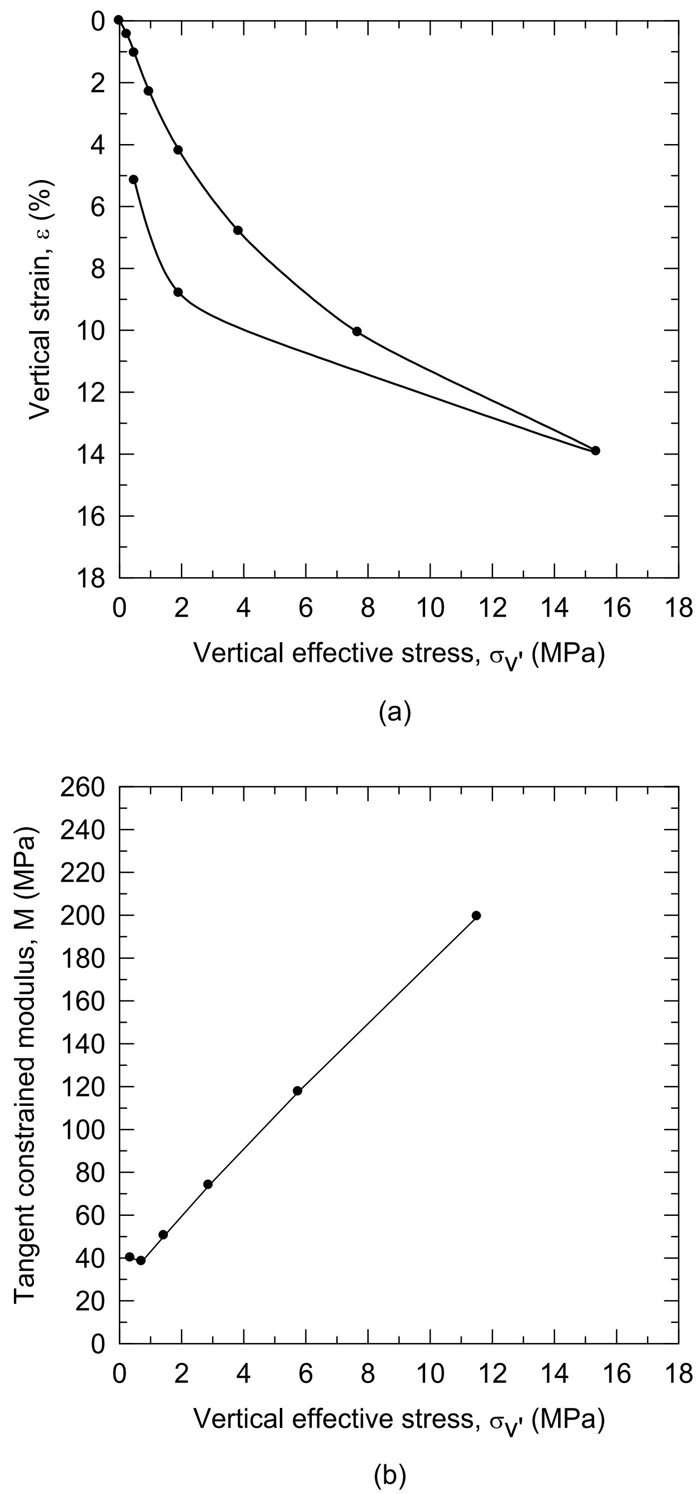


Figure 16
Results of consolidation test for Evangeline site, depth = 4.5-6 m; (a) vertical effective stress (σ_v') versus vertical strain, (b) σ_v' versus tangent constrained modulus (*M*)

Table 5
Summary of soil properties for the investigated sites

Site	Unit weight (kN/m^3)	Water content (%)	Liquid Limit (%)	Plasticity Index	Clay content (%)	Su (kN/m^2)	OCR
Manwell Bridge Evangeline	16 – 20 (18.5)	17 – 48 (32)	23 – 77 (48.9)	6 – 44 (25)	17 – 66 (42.3)	29 – 142 (71)	1 – 5.2
US 90 – La 88 New Iberia	18.2–18.8 (18.3)	23 – 33 (25.5)	30 – 35 (33.2)	9 – 17 (12)	22 – 26 (24.3)	38 – 118 (87)	1.2 – 4.3
LA Peans canal bridge Lafourche	15 – 19 (16.8)	29 – 61 (38.8)	34 – 66 (46.8)	13 – 39 (21.4)	42 – 57 (52.2)	12.5 – 48 (28.4)	1 – 3.4
PRF	16–16.9 (16.6)	31–63 (49.1)	64 – 115 (91.7)	25 – 41 (31.8)	25 – 45 (41.4)	18.3– 43.9 (25.7)	2 – 16.5
Pearl River	15 – 18.5 (16.2)	21 – 45 (32.2)	42 – 64 (53.6)	22 – 39 (30.3)	26 – 68 (43.6)	14.5–43.9 (25.7)	1.5 – 9.8
East Airport Baton Rouge	16.5 – 19 (17.6)	12.4-28.1 (20)	30 – 41 (33.7)	12 – 23 (16.8)	26.2-69.6 (51)	38.3–118 (80.8)	3.5 - 21
Flat River Bossier	15.8–19.2 (17.4)	29.5–46.0 (36.1)	44 – 81 (63.6)	25 – 49 (36)	41.2–83 (66.6)	43.2–75.9 (54.8)	1 – 5.84

At each site, several in-situ PCPTs were performed around the drilled boreholes using the10 cm^2 and 15 cm^2 piezocone penetrometers. The piezocones used in this study are subtraction Fugro type cone penetrometers. The 10 cm^2 piezocone has a sleeve area of 150 cm^2 with a pore pressure transducer located 5 mm behind the base (u_2 configuration), while the 15 cm^2 piezocone has a sleeve area of 200 cm^2 with two pore pressure transducers located on the cone face and behind the sleeve (u_1 and u_3 configuration). The schematics of the 10 cm^2 and 15 cm^2 piezocone penetrometers are depicted in figure 18. During PCPT tests, the piezocone was pushed at the rate of 2 cm/sec, and the data was collected every 2 cm. The 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 15 cm^2 piezocone provided measurements of q_c, f_s, and pore water pressure at the cone tip, u_1. The profile of PCPT tests were then used to identify the soil type, evaluate the undrained shear strength, s_u, tangent constrained modulus, M, and overconsolidation ratio, OCR for each site using different interpretation methods.

To perform dissipation tests with respect to time, the penetration of the piezocone was arrested at previously specified penetration depths that corresponded to the same depths of the recovered samples. The dissipation tests were then used to estimate the horizontal and vertical coefficient of consolidation, c_h, and c_v, respectively, based on different interpretation methods.

Figure 17
Louisiana cone penetration systems: REVEGITS cone truck on the left and CIMCPT cone truck on the right

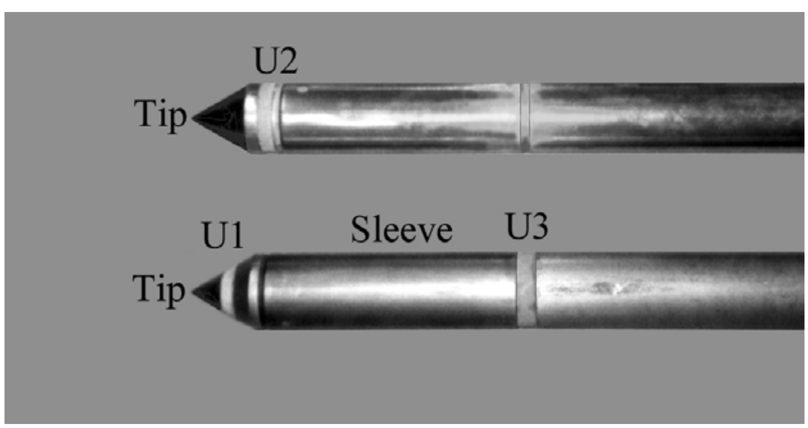


Figure 18
10 cm^2 and 15 cm^2 piezocone penetrometers

Investigated Sites

Seven Louisiana sites were selected to evaluate the different PCPT interpretation methods and develop new correlations. Three other sites with field measured settlements were selected for verification. Table 5 presents a summary of the soil properties for the investigated sites. Brief descriptions of theses sites with the field and laboratory test results are discussed below:

Manwell Bridge, Evangeline Site

The Manwell Bridge is located at about 20 miles northwest of Opelousas. The results of a soil boring at this site indicated that the soil profile consists of 1.5 m of brown silty sand, followed by a layer of medium brown and gray clay from 1.5 m to 6.5 m, brown lean clay from 6.5 m to 7.5 m, and a layer of brown silt and sand from 7.5 m to 10.5m. Underneath it, there is a layer of brown and gray clay with lenses of silt from 10.5 m to 13.5 m, followed by a 1.5 m thick gray sand layer. A gray and brown clay layer lays from 15m to 18.5m, and lean clay with some lenses of silt exists from 18.5m to 21.5m. This is followed by gray silt and silty sand layers from 21.5 m to 26.0 m. The soil profile and the corresponding soil properties are presented in figure 19 and table 5, row 1. The results of laboratory tests show that the water content ranges from 17 percent to 48 percent and the clay content ranges from 17 percent to 66 percent. The undrained shear strength, s_u, however, ranges from 29 - 142 kPa. The *OCR* varies with depth, from 5.2 at about 3.5 m to 1.0 at about 15 m. The results of four triaxial tests conducted on undisturbed samples recovered from different depths are presented in figure 20. The rigidity index (I_r) for this site was estimated to be 40.

Three PCPT tests were conducted at the Manwell Bridge site, two PCPT using u_1 measurements and one PCPT using u_2 measurement. The profiles of two PCPT test results are presented in figure 21. Column 1 presents the corrected cone tip resistance, q_t, profile. Column 2 presents the sleeve friction (f_s) profile. Column 3 presents the friction ratio (R_f) profile, which is the ratio between the sleeve friction and tip resistance. Column 4 presents the pore pressure profiles of u_1 and u_2. These data will be used later to predict the constrained modulus, *M*, and OCR. Figure 21 also describes the soil classification using the CPT probabilistic region estimation method developed by Zhang and Tumay [44]. The results of eight dissipation tests conducted at different depths (3.7 m, 5.26 m, 6.46 m, 12.6 m, 19.3 m, 20.78 m, and 22.13 m) are presented in figure 22, which will be used later to predict the vertical coefficient of consolidation, c_v. The water table at this site was at about 2 m.

US 90 - La 88 Interchange Site - New-Iberia

This site is located 10 miles south of New Iberia at the US 90 interchange at LA highway 88. The soil profile at the New Iberia site consists of stiff to medium silty clay soils down to 7.5 m, silty sand and sandy soils from 7.5 m to 12.0 m interbedded with thin layers of silty clay, silty clay soil from 12.0 m to 13.3 m, and sandy soils down to 16.0 m. The soil profile and laboratory soil properties of the New Iberia site are presented in figure 23 and table 5, row 2. The soil has moisture content ranges from 23 percent to 33percent and clay content ranges from 22 percent to 26 percent. The OCR varies from 4.3 near the surface to 1.2 at about 7 m. The undrained shear strength, s_u, ranges from 38 to 118 kPa. The results of two triaxial tests conducted on undisturbed samples recovered from different depths are presented in figure 24. The rigidity index was estimated to be I_r = 50.

Three PCPT tests were also conducted at this site, two using u_1 and one using u_2 measurements. Figure 25 presents the profiles of PCPT test data (q_t, f_s, R_f, u_1 and u_2) and the corresponding CPT soil classification for the New Iberia site [44]. Five dissipation tests were conducted at this site at depths of 1.8 m, 2.8 m, 4.28 m, 5.8 m, and 7.24 m. The water table was located at about 1.5 m. The dissipation tests are presented in figure 26.

LA Peans Canal Bridge Site - Lafourche

The LA Peans canal bridge site located five miles southeast of Thibodaux was selected for this study. The soil boring showed that the profile consists of medium silty clay to 4 m, which is underlain by a silty sand layer from 4 to 5.5 m, and soft to medium silty clay and clay soils from 5.5 m to 12 m. This is followed by a silty sand layer interbedded with lenses of silty clay down to about 15.5 m. The soil profile and laboratory soil properties of the LA Peans site are presented in figure 27 and table 5 row 3. The silty clay soil in this site has moisture content ranges from 29 percent to 61 percent and clay content ranges from 42 percent to 57 percent. The OCR varies from 3.4 near the surface to 1 at about 7.5 m. The undrained shear strength, s_u, ranges from 12.5 to 48 kPa. The results of two triaxial tests conducted on undisturbed samples recovered from different depths are presented in figure 28. The rigidity index was estimated to be I_r = 35.

The results of the PCPT tests conducted at this site are presented in figure 29. This figure includes profiles of tip resistance, q_c, sleeve friction, f_s, friction ratio, R_f, and pore water pressure profiles, u_1 and u_2. Figure 29 also describes the soil classification using the CPT probabilistic region estimation method [44]. The results of four dissipation tests conducted at different depths

(2.5 m, 7.0 m, 8.5 m, and 11.0 m) are presented in figure 30. The water table at this site was located at about 1.75 m.

Pavement Research Facility Site

The Pavement Research Facility (PRF) is located two miles west of Baton Rouge. This site was used evaluate the PCPT interpretation methods and to verify settlement prediction. The boring profile and soil properties of the PRF site are presented in figure 31 and table 5, row 4. The soil deposit consists of 3.6 m of medium brown and gray silty clay layer, a stiff clay layer from 3.6 m to 5.5 m, soft to medium gray clay from 5.5 m to 6.7m, followed by alternating layers of sandy and silty clay soils from 6.7 to 10.5 m, and sandy layers from 10.5 to 16.0 m. The moisture contents ranged from 31 percent to 63 percent. The clay content ranges from 25 percent to 45 percent.The undrained shear strength (s_u) ranges from 18.3 to 43.9 kPa. The OCR varies from 16.5 at 0.5 m to 2 at 6.5 m depth. The rigidity index for the PRF site is $I_r = 30$. The unconfined compression tests conducted on undisturbed samples recovered from different depths are presented in figure 32.

The profiles of PCPT test results (q_t, f_s, R_f, u_1 and u_2) and the corresponding CPT soil classification using Zhang and Tumay [44] method are presented in figure 33 [44]. Six dissipation tests were conducted at the PRF site at 1.66 m, 2.64 m, 3.32 m, 3.8 m, 4.36 m and 5.08 m depths. The water table at the PRF site was about 1.0 m below the surface. Figure 34 depicts the results of these dissipation tests. It is interesting to notice that some of the dissipation tests at this site show initial increase in pore pressures (during the first 20 to 30 sec.) before real dissipation starts. This type of abnormal dissipation curve is usually observed with u_2 measurement in overconsolidated soils. Interpretation of these types of dissipation curves will be discussed later.

Pearl River Bridge Site

The Pearl River Bridge is located at I-10 near the border between Louisiana and Mississippi. The layout of the soil boring and PCPT test locations conducted at the east bank of Pearl River are shown in figure 35. The soil boring indicates that the soil deposit consists of 0.6 m of loose tan fine sand, followed by medium stiff tan and gray sandy clay to 1.5 m below the surface. A layer of soft gray silty clay with clay layers and wood lies from 1.5 m to 4.0 m. Below that layer, a very soft gray silty clay with wood exists from 4.0 m to 6.4 m, and stiff gray clay with wood exists from 6.4 m to 9.1m. Underneath this lies loose tan fine sand. The soil profile and the corresponding soil properties are presented in figure 36 and table 5, row 5. The results of laboratory tests show that the water content ranges from 21 percent to 45 percent and the clay content ranges from 26 percent to 68 percent. The undrained shear strength, s_u, ranges from 14.5 – 43.9 kPa. The OCR varies with depth from 9.8 near the surface to 1.5 at about 5.5 m depth. The results of two triaxial tests conducted on undisturbed samples recovered from different depths are presented in figure 37. The rigidity index of this site was estimated to be $I_r = 22$.

Three PCPT tests were performed at the east bank of Pearl River as shown in figure 38. During piezocone penetration, the PCPT test data (q_t, f_s, u_1 and u_2) were recorded at 5-cm depth intervals. Pore pressure measurements were recorded using u_2 configuration during PCPT-1, but pore pressure measurements were recorded using u_1 configuration during PCPT-2 and PCPT-3. The PCPT profiles and the corresponding Zhang and Tumay CPT soil classification with depth for the Pearl River site are presented in figure 38 [44]. Pore pressure dissipation tests were performed at six depths during the performance of PCPT-2 and PCPT-3. Dissipation tests were performed at the 1.68, 2.60, and 4.42 m depths in PCPT-2, and at the 6.25, 7.15, and 9.0 m depths in PCPT-3. The water table was at about 1.0 m depth. The results of the dissipation tests are presented in figure 39. The dissipation test curve obtained at 2.6 m showed an initial increase in pore pressure before real dissipation started, similar to the results obtained at PRF site.

East Airport Site

This site is located at 300 East Airport Road in Baton Rouge. Five boreholes were drilled in the site with depths up to 10 m. The results of soil boring indicate that the soil deposit consists of 1.5 m of gray clay with organic traces and clayey sand with layers of sand from 1.5 m to 2.5 m, followed by brown stiff clay layer down to 4.0 m. A layer of medium clay with clayey sand layers exist from 4.0 m to 5.7 m, followed by stiff clayey sand from 5.7 m to 7.0 m, and dense sand below that down to 10 m. Underneath this lies loose tan fine sand. The soil profile and the corresponding soil properties are presented in figure 40 and table 5, row 6. The results of laboratory tests show that the water content of soil ranges from 12.4 percent to 28.1 percent, the clay content ranges from 26.2 percent to 69.6 percent, and the undrained shear strength, s_u, ranges from 38.3 to 118 kPa. The *OCR* varies with depth from 21 near the surface to 3.5 at about 7.0 m. The results of triaxial tests conducted on undisturbed samples are presented in figure 41. The rigidity index was estimated to be $I_r = 30$. The profiles of PCPT test data and the CPT soil classification with depth are presented in figure 42 [44]. The water table in this site was at about 1.0 m. The dissipation tests conducted at different depths (1.5 m, 3.2 m, 4.7 m, 6.1 m and 6.74 m) are also presented in figure 43.

Flat River-Bossier Site

The site is located on the east bank of the Flat River in Bossier City. The soil profile at this site consists of soft to medium silty clay soils down to 4.6 m and medium to stiff heavy clay from 4.6 to 8 m, followed by sand underneath it. The soil profile and laboratory soil properties of the Flat River site are presented in figure 44 and table 5, row 7. The soil has moisture content ranges from 29.5 percent to 46 percent and clay content ranges from 41.2 percent to 83 percent. The *OCR* varies from 5.84 near the surface to 1 at about 6 m. The undrained shear strength (s_u) ranges from 43.2 to 75.9 kPa. The profiles of PCPT test results and the corresponding CPT soil classification with depth are presented in figure 45 [44]. The water table of this site was deeper than the clay layer, as seen in the pore pressure profile. Therefore, dissipation tests were not conducted in this site, and only the relations that are not dependent on pore pressure measurements will be used in the analysis.

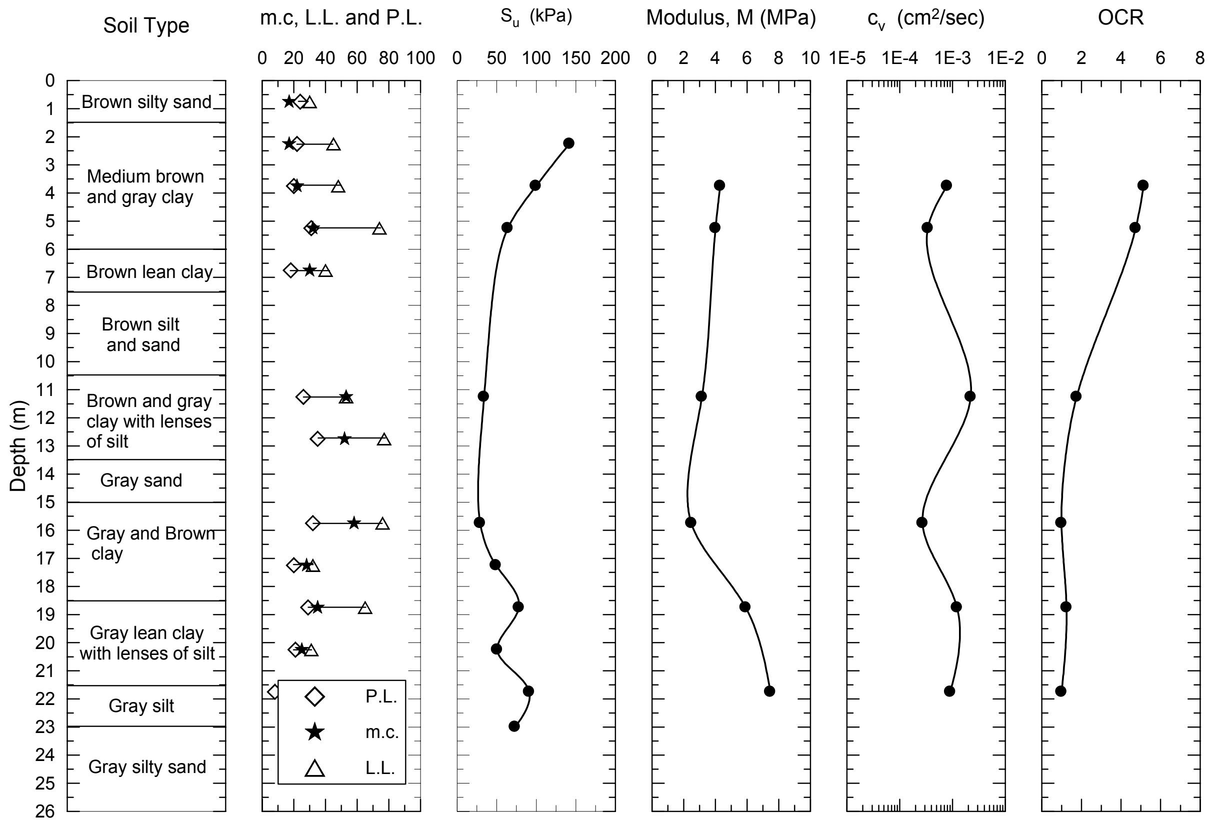
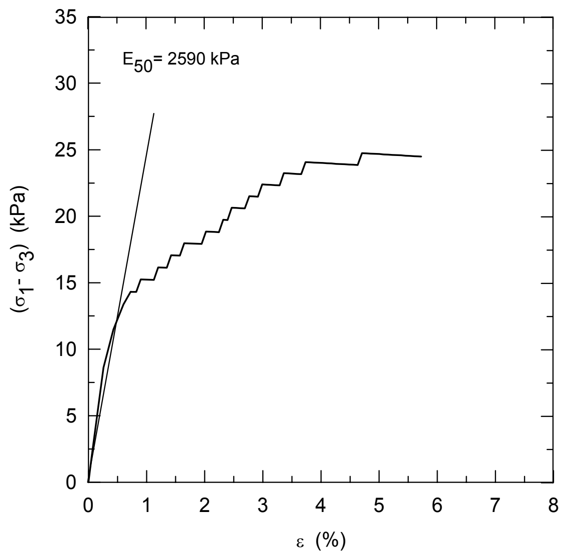
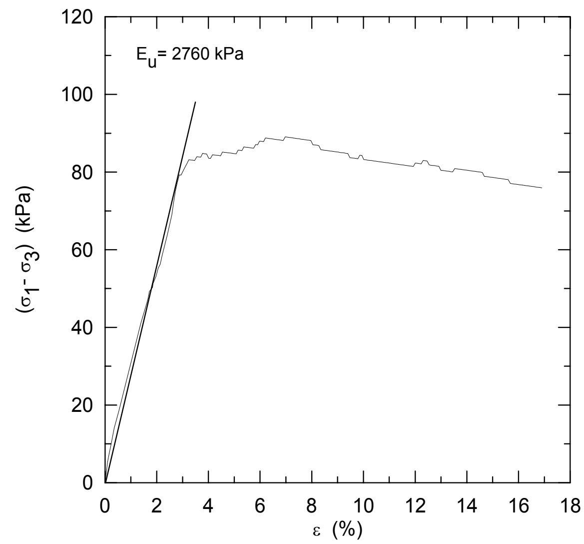


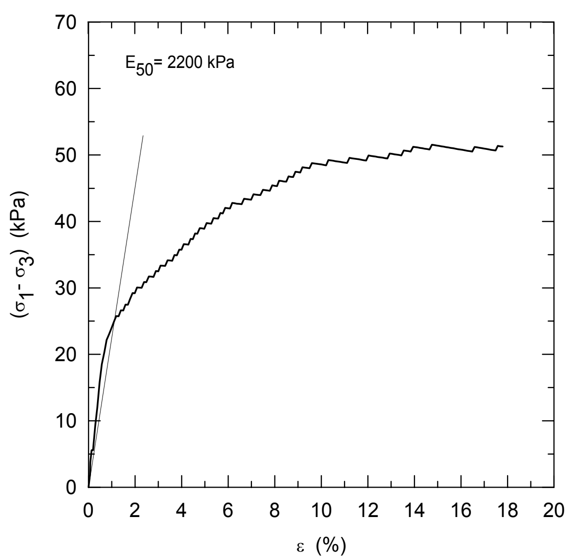
Figure 19
Soil boring profile for Manwell Bridge, Evangeline site



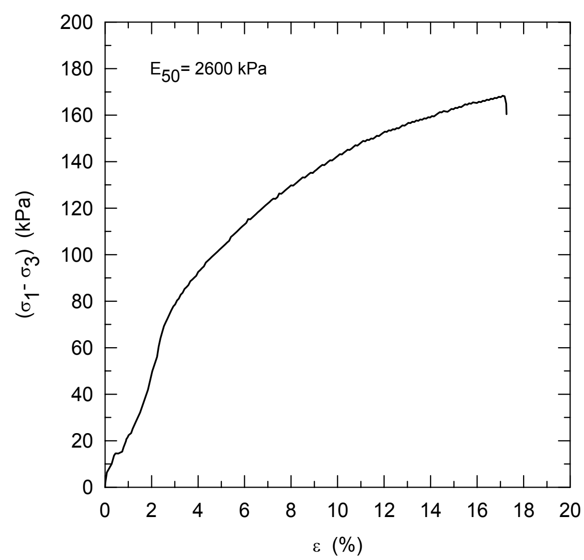
(a) 4-5.5 m



(b) 15-16.5 m



(a) 18-19.5 m



(b) 19.5-21 m

Figure 20
Triaxial tests for Manwell Bridge, Evangeline site

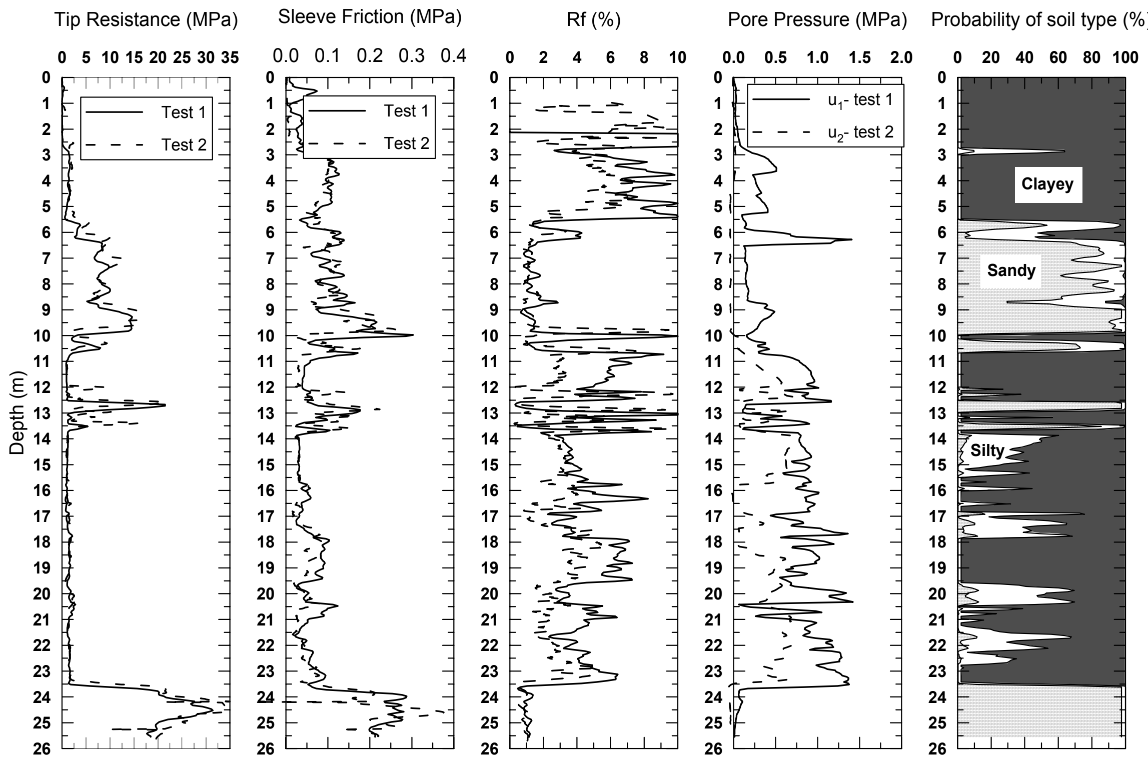


Figure 21
PCPT profiles and soil classification for Evangeline site

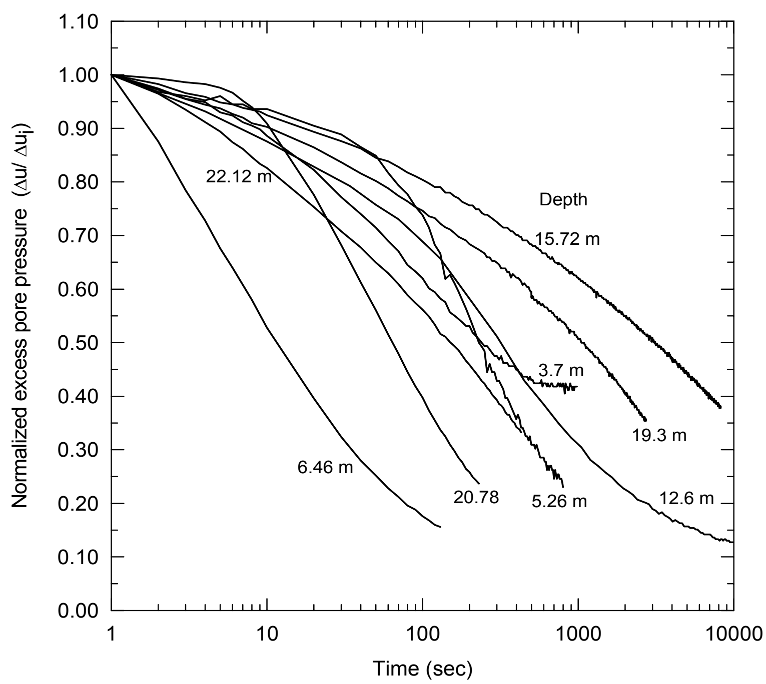


Figure 22
Dissipation tests at Evangeline site

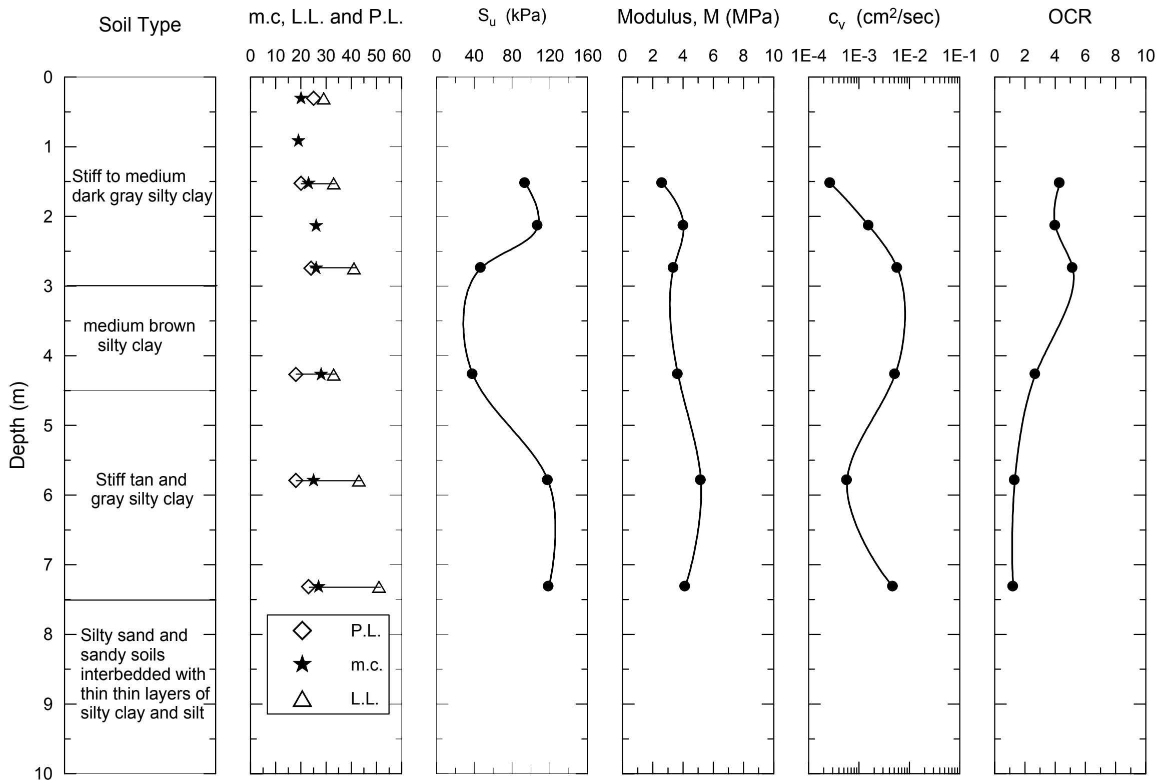
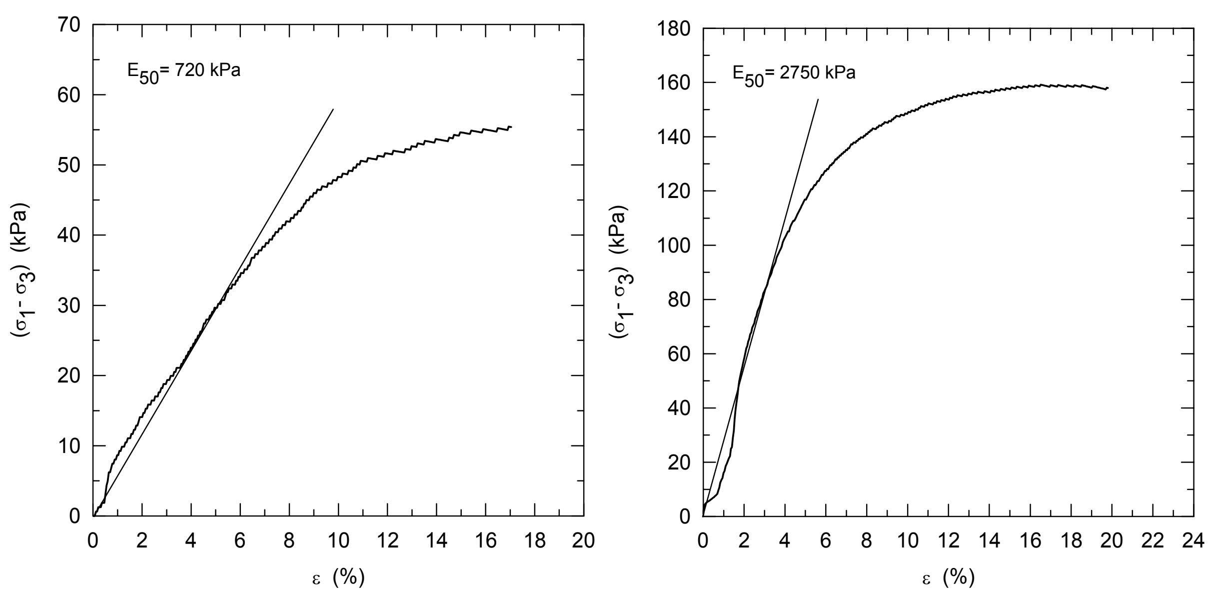


Figure 23
Soil profile for New Iberia site at US 90 and La 88



(a) 2.44 – 3.0 m

(b) 5.5 – 6.1 m

Figure 24
Triaxial tests for New Iberia site at US 90 and LA 88

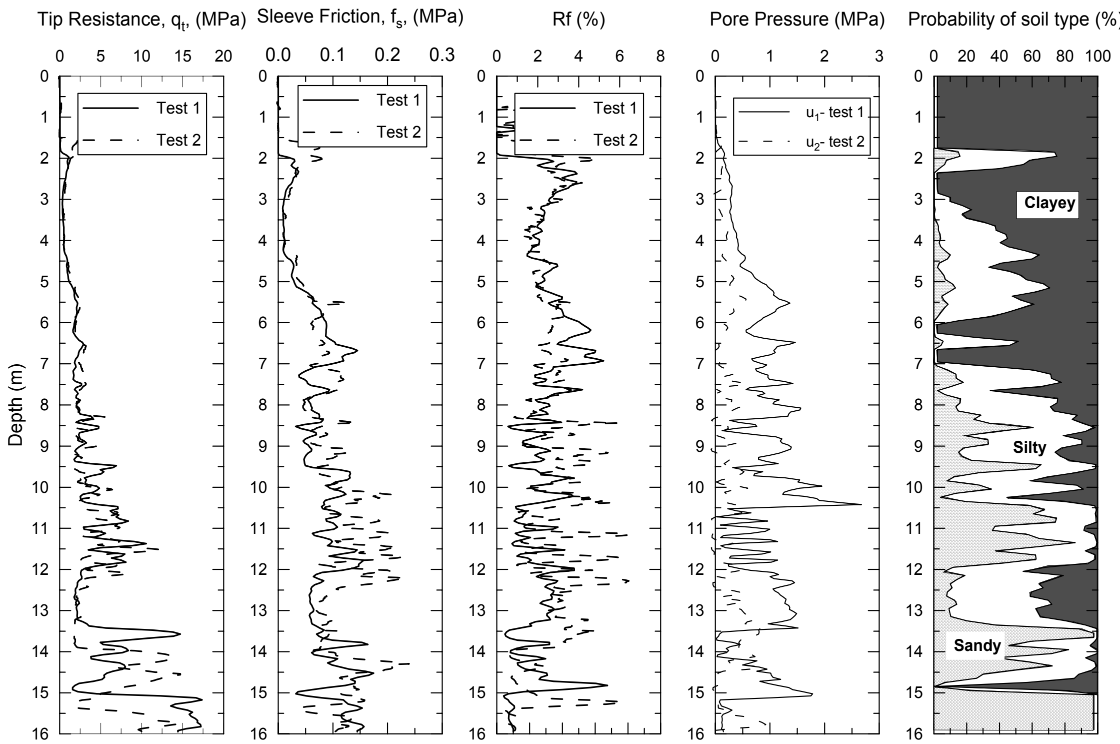


Figure 25
PCPT profiles and soil classification at US 90–LA 88 interchange, New Iberia site

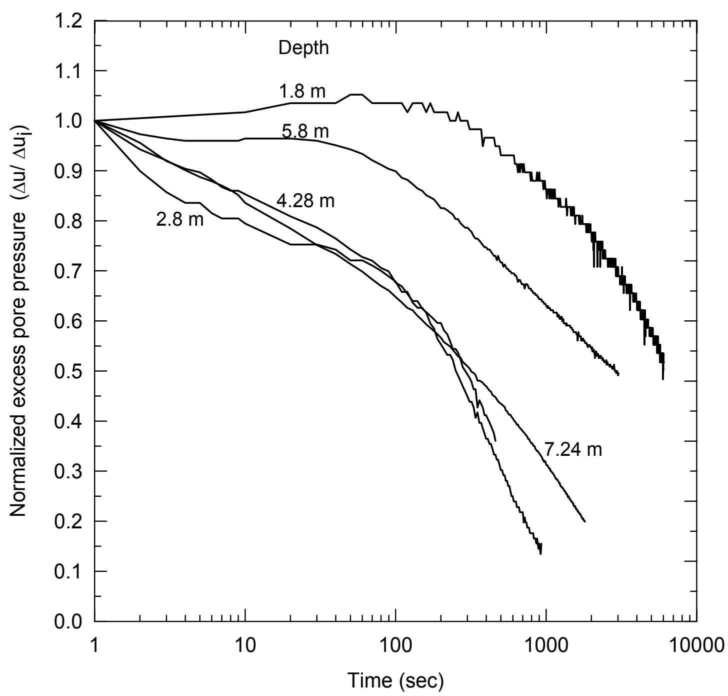


Figure 26
Dissipation tests at US 90 – LA 88 interchange, New Iberia site

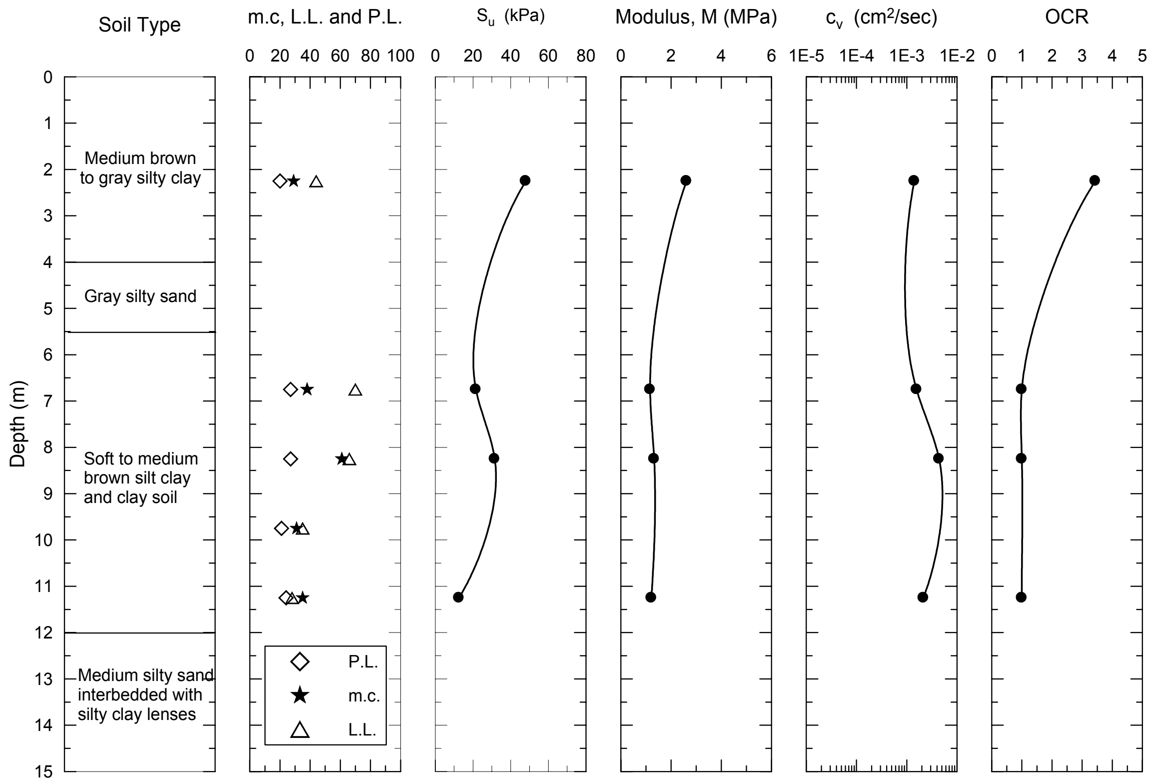
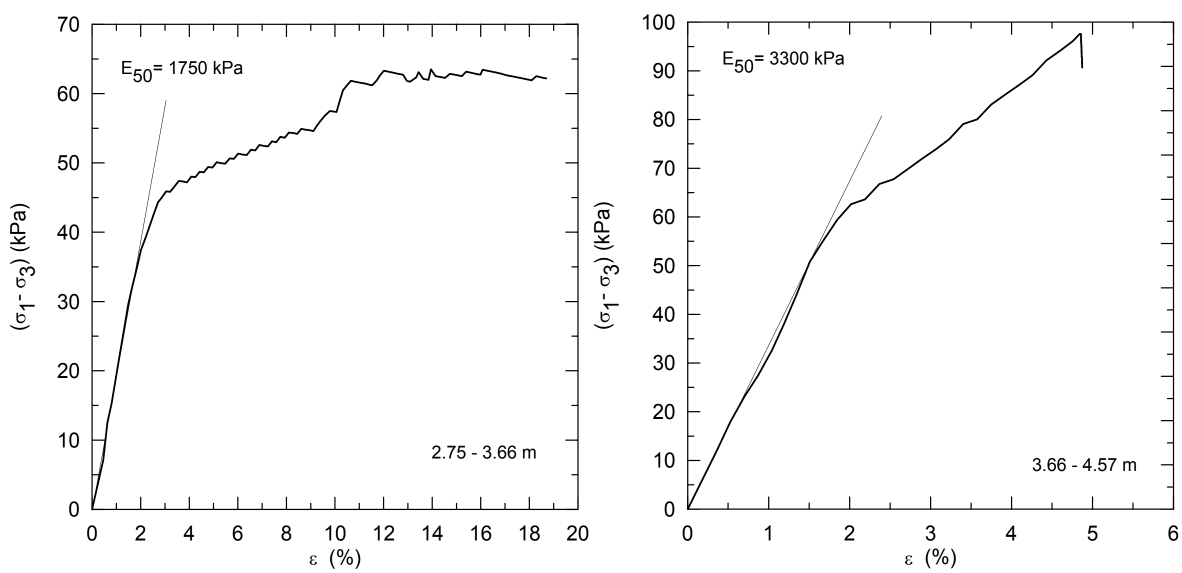


Figure 27
Soil profile for LA Peans canal Bridge, Lafourche site



(a) 2.75 – 3.66 m (b) 3.66 – 4.57 m

Figure 28
Triaxial tests for LA Peans canal bridge, Lafourche site

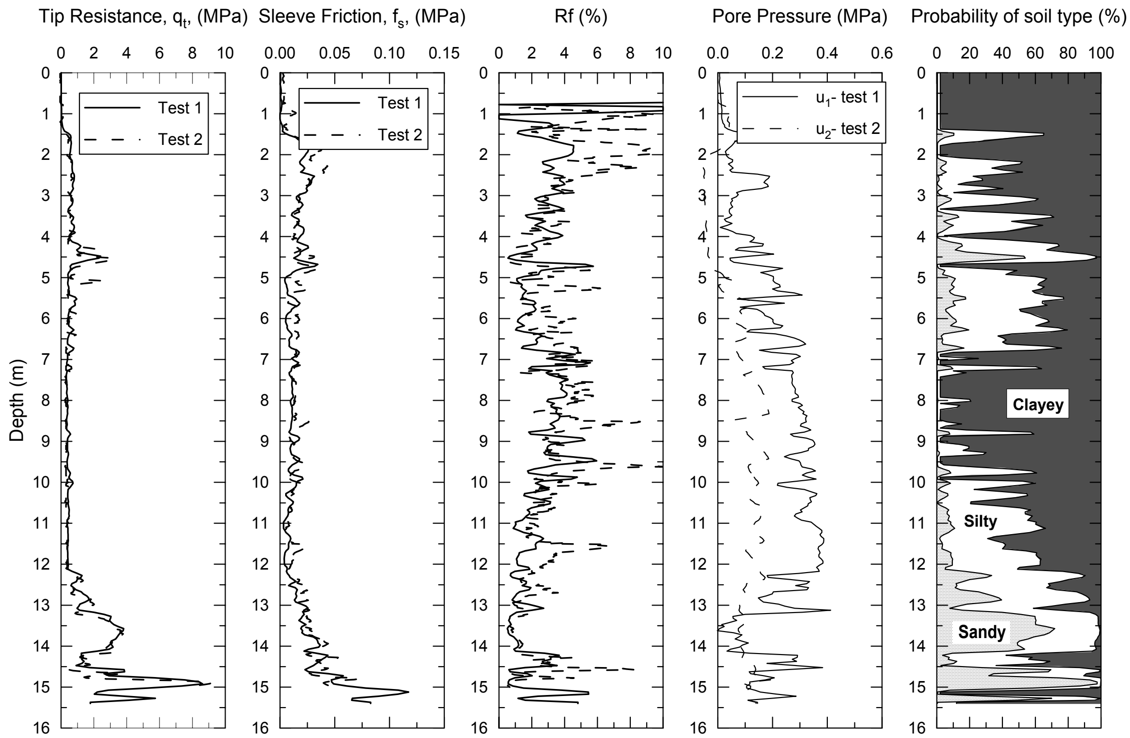


Figure 29
PCPT profiles and soil classification for LA Peans canal bridge, Lafourche site

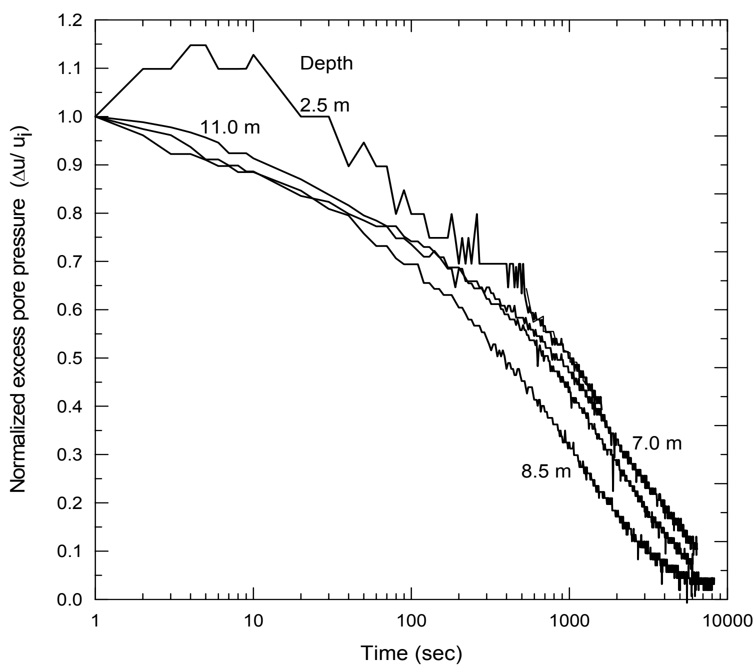


Figure 30
Dissipation tests at LA Peans canal Bridge, Lafourche site

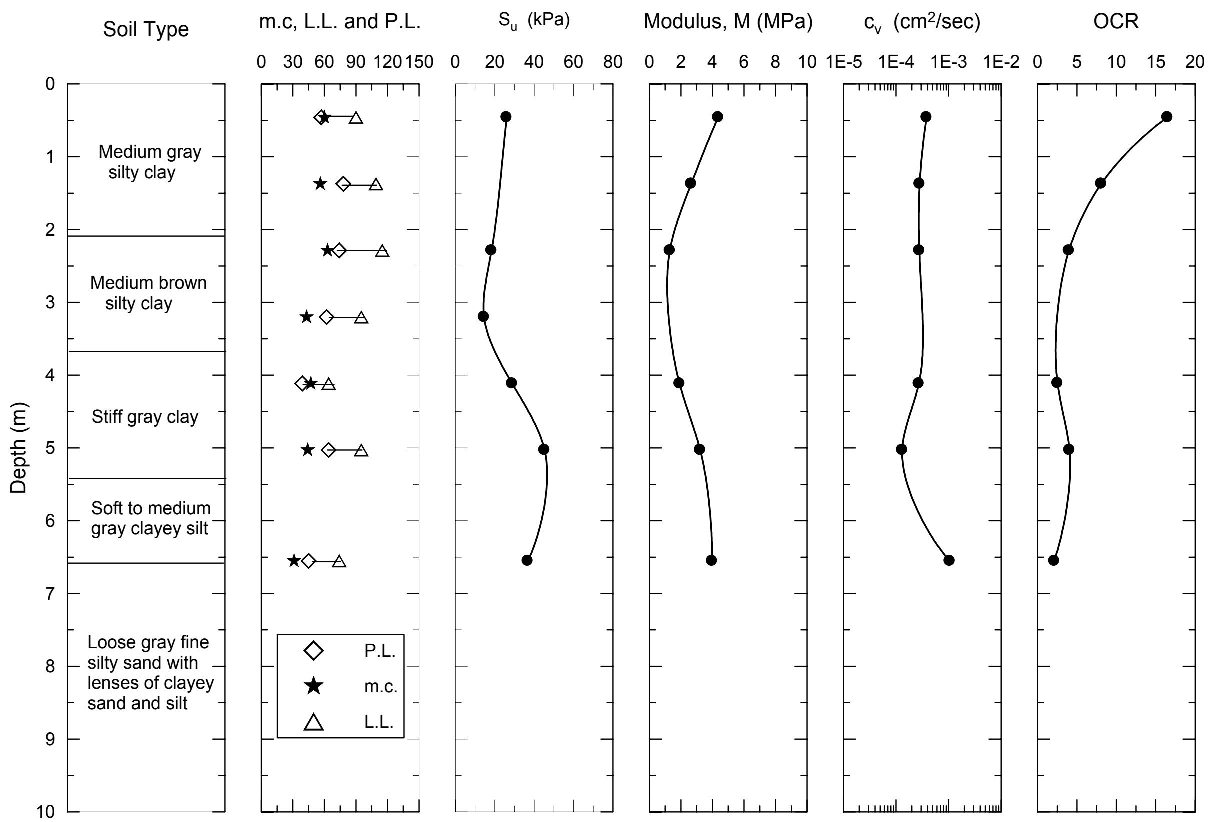
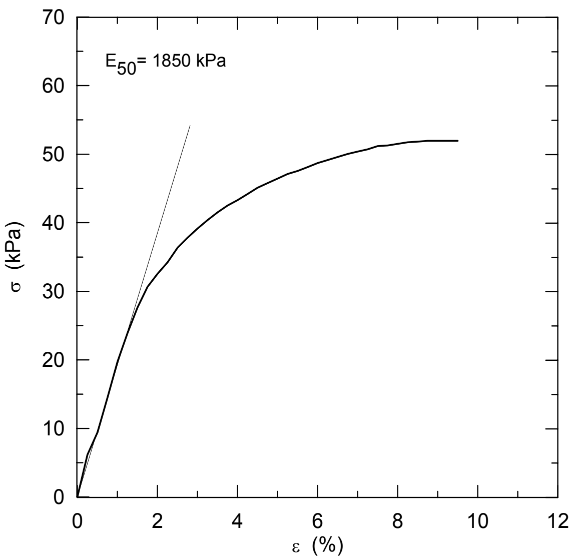
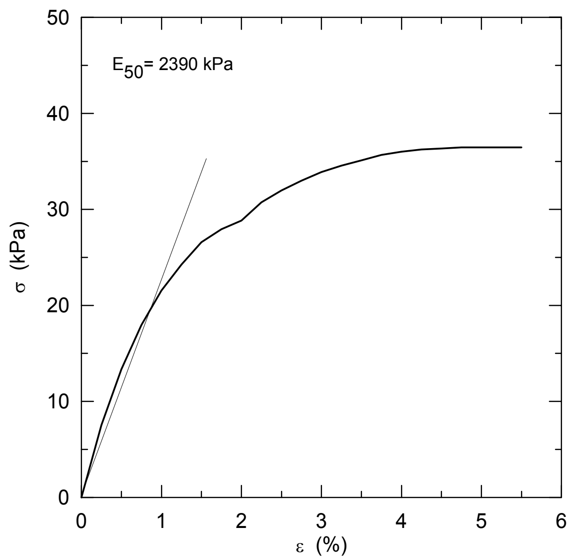


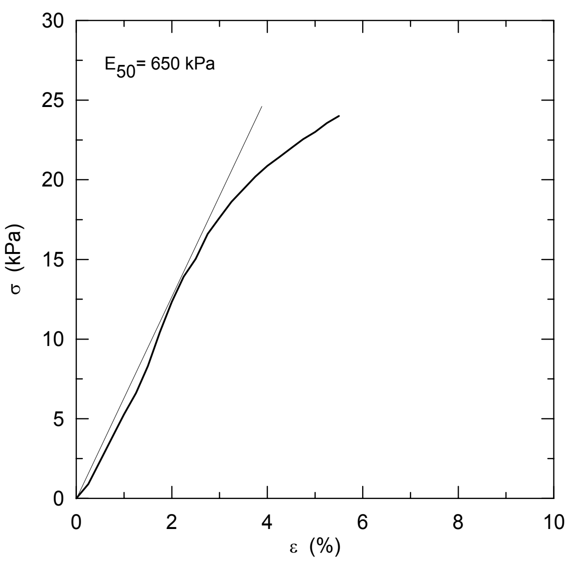
Figure 31
Soil boring profile for PRF site



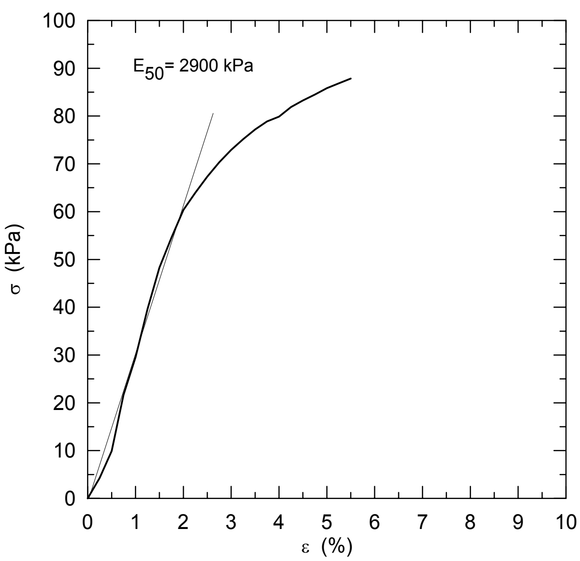
(a) 0-0.9 m



(b) 1.8-2.7 m



(c) 2.7-3.7 m



(d) 4.6-5.5 m

Figure 32
Unconfined compression tests for PRF site

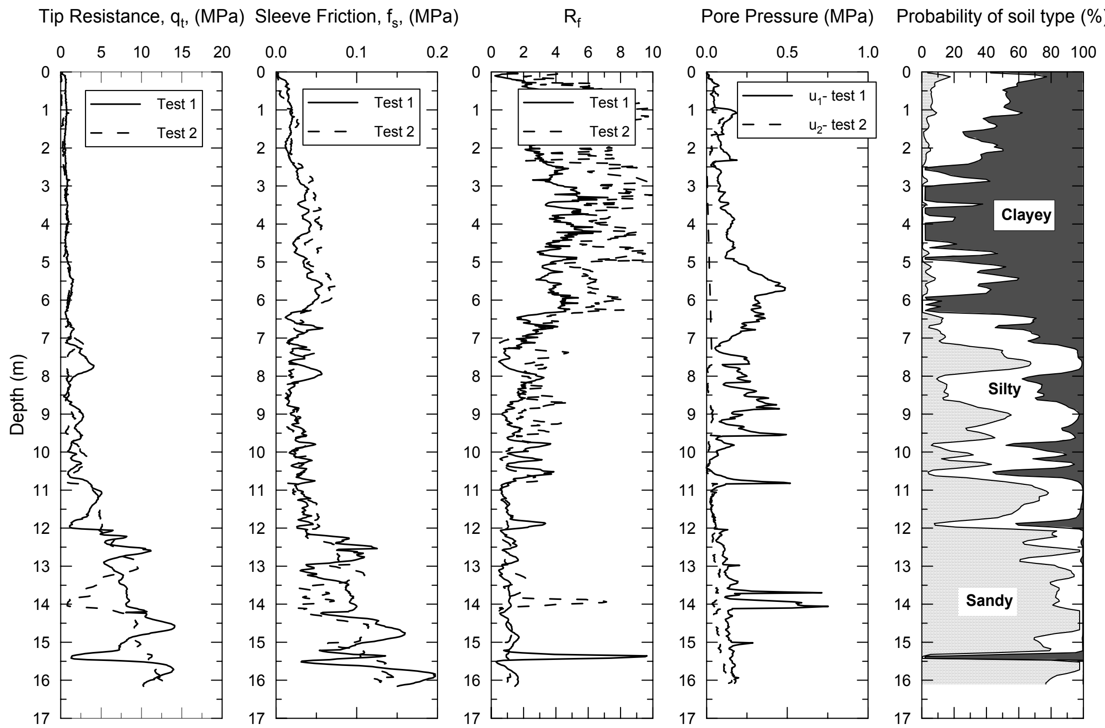


Figure 33
PCPT profiles and soil classification for PRF site

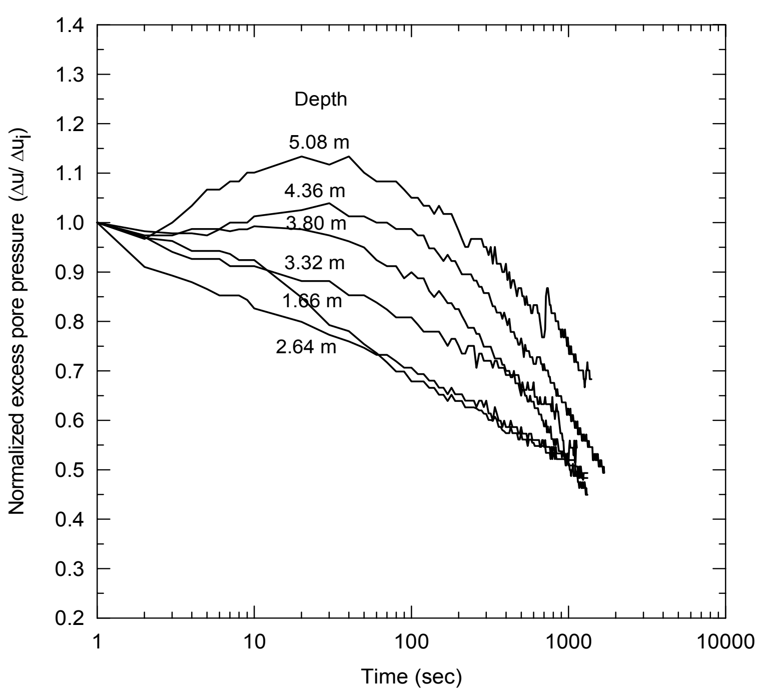


Figure 34
Dissipation tests at PRF site

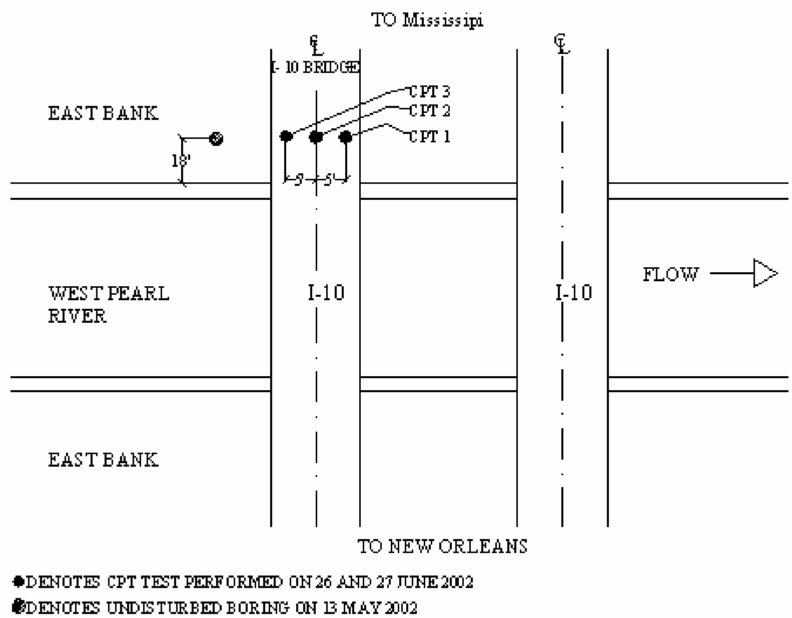


Figure 35
Layout of soil boring and PCPT tests for Pearl River site

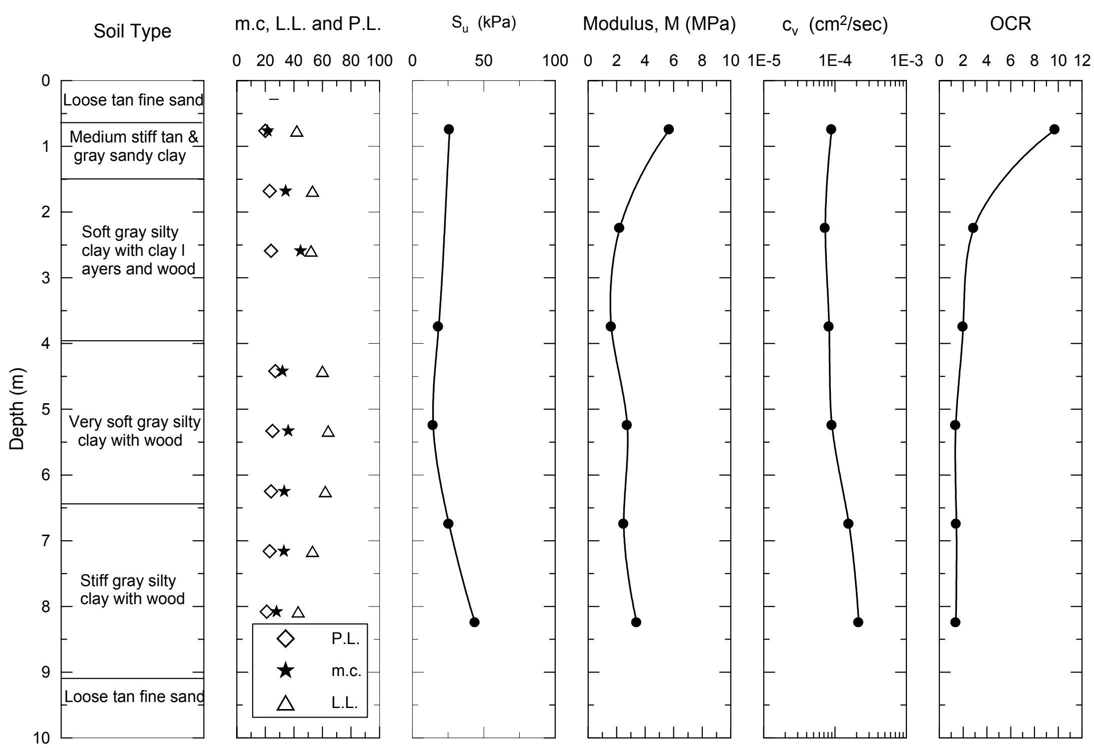
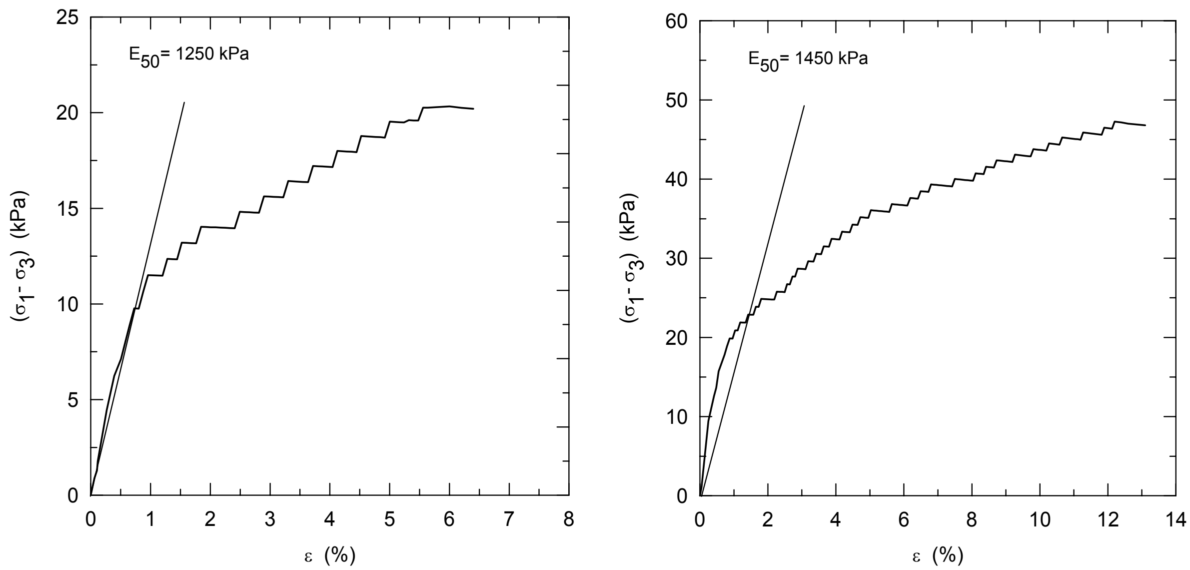


Figure 36
Soil boring profile for Pearl River site



(a) 2.1 – 3.0 m　　(b) 5.8 – 6.7 m

Figure 37
Triaxial tests for Pearl River site

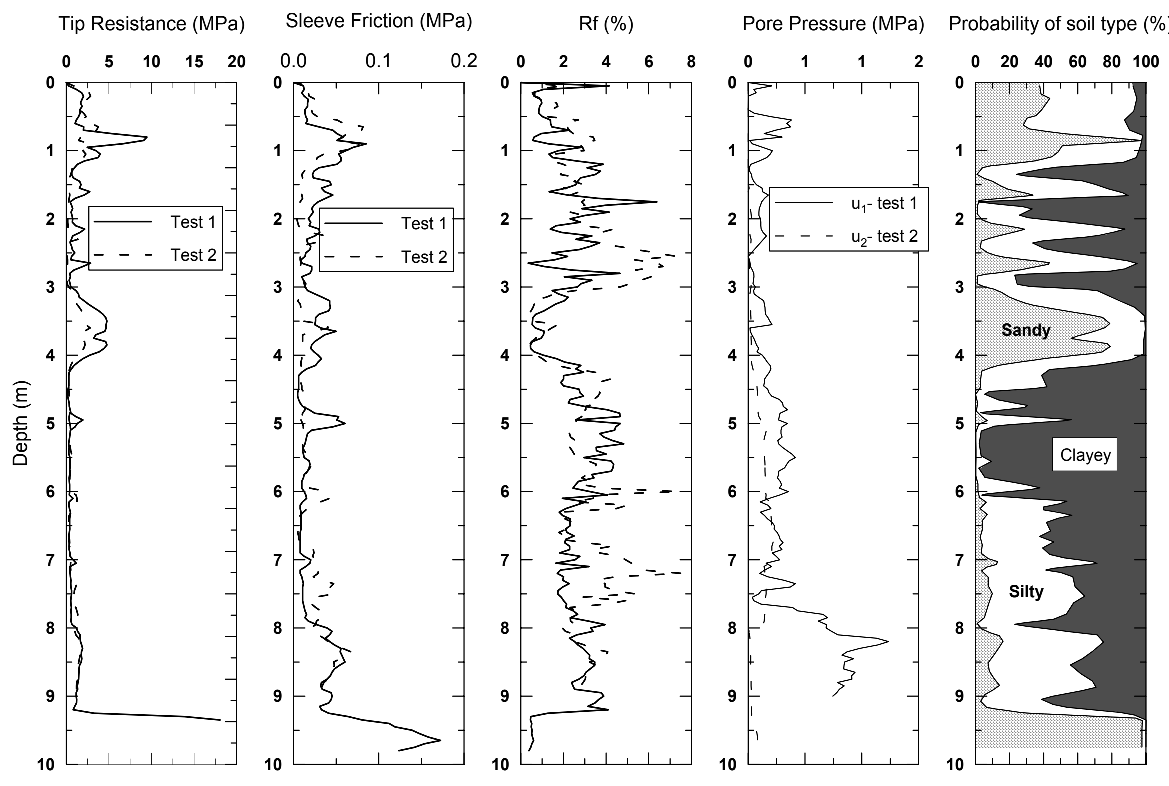


Figure 38
PCPT profiles and soil classification for Pearl River site

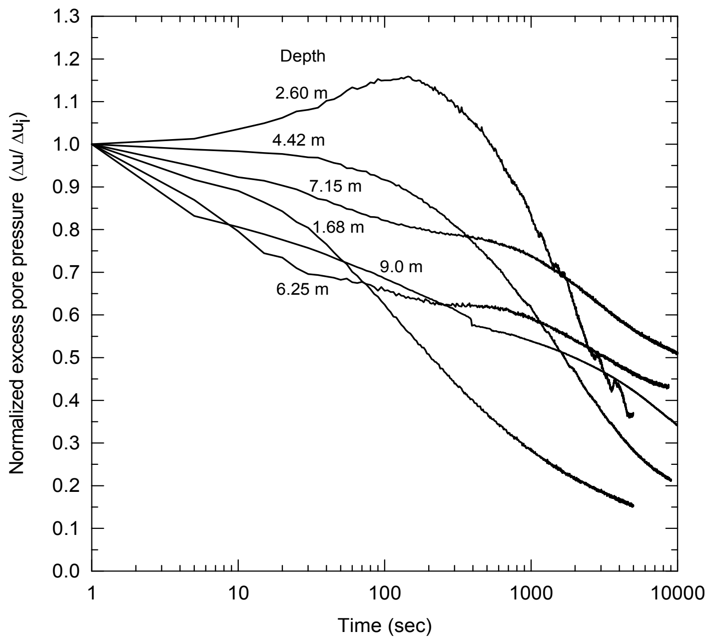


Figure 39
Dissipation tests at Pearl River site

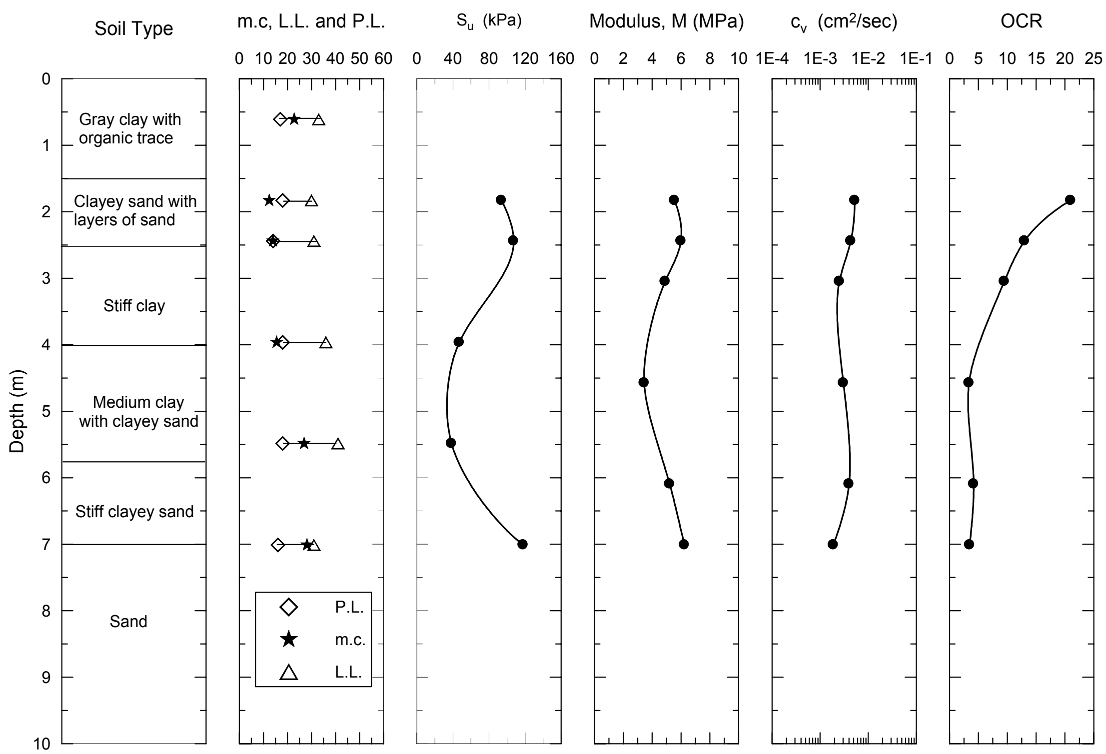
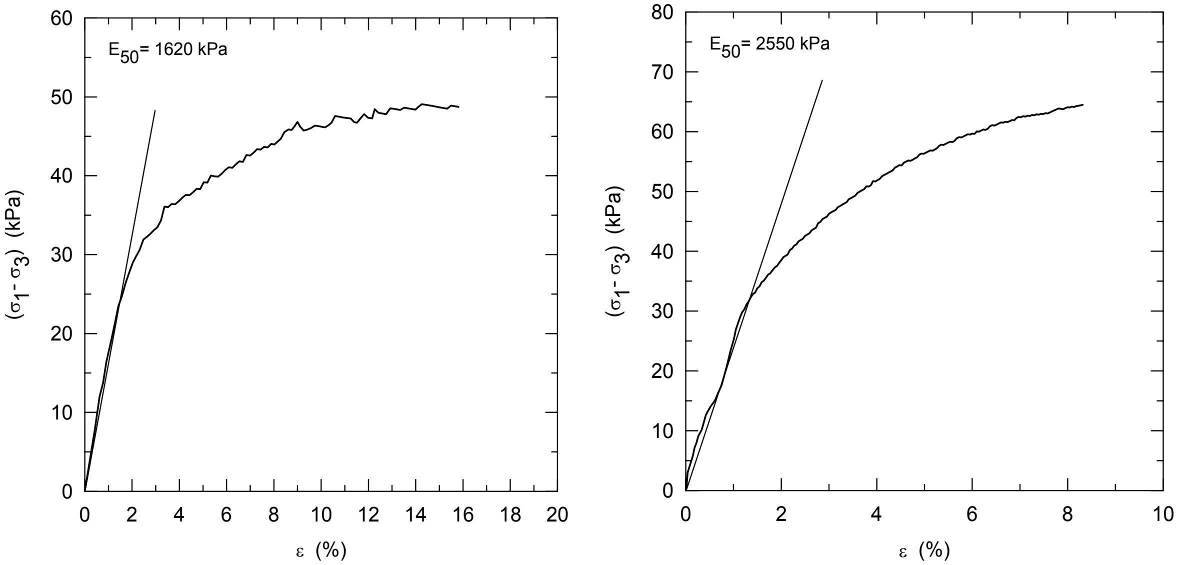


Figure 40
Soil boring profile for East Airport site



(a) 2.74 – 3.66 m (b) 3.66 – 4.57 m

Figure 41
Triaxial tests for East Airport site

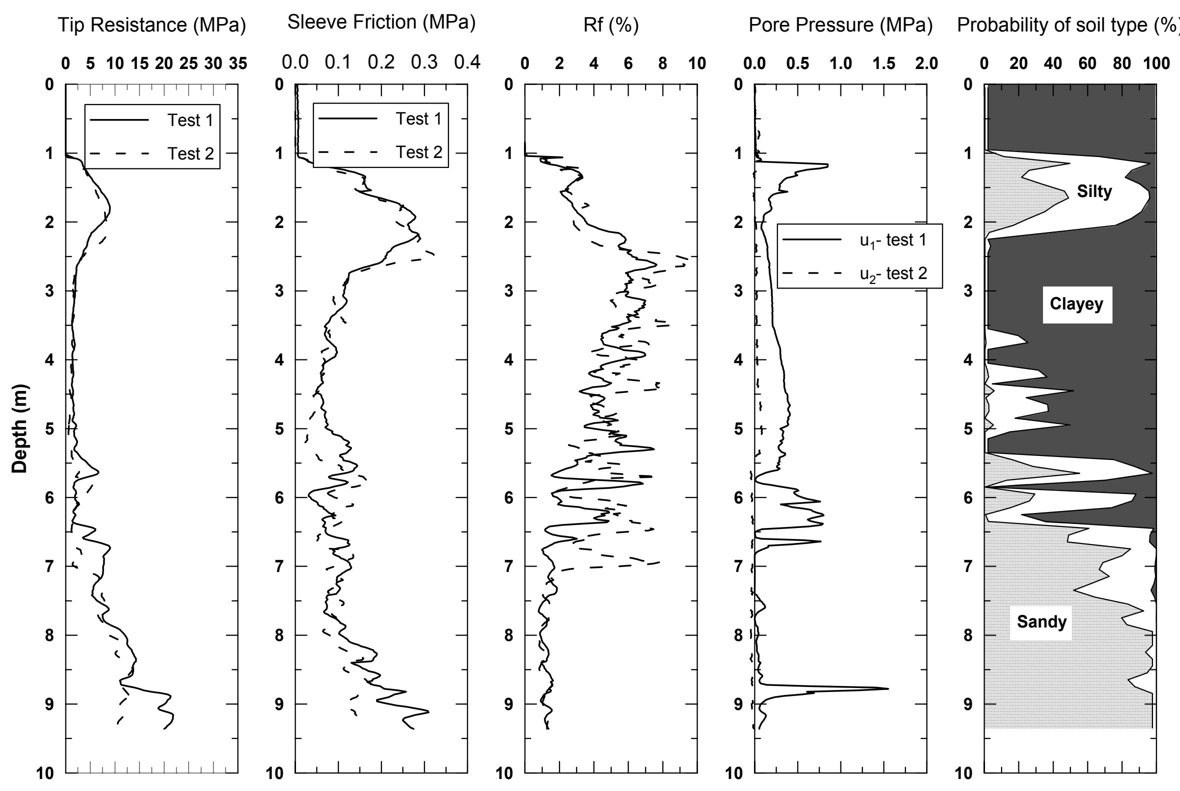


Figure 42
PCPT profiles and soil classification for East Airport site

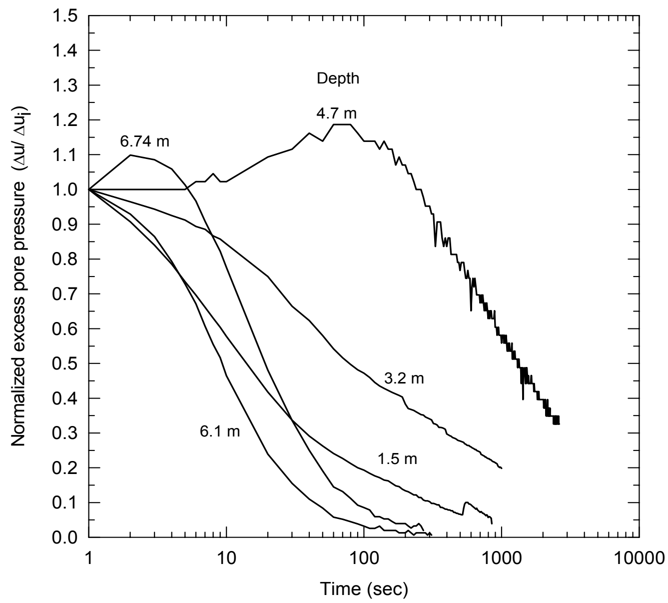


Figure 43
Dissipation tests at East Airport site

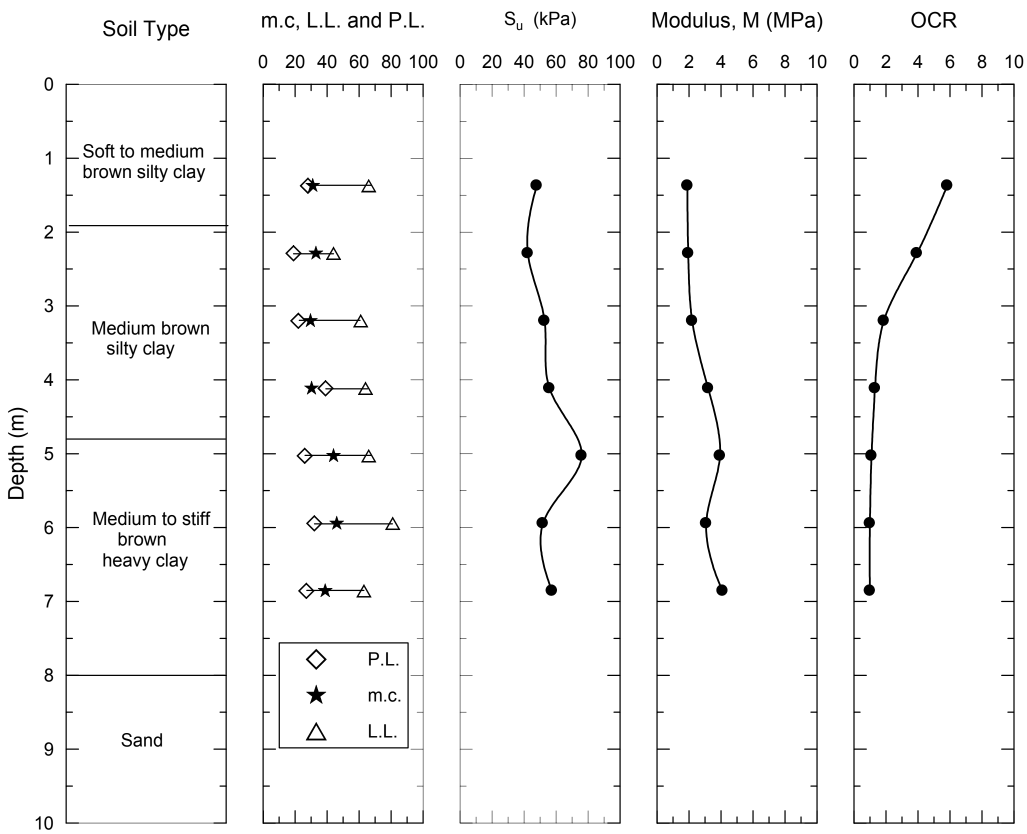


Figure 44
Soil boring profile for Flat River site

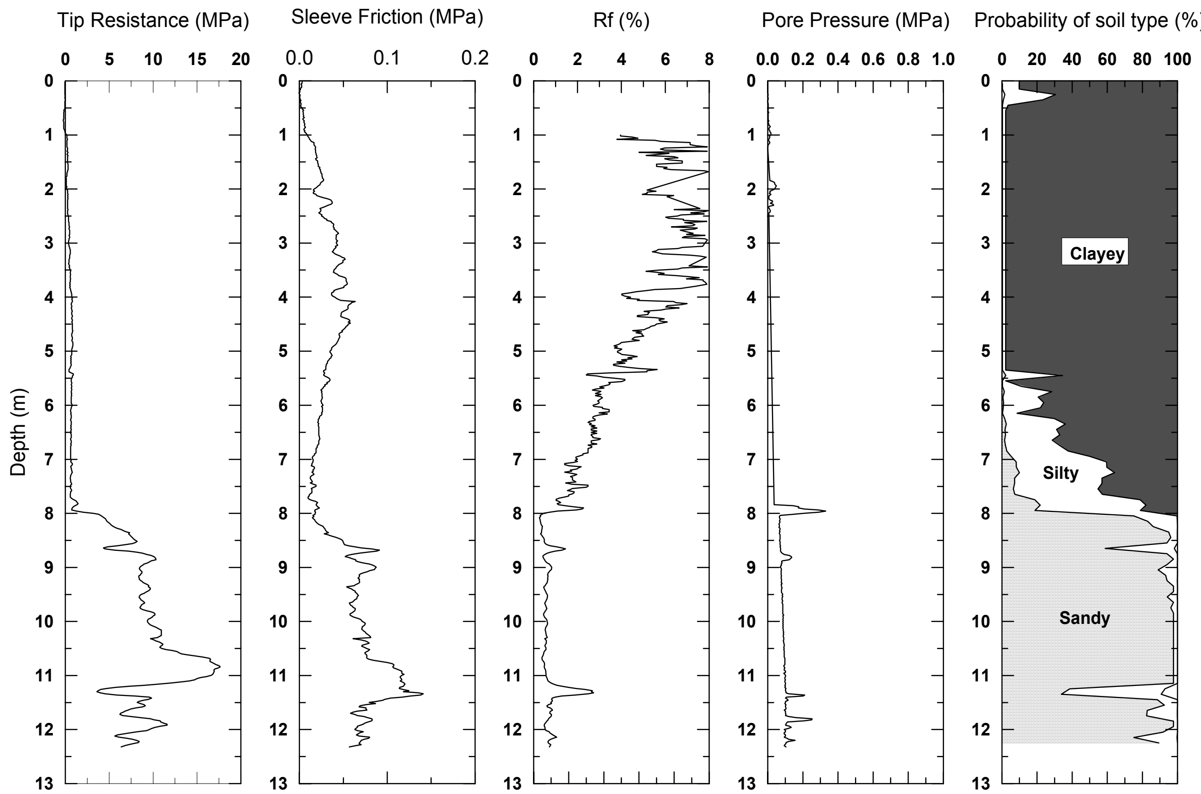


Figure 45
PCPT profiles and soil classification for Flat River site

ANALYSIS OF RESULTS

This section evaluates the capability of the different PCPT interpretation methods for predicting the measured constrained modulus (M), overconsolidation ratio (OCR), and the vertical coefficient of consolidation (c_v). The values predicted from different interpretation methods were first compared with the measured values obtained from laboratory tests to determine the best fit line. The arithmetic mean and standard deviation of predicted to measured values were also calculated and used in the evaluation as described below.

Constrained Modulus (M)

In each site, PCPTs were conducted close to the drilled boreholes. The profiles of PCPT test data were used to calculate the average cone tip resistance (q_t) values that corresponded to the same depths of the extracted shelby tube samples in order to predict the constrained modulus (M) values using the four prediction methods. The average values of total overburden pressure (σ_{vo}) needed for some methods were estimated from the soil borings. The investigated PCPT interpretation methods were Kulhawy and Mayne [9], Jones and Rust [10], Sanglerat [7], and Senneset et al. [8]. The predicted constrained modulus (M_p) from the different PCPT interpretation methods were compared with the measured constrained modulus (M_m) obtained from the oedometer one-dimensional consolidation laboratory tests conducted on samples recovered from boreholes. The results of this comparison are shown in figures 46 through 49 for Kulhawy and Mayne [9], Jones and Rust [10], Sanglerat [7], and Senneset et al. [8], respectively. The figures also present the best fit line (M_{fit}) of the predicted (M_p) to measured constrained modulus (M_m) and the corresponding coefficients of determination (R^2). The arithmetic mean and standard deviation of predicted to measured constrained modulus ratio (M_p/M_m), along with the best fit calculations are summarized in table 6. The results of this analysis and comparison demonstrate that the Kulhawy and Mayne [9] and Senneset et al. [8] methods overpredict the constrained modulus by a factor of 1.5 to 2, while the Jones and Rust [10] method tends to underpredict the constrained modulus. The Sanglerat [7] method, however, shows good prediction of the measured constrained modulus with best fit line of (M_{fit}/M_m) =1.07 and R^2 =0.91. The arithmetic mean and standard deviation for Sanglerat [7] method also show the same result. However, in this method, there is a wide range of α factors to select depending on the q_c value within wide ranges given in a table, which is subjected to the judgment of the user. In this analysis, the author selected the α factor based on the value of q_c within the given interval for the given soil type using interpolation.

Table 6
Evaluation summary of different PCPT methods for predicting *M*

Method	Best fit calculations		Arithmetic calculations M_p/M_m	
	M_{fit}/M_m	R^2	Mean	Standard Deviation
Kulhawy and Mayne [9]	2.16	0.87	2.05	0.81
Jones and Rust [10]	0.80	0.90	0.78	0.26
Sanglerat [7]	1.07	0.91	1.23	0.43
Senneset et al. [8]	1.57	0.88	1.51	0.55

To examine the possibility for better correlations to estimate the constrained modulus from PCPT data, the corrected cone tip resistance (q_t) and the net cone tip resistance ($q_t - \sigma_{vo}$) were plotted against the measured constrained modulus as shown in figures 50 and 51. A linear correlation was obtained between M and q_t as follows:

$$M = 3.15\, q_t \quad , \quad \text{with } R^2 = 0.91 \tag{37}$$

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

$$M = 3.58\, (q_t - \sigma_{vo}) \quad , \quad \text{with } R^2 = 0.88 \tag{38}$$

The arithmetic mean and standard deviation of (M_p/M_m) are 0.98 and 0.33 for the first correlation ($M = 3.15\, q_t$), and 1.01 and 0.52 for the second correlation ($M = 3.58\, (q_t - \sigma_{vo})$).

Figures 52 through 58 compare the profiles with depth of laboratory measured constrained modulus and the predicted constrained modulus using the different interpretation methods including the proposed correlations for the different investigated sites. The comparison figures clearly show that the proposed two relations predicted the measured constrained modulus (M) of the different sites better than the other PCPT interpretation methods, as expected. However, the proposed relations should be verified using other sites.

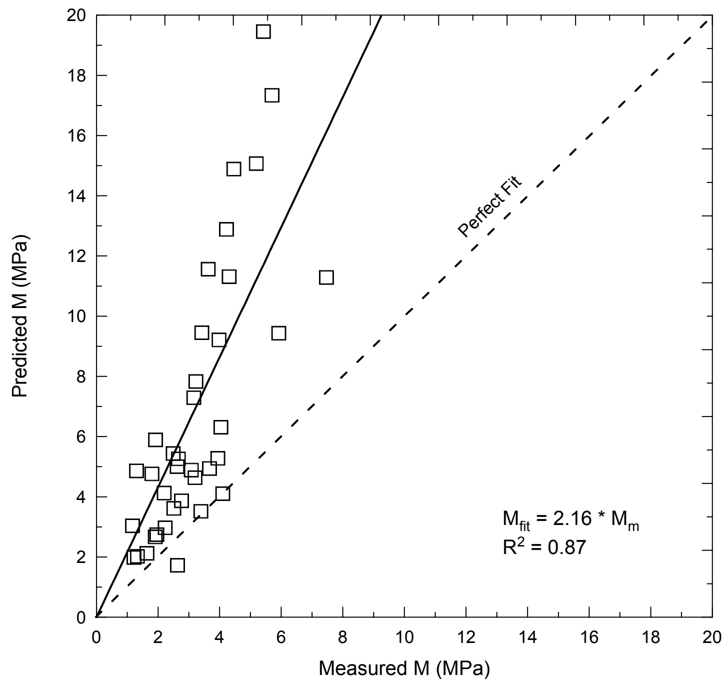


Figure 46
Measured versus predicted *M* using Kulhawy and Mayne [9]

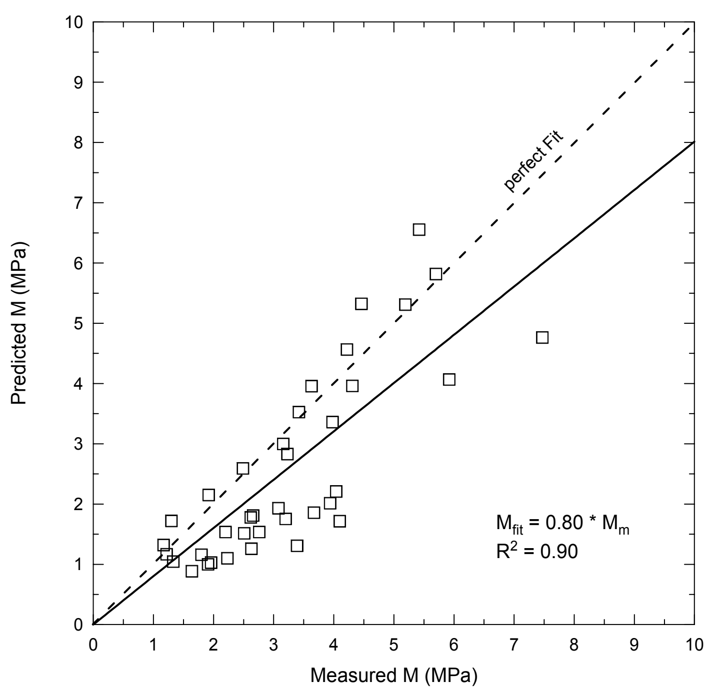


Figure 47
Measured versus predicted *M* using Jones and Rust [10]

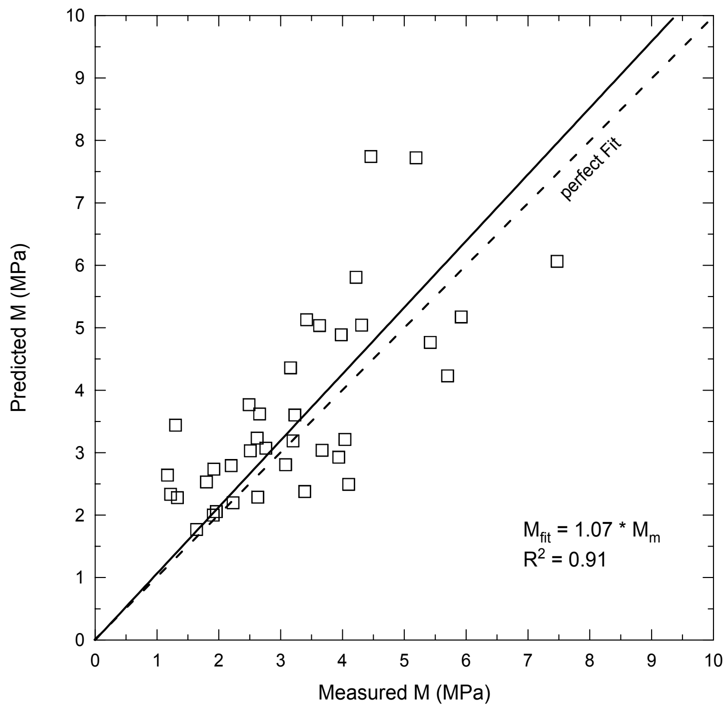


Figure 48
Measured versus predicted *M* using Sanglerat [7]

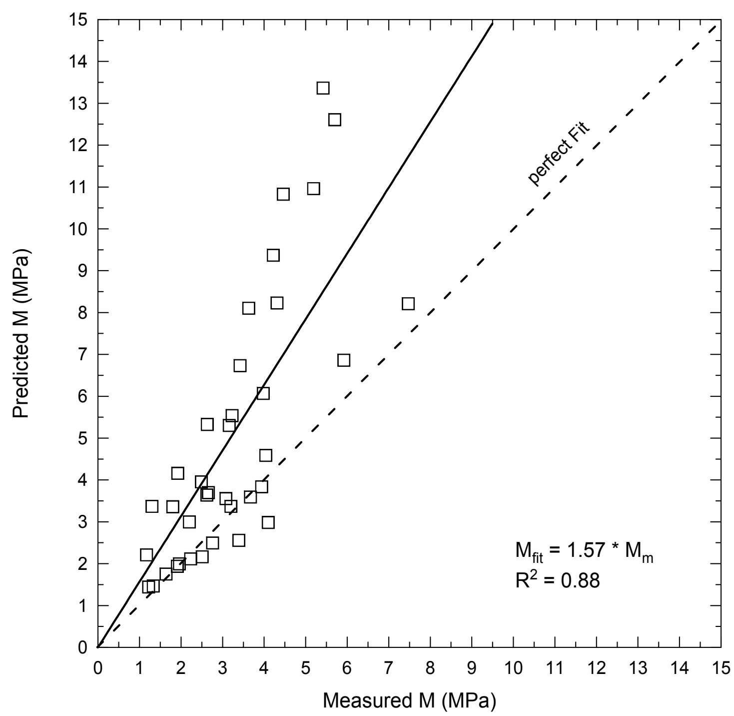


Figure 49
Measured versus predicted *M* using Senneset et al. [8]

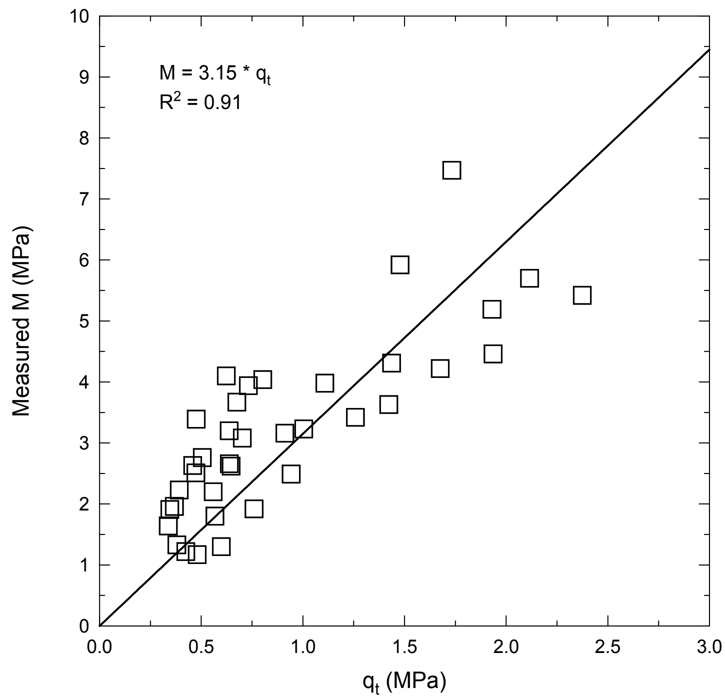


Figure 50
q_t versus measured M

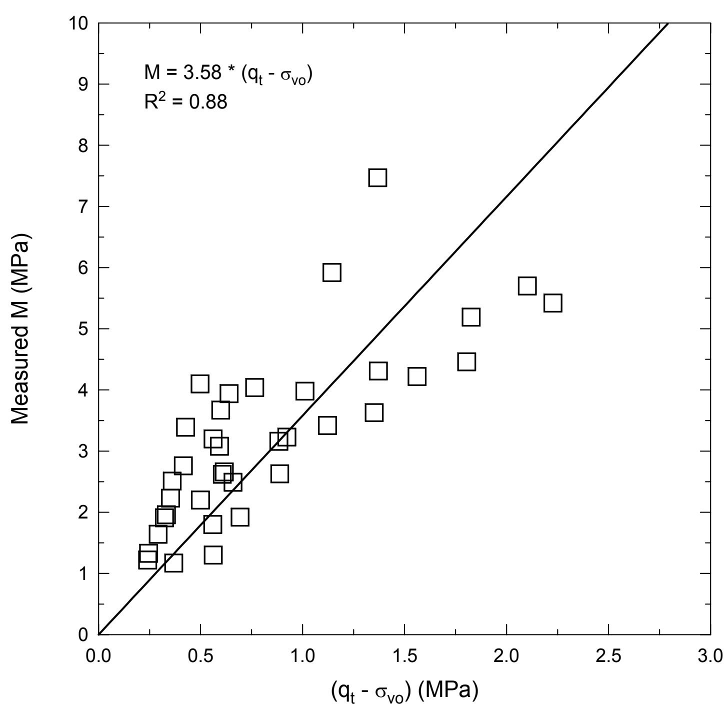


Figure 51
$(q_t - \sigma_{vo})$ versus measured M

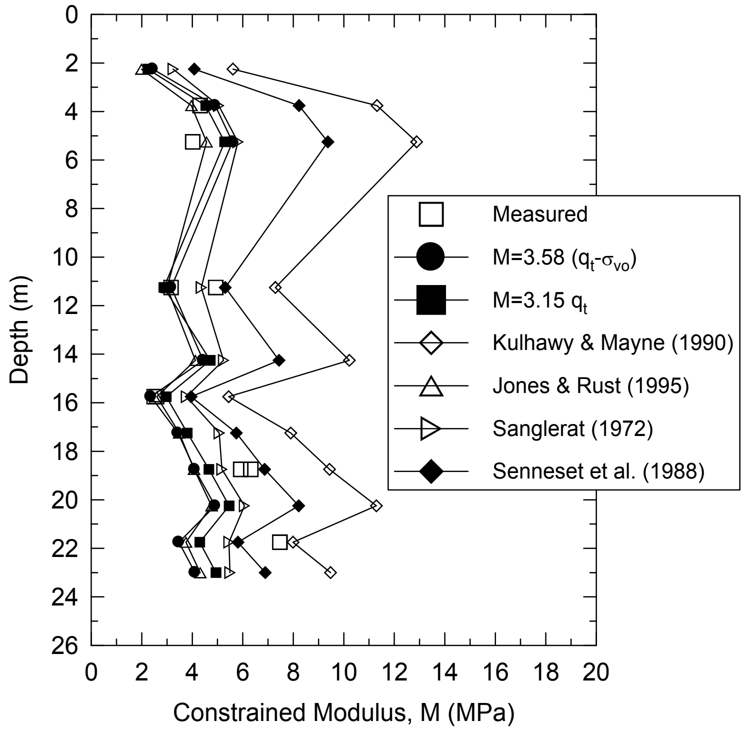


Figure 52
Measured versus predicted *M* for Manwell Bridge - Evangeline site

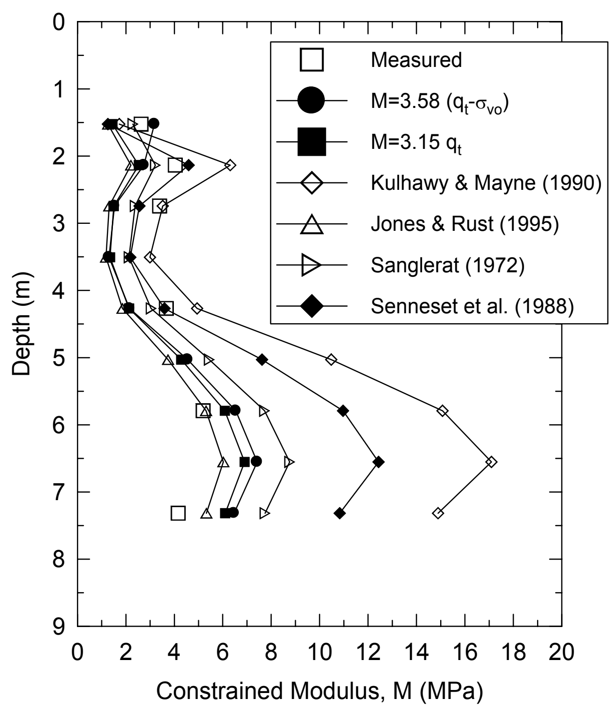


Figure 53
Measured versus predicted *M* for US 90–LA 88 - New Iberia site

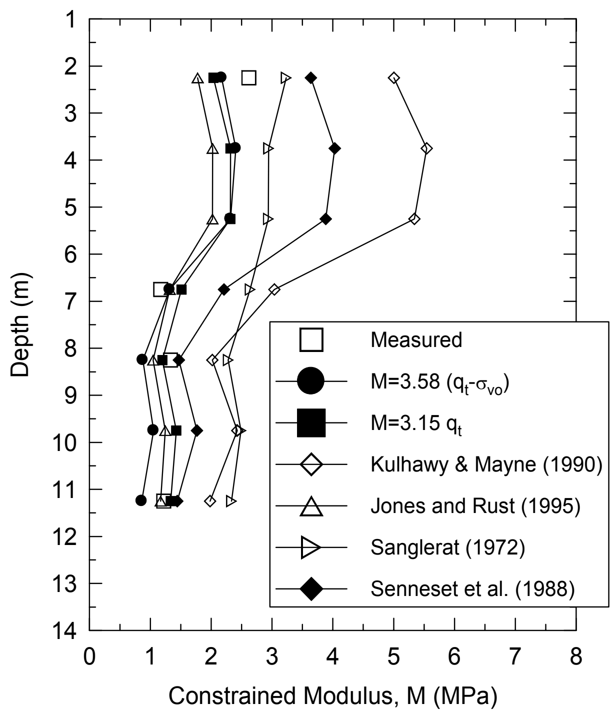


Figure 54
Measured versus predicted *M* for LA Peans canal bridge - Lafourche site

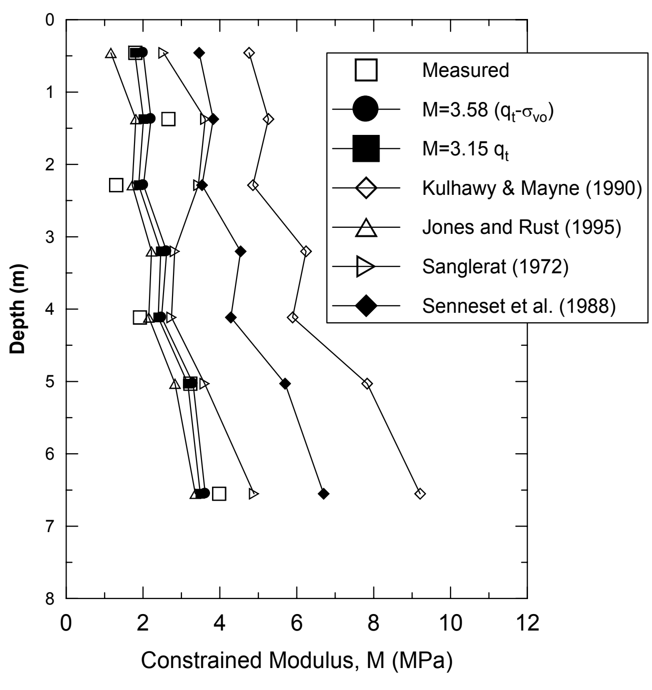


Figure 55
Measured versus predicted *M* for PRF site

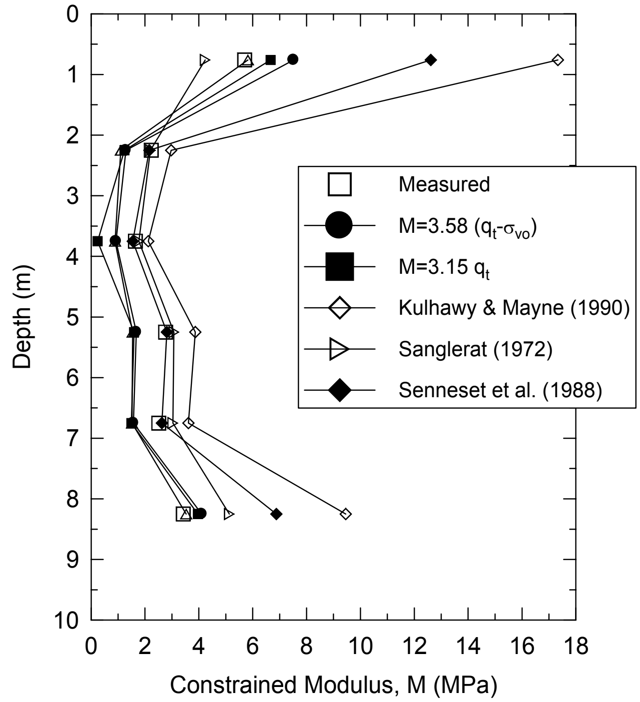


Figure 56
Measured versus predicted *M* for Pearl River Bridge site

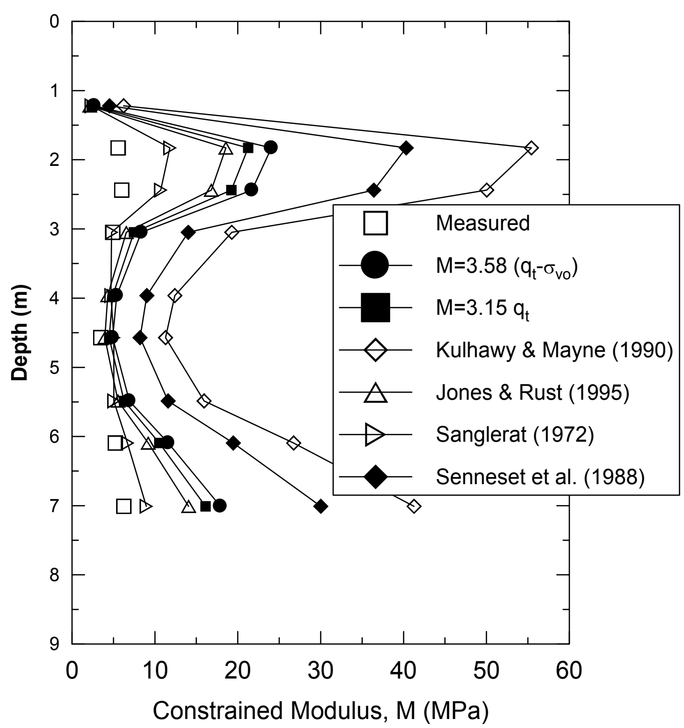


Figure 57
Measured versus predicted *M* for East Airport – Baton Rouge site

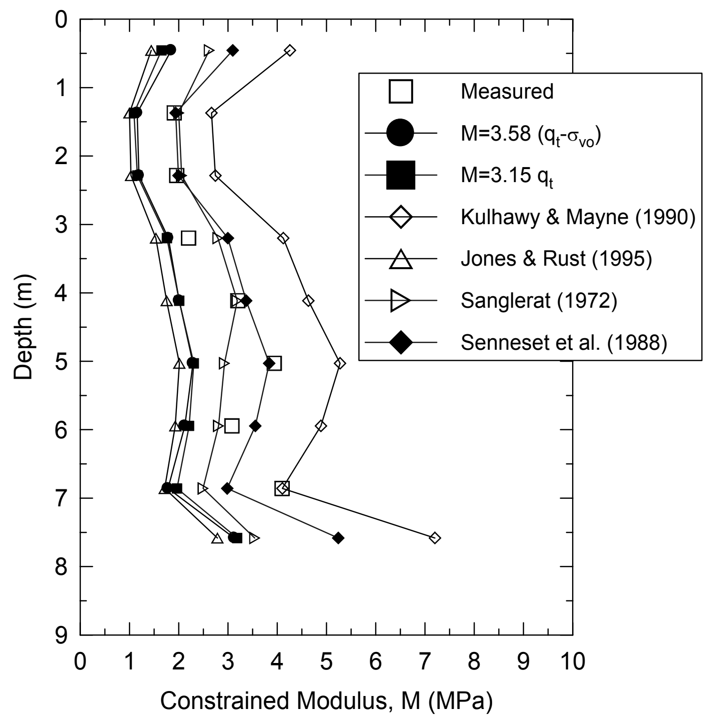


Figure 58
Measured versus predicted *M* for Flat River – Bossier site

Overconsolidation Ratio

Four different PCPT methods were selected to evaluated their capability to reliably predict the overconsolidation ratio (*OCR*) utilizing the PCPT test data. These methods were the Schmertmann [16] method using undrained shear strength, the Kulhawy and Mayne [9] method using the normalized net tip resistance, and the Chen and Mayne [18] methods using pore pressure measurements either at the cone tip (u_1) or at the cone base (u_2). In order to evaluate these methods, the profiles of PCPT test data obtained from the investigated sites were used to calculate the average q_t, u_1, and u_2 values that corresponded to the depths of the extracted samples. The profiles were also used to predict the *OCR* values using the different PCPT interpretation methods. The total and effective overburden pressure (σ_{vo}, σ'_{vo}) were calculated from the soil borings. The predicted *OCR* was then compared with the measured *OCR* obtained from the oedometer laboratory tests conducted on samples recovered from boreholes. Figures 59 through 62 present the comparison between the measured to predicted *OCR* for Schmertmann [16], Kulhawy and Mayne [9], Chen and Mayne [18] using u_1, and Chen and Mayne [18] using u_2, respectively. The best fit line of the predicted to the measured overconsolidation ratio (OCR_{fit}/OCR_m) and the corresponding coefficients of determination (R^2) for the different methods were calculated and presented in the figures and in table 7. The arithmetic mean and standard deviation of predicted to measured overconsolidation ratio (OCR_p/OCR_m) were also calculated and summarized in table 7.

The results of the comparison and arithmetic analysis clearly indicate that all the investigated PCPT prediction methods overestimate the *OCR* by a factor ranging from 2.0 times for Kulhawy and Mayne [9] to 4.27 times for Chen and Mayne [18] using u_1. Therefore, the possibility of having better correlations between PCPT data and measured *OCR* were examined. For this purpose, the measured *OCR* were compared with $(q_t - u_1)/\sigma'_{vo}$ and $(q_t - \sigma_{vo})/\sigma'_{vo}$ ratios as shown in figures 63 and 64, respectively. A linear correlation was obtained between *OCR* and $(q_t - u_1) / \sigma'_{vo}$ as follows:

$$OCR = 0.161\ (q_t - u_1) / \sigma'_{vo} \quad , \quad \text{with } R^2 = 0.91 \qquad (39)$$

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

$$OCR = 0.152\ (q_t - \sigma_{vo}) / \sigma'_{vo} \quad , \quad \text{with } R^2 = 0.90 \qquad (40)$$

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. It should be noted here that 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 [29].

The profile of laboratory-measured *OCR* using oedometer tests for the different investigated sites was compared with the predicted *OCR* using the different PCPT interpretation methods as well as the two proposed correlations as shown in figures 65 through 71. As expected, for the different sites, the two proposed relations predicted of the measured *OCR* better than the other methods. The validity of these relations needs to be verified for other sites.

Table 7
Evaluation summary of different PCPT methods for predicting *OCR*

Method	Best fit calculations		Arithmetic calculations OCR_p/OCR_m	
	OCR_{fit}/OCR_m	R^2	Mean	Standard Deviation
Schmertmann [16]	3.25	0.88	2.43	1.99
Kulhawy and Mayne [9]	2.00	0.89	2.53	1.80
Chen and Mayne [18] – u_1	4.27	0.92	4.04	2.20
Chen and Mayne [18] – u_2	2.79	0.91	3.02	1.88

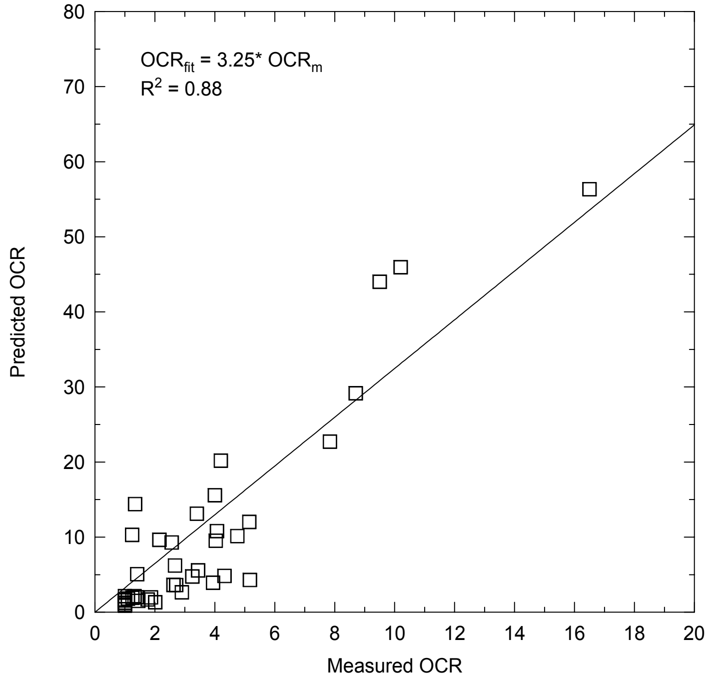


Figure 59
Measured versus predicted *OCR* using Schmertmann [16]

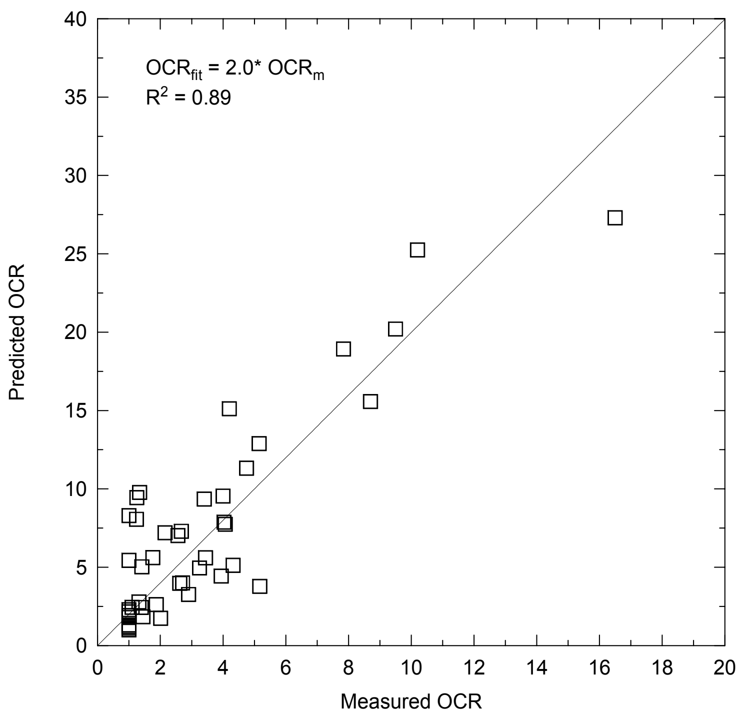


Figure 60
Measured versus predicted *OCR* using Kulhawy and Mayne [19]

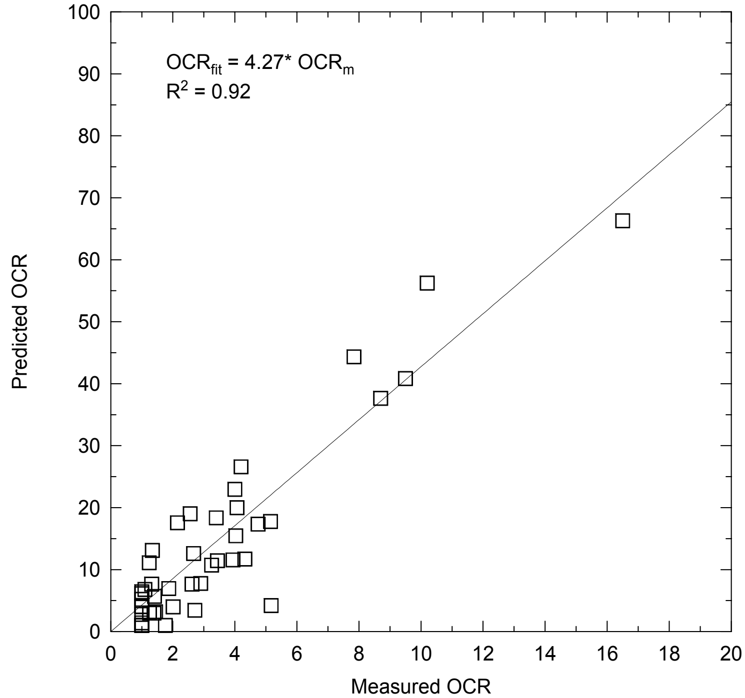


Figure 61
Measured versus predicted *OCR* using Chen and Mayne [18] - u_1

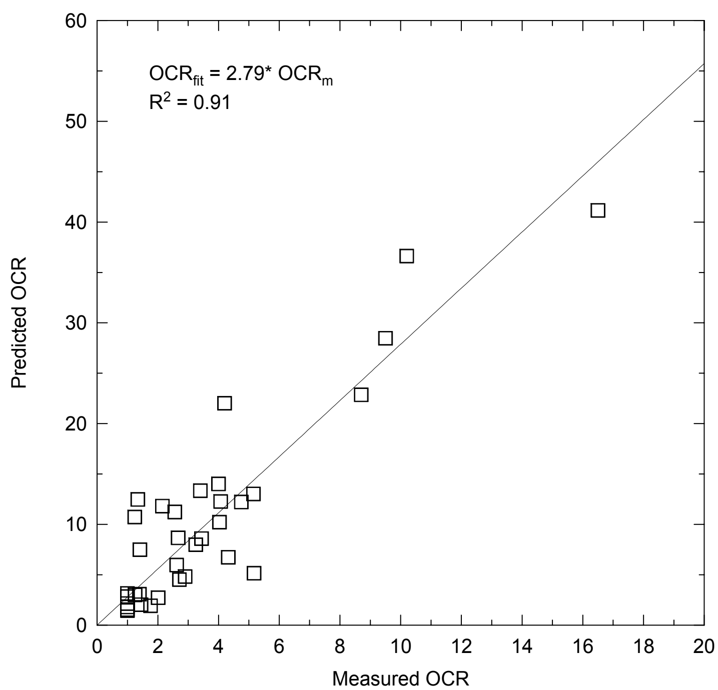


Figure 62
Measured versus predicted OCR using Chen and Mayne [18] - u_2

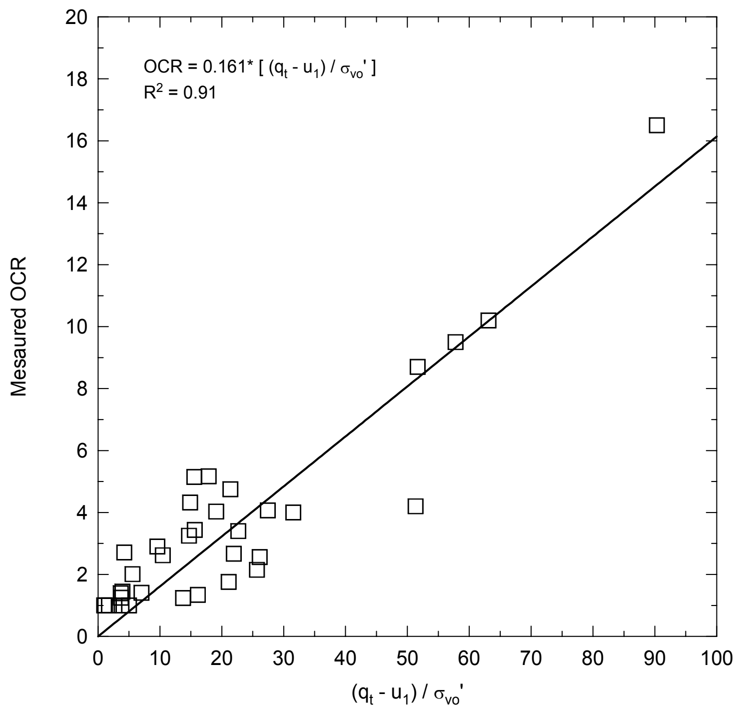


Figure 63
Measured *OCR* versus (q_t - u_1) / σ'$_{vo}$

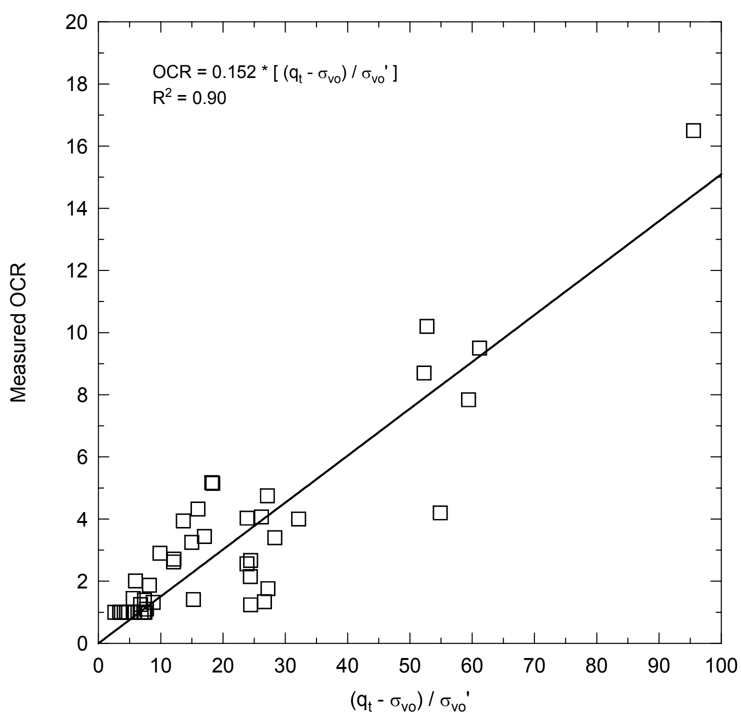


Figure 64
Measured *OCR* versus (q_t - σ_{vo}) / σ'$_{vo}$

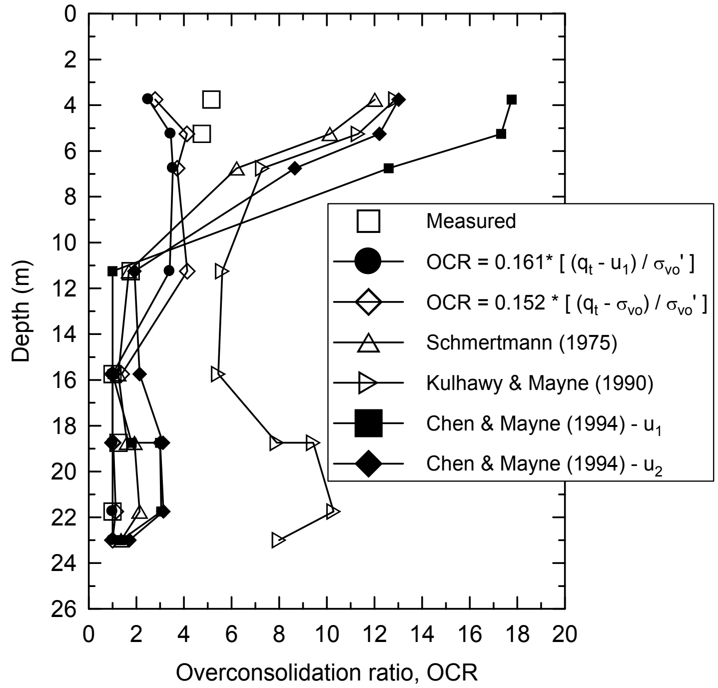


Figure 65
Measured versus predicted *OCR* for Manwell Bridge - Evangeline site

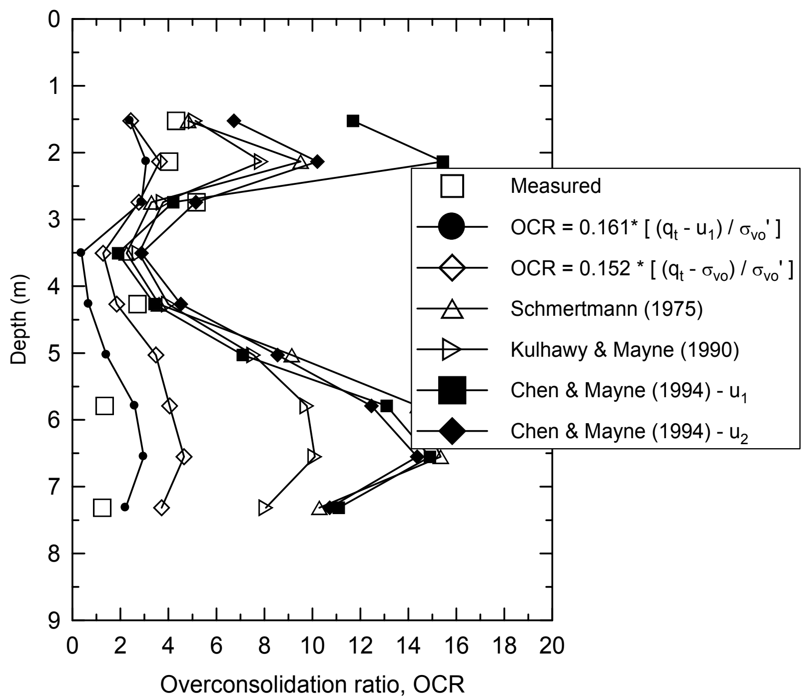


Figure 66
Measured versus predicted *OCR* for US 90–LA 88 - New Iberia site

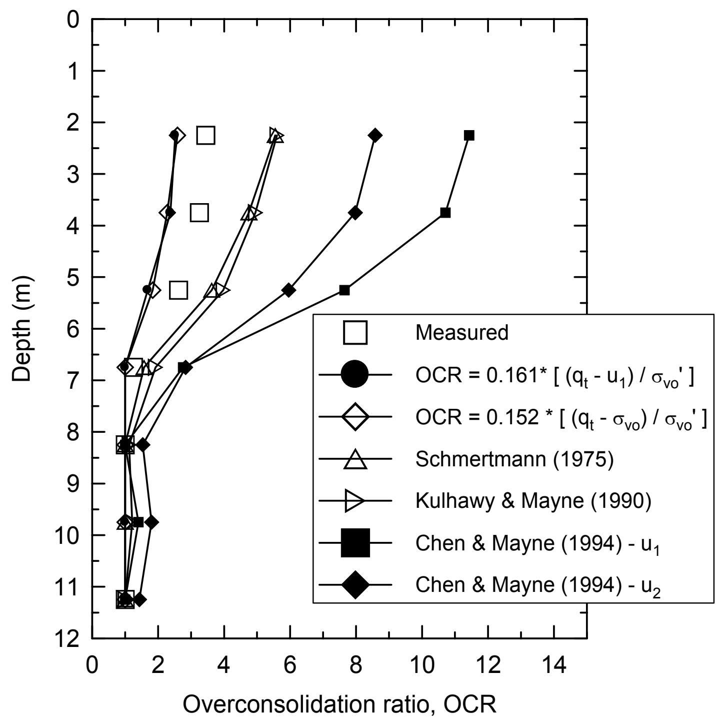


Figure 67
Measured versus predicted *OCR* for LA Peans canal bridge - Lafourche site

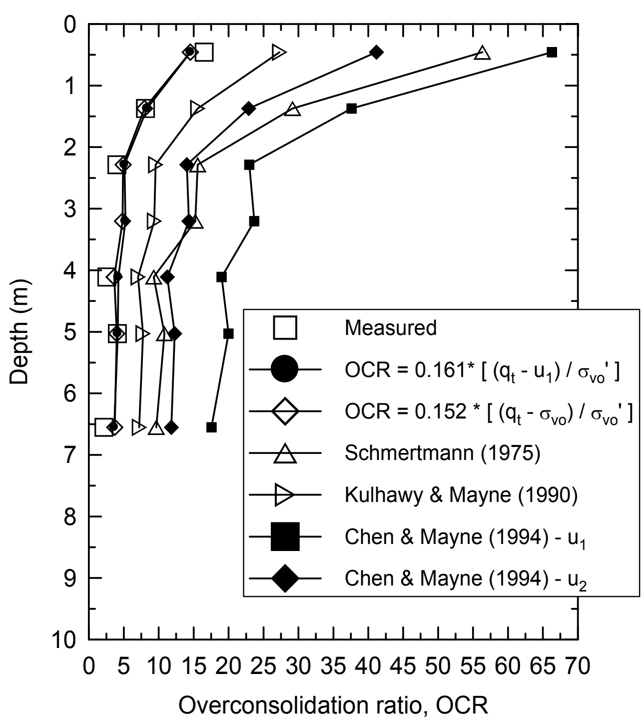


Figure 68
Measured versus predicted *OCR* for PRF site

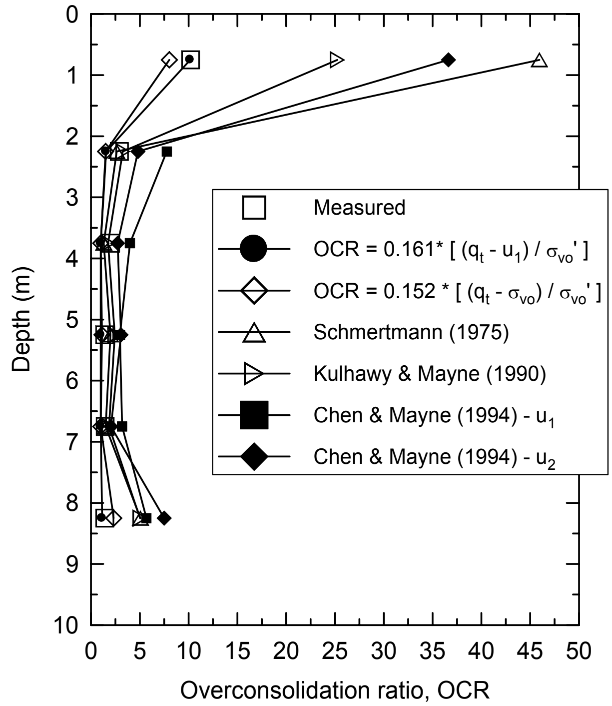


Figure 69
Measured versus predicted *OCR* for Pearl River Bridge site

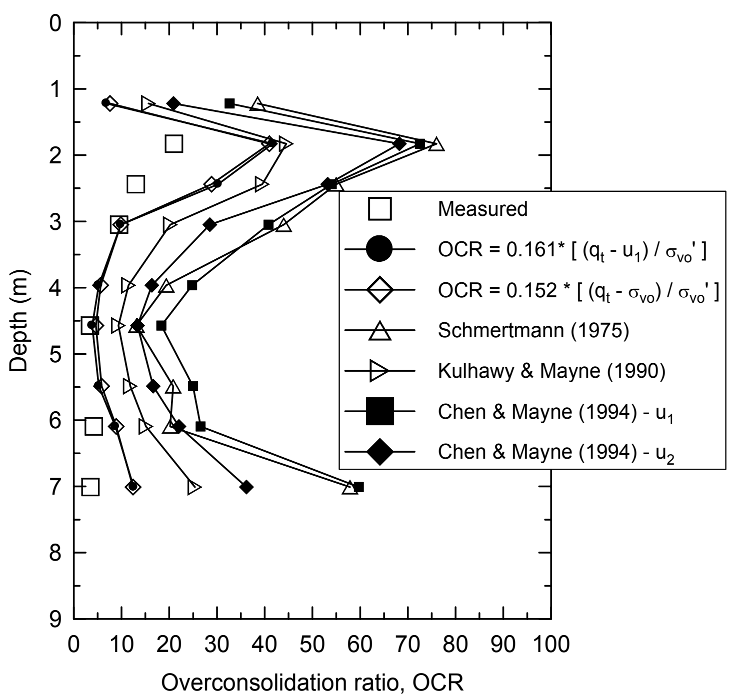


Figure 70
Measured versus predicted *OCR* for East Airport – Baton Rouge site

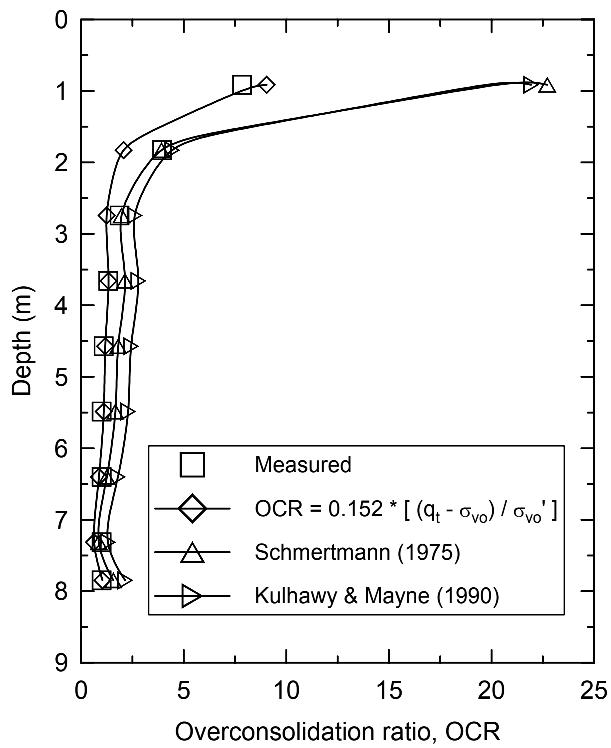


Figure 71
Measured versus predicted *OCR* for Flat River – Bossier site

Coefficient of Consolidation

During PCPT tests, penetration was stopped at pre-specified depths to conduct dissipation tests. Several dissipation tests were performed at different depths in each of the investigated sites. The dissipation test curves obtained from the different sites at different depths were presented earlier. The horizontal coefficient of consolidation (c_h) of the soil can be estimated by evaluating the characteristic shape of the dissipation curves. When investigating the shapes of dissipation curves, one can distinguish between three different types. Type I curves are the typical dissipation curve that show a gradual decrease of excess pore pressure with time. This type of curve is usually obtained in normally consolidated soils for both u_1 and u_2 measurements. Type II curves show a sudden reduction in excess pore pressure at the early stages of dissipation mainly due to unloading in overconsolidated soils with pore pressure measurement at the cone tip (u_1) (figure 72a). Once the pore pressure reduction occurs, type II curves follow the same trend as type I curves. Type III curves are usually obtained for pore pressure measurements behind the tip (u_2 and u_3) in overconsolidation soils. This is mainly due to the redistribution of excess pore pressure that usually occurs around the cone at the early stages of dissipation before it dissipates to the surrounding media. In this type of dissipation curve, the excess pore pressure continues to increase after penetration has stopped and before the real dissipation starts (figure 72b). It is interesting to notice that, in this study, all three types of dissipation curves were obtained for over-and normally-consolidated soils with pore pressure measurement at the cone tip (u_1). Sully et al. [47] evaluated type II and III dissipation curves and suggested applying certain corrections before interpreting these curves by evaluating the time t_c as described in figure 72. The time t_c is taken as the new zero time and the corresponding pore pressure is taken as the peak initial excess pore pressure for the dissipation curve.

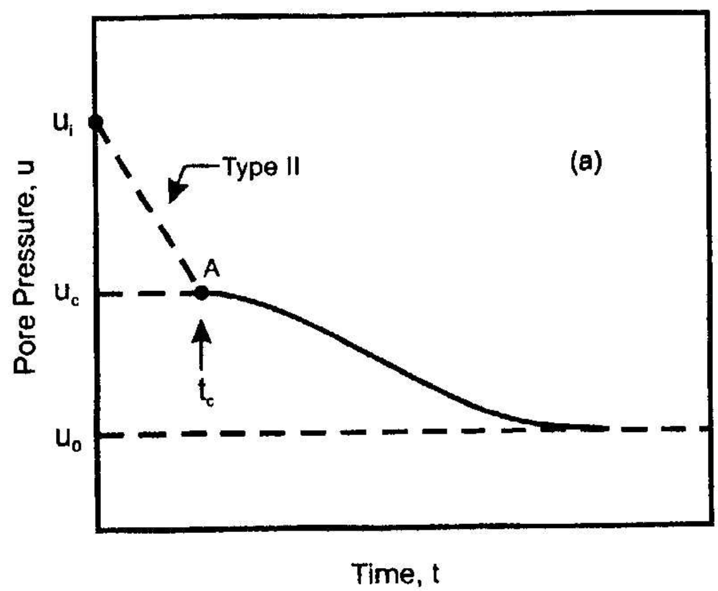
The results of piezocone dissipation tests were first used to estimate the in-situ horizontal coefficient of consolidation (c_h) using the different PCPT interpretation methods. These interpretation methods included the following methods: Teh and Houlsby [1], Levadoux and Baligh [13], Robertson and Campanella [15], Teh [14], Senneset et al. [12] (two methods), and Jones and Rust [10]. The vertical coefficient of consolidation (c_v) can then be calculated from c_h values using the relation proposed by Levadoux and Baligh [13] which is based on the (k_v/k_h) ratio. In this study, one-dimensional oedometer consolidation tests were conducted on samples oriented both vertically and horizontally in order to evaluate the vertical to horizontal coefficient of consolidations (c_v/c_h) as well as the (k_v/k_h) ratio. The predicted vertical coefficient of consolidation (c_v) obtained from the different PCPT interpretation methods was compared with

c_v values obtained from laboratory one-dimensional consolidation tests as shown in figures 73 through 79. The best fit line of the predicted to measured logarithmic vertical coefficient of consolidation ratios [$Log(c_{v\text{-}Fit}) / Log(c_{v\text{-}m})$] and the corresponding coefficients of determination (R^2) for the different interpretation methods were calculated and presented in the figures and in table 8. The arithmetic mean and standard deviation of the predicted to measured [$Log(c_{v\text{-}p}) / Log(c_{v\text{-}m})$] were also calculated and summarized in table 8. Evaluating the comparison plots shows that there are wide variations between the measured and the predicted c_v values; these variations will reflect on the reliability of these methods to predict the c_v values. However, this finding is consistent with other comparisons reported in the literature and considered acceptable and within the range of variations of laboratory-calculated c_v values [e.g., 48]. So it is not clear here whether this scatter is due to variation in PCPT dissipations or from laboratory variation. The results of the best fit relation and the arithmetic analysis indicate that the Teh and Houlsby [1] and Teh [14] methods better predict the measured vertical coefficient of consolidation (c_v) compared to the other prediction methods, as described in table 8. The $Log(c_{v\text{-}Fit})/Log(c_{v\text{-}m})$ of the best fit line and the corresponding coefficient of determination (R^2) are (1.05, 0.88) and (0.98, 0.89) for the Teh and Houlsby [1] and Teh [14] methods, respectively. And the arithmetic mean and standard deviation of $Log(c_{v\text{-}p})/Log(c_{v\text{-}m})$ are (1.07, 0.19) and (0.99, 0.22) for Teh and Houlsby [1] and Teh [1], respectively. The arithmetic mean and standard deviation of $Log(c_{v\text{-}p})/Log(c_{v\text{-}m})$ for Levadoux and Baligh [13], Robertson and Campanella [15], Senneset et al. [12] method *a* and method *b*, and Jones and Rust [10] methods are (0.75, 0.20), (0.73, 0.19), (0.82, 0.20), (0.85, 0.19) and (0.71, 0.20), respectively.

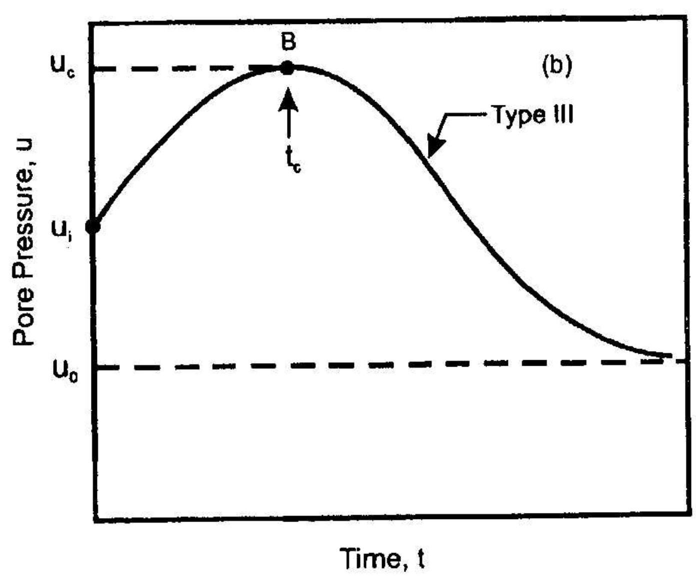
For the different investigated sites (except the Bossier site), the profile of measured c_v values obtained from the laboratory consolidation tests are compared with the c_v values predicted using the different PCPT interpretation methods and are presented in figures 80 through 85. The figures further confirm that the Teh and Houlsby [1] and Teh [14] methods predict the c_v measured values better than the other PCPT interpretation methods.

Table 8
Evaluation summary of different PCPT methods for predicting c_v

Method	Best fit calculations		Arithmetic calculations of $Log(c_{v\text{-}p})/ Log(c_{v\text{-}m})$	
	$Log(c_{v\text{-}Fit})/ Log(c_{v\text{-}m})$	R^2	Mean (cm^2/sec)	Standard Deviation
Teh and Houlsby [1]	1.05	0.88	1.07	0.19
Levadoux and Baligh [13]	0.74	0.85	0.75	0.20
Robertson and Campanella [15]	0.72	0.84	0.73	0.19
Teh [14]	0.98	0.89	0.99	0.22
Senneset et al. [12] -a	0.81	0.85	0.82	0.20
Senneset et al. [12] -b	0.84	0.86	0.85	0.19
Jones and Rust [10]	0.71	0.84	0.71	0.20



(a)



(b)

Figure 72
Types II and III of dissipation curves [47]

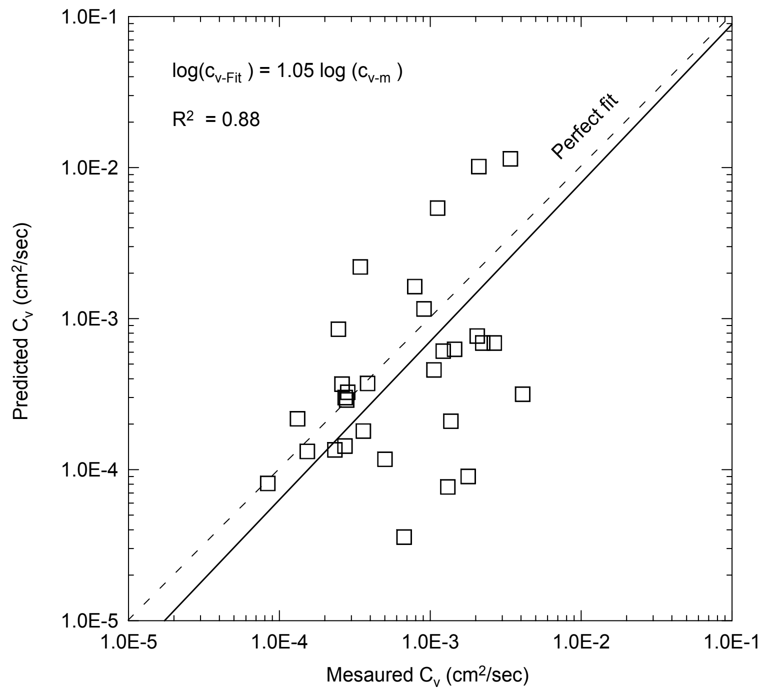


Figure 73
Measured versus predicted c_v using Teh and Houlsby [1] method

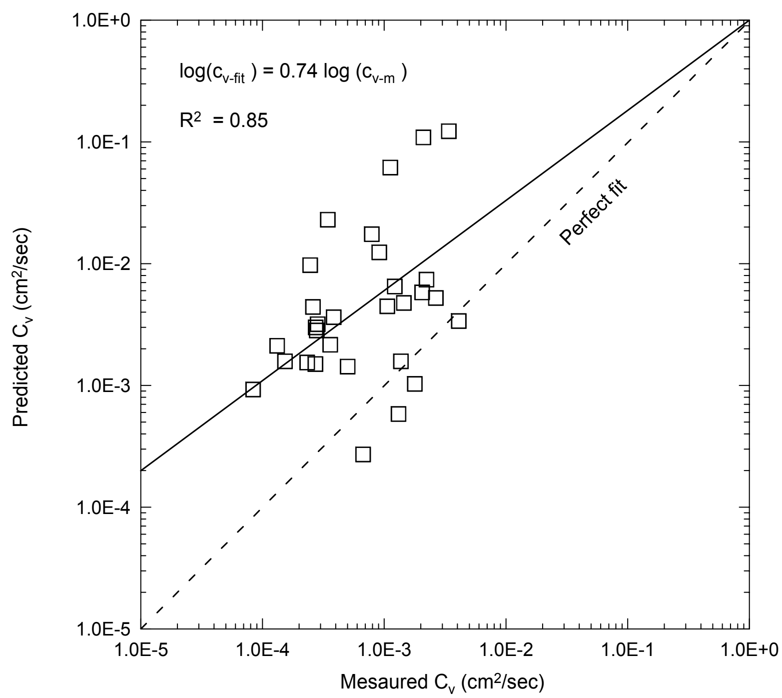


Figure 74
Measured versus predicted c_v using Levadoux and Baligh [13] method

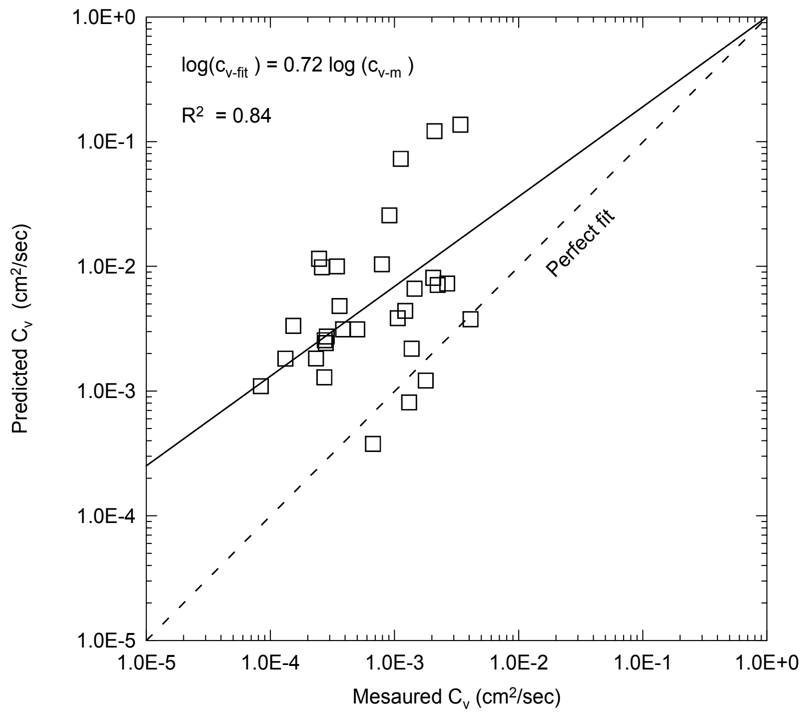


Figure 75
Measured versus predicted c_v using Robertson and Campanella [15] method

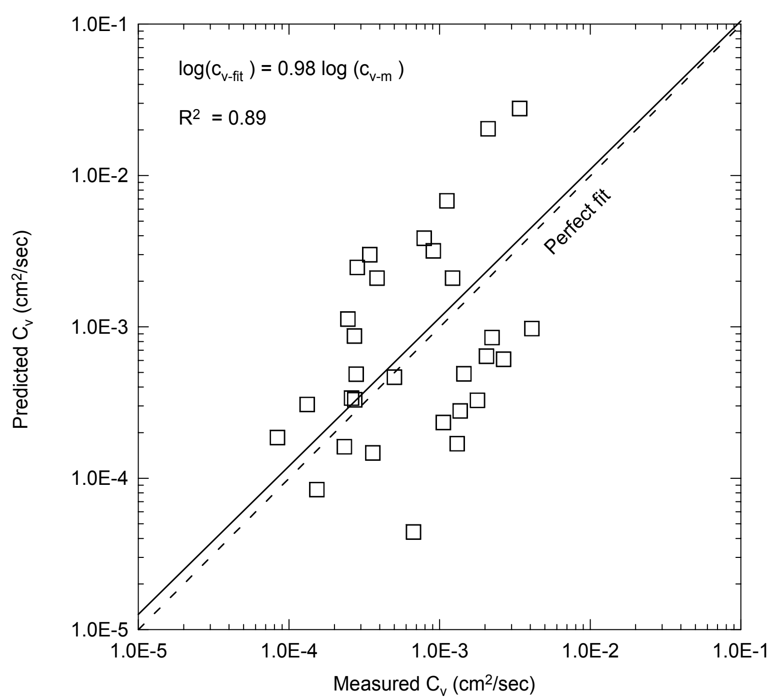


Figure 76
Measured versus predicted c_v using Teh [14] method

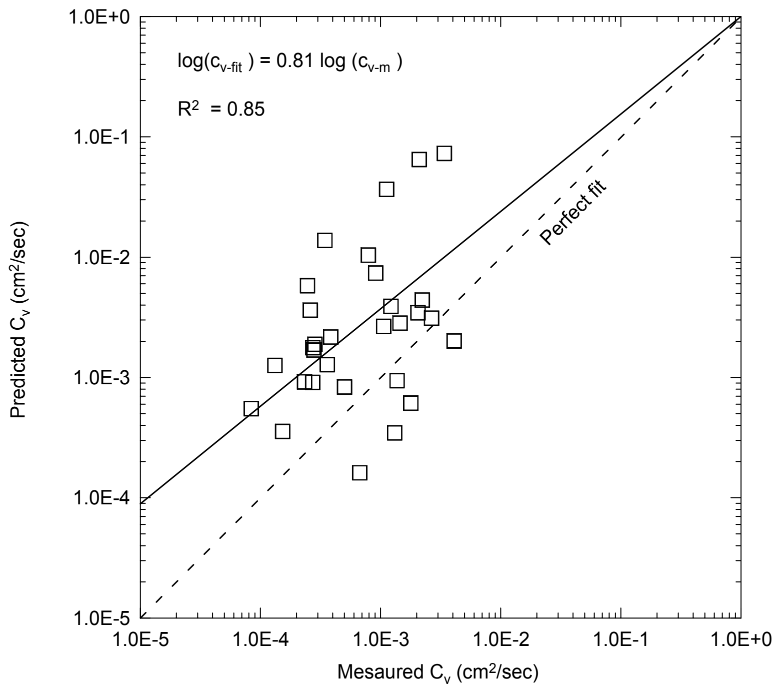


Figure 77
Measured versus predicted c_v using Senneset et al. [12] method-a

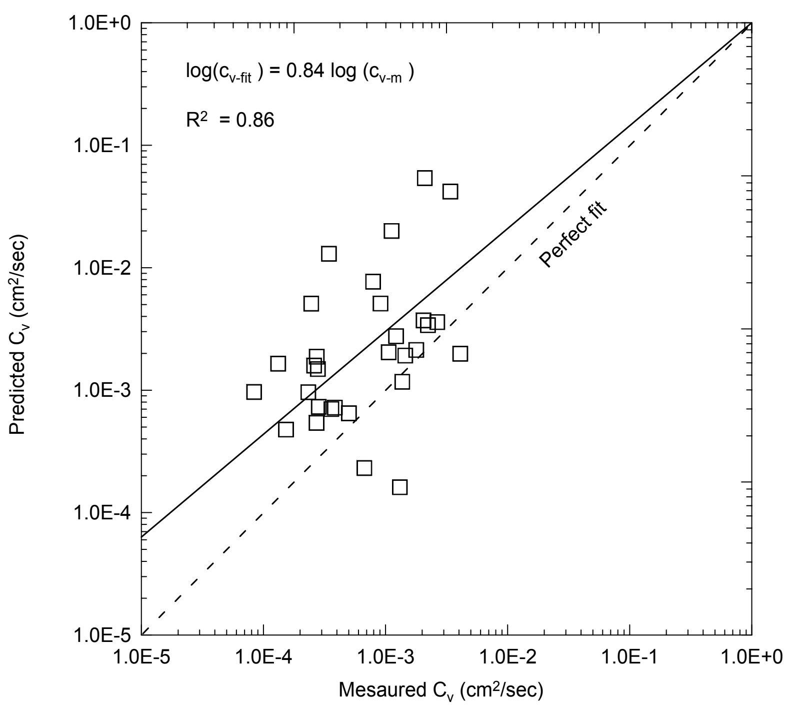


Figure 78
Measured versus predicted c_v using Senneset et al. [12] method-b

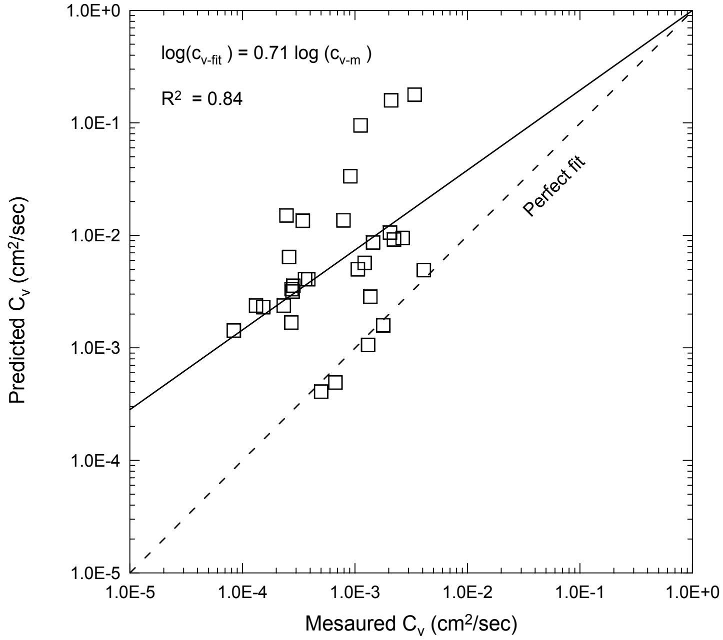


Figure 79
Measured versus predicted c_v using Jones and Rust [10] method

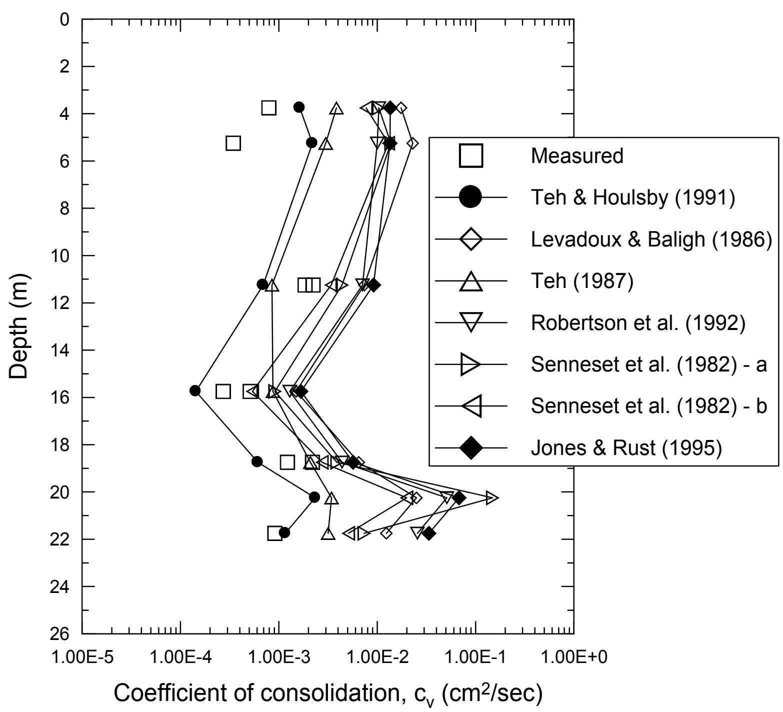


Figure 80
Measured versus predicted c_v for Manwell Bridge - Evangeline site

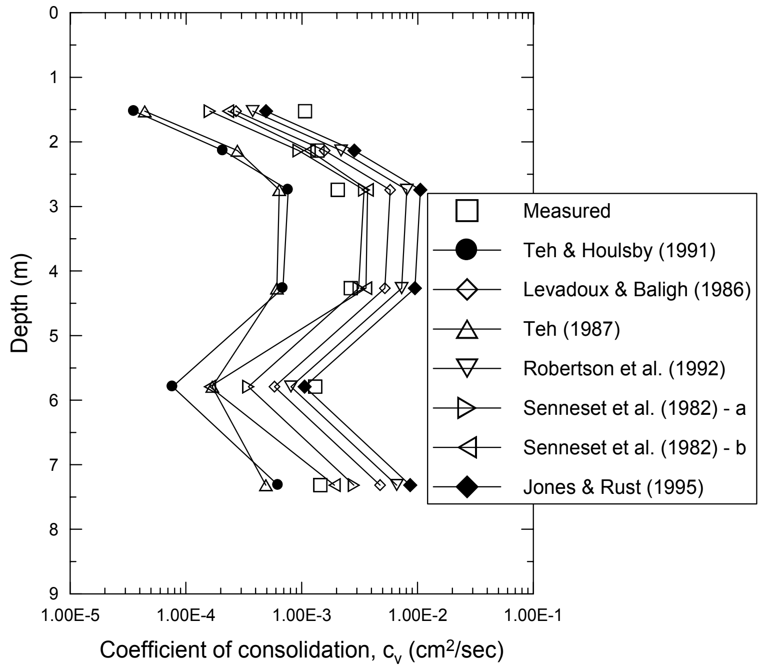


Figure 81
Measured versus predicted c_v for US 90–LA 88 - New Iberia site

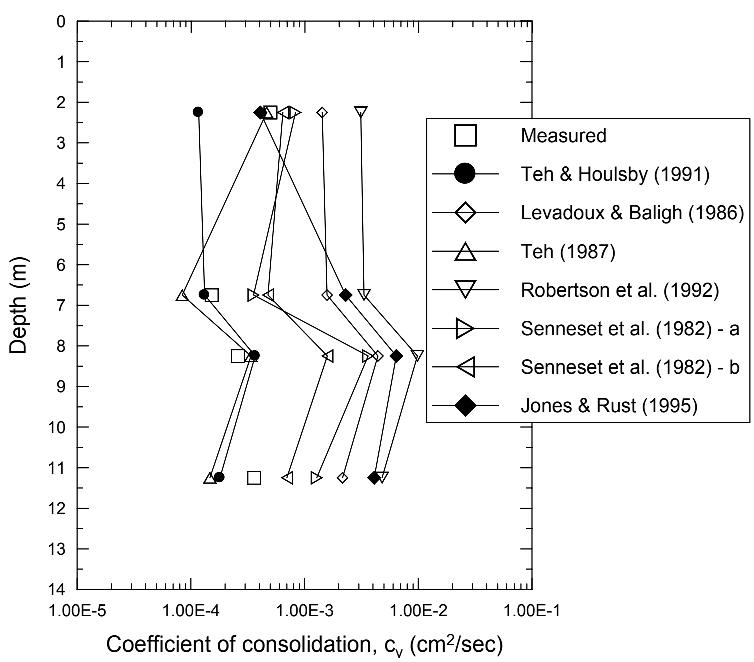


Figure 82
Measured versus predicted c_v for LA Peans canal bridge - Lafourche site

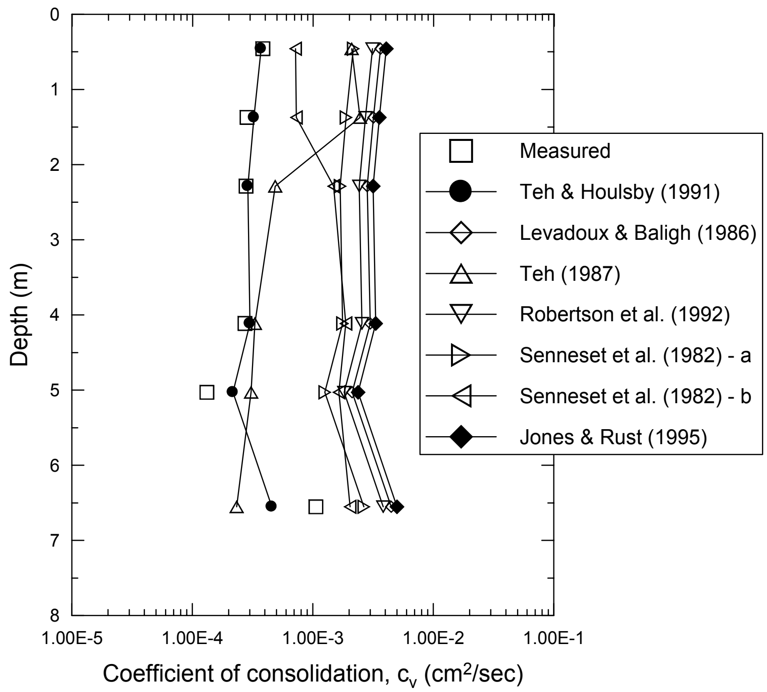


Figure 83
Measured versus predicted c_v for PRF site

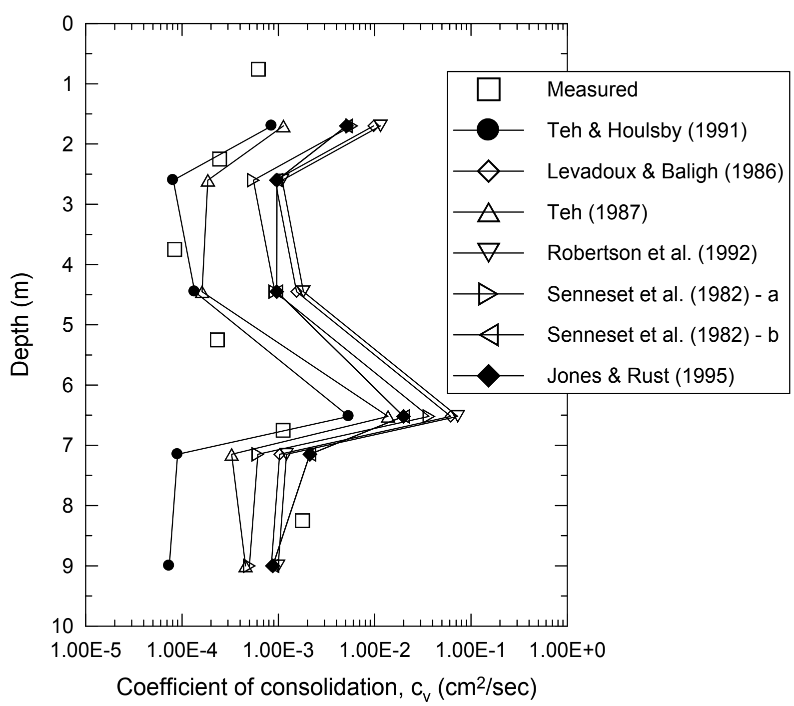


Figure 84
Measured versus predicted c_v for Pearl River Bridge site

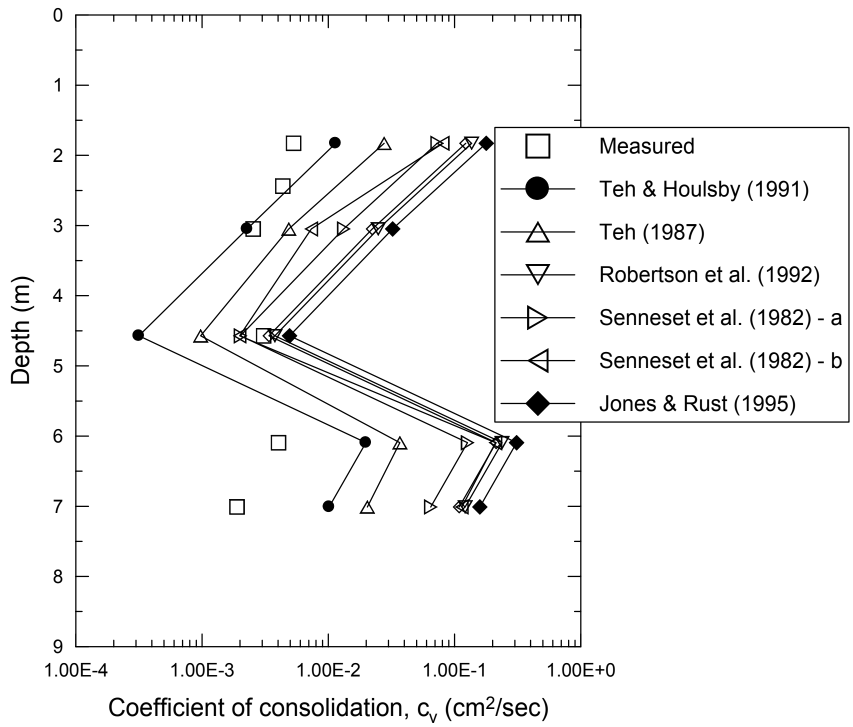


Figure 85
Measured versus predicted c_v for East Airport – Baton Rouge site

Verification

The results of this study were verified by comparing the predicted consolidation settlements with the measured field settlements in three cases: the settlement of the LTRC test wall at the PRF site, the settlement of the west approach of the embankment at John Darnell Road at LA 88, and the settlement of the east approach of the embankment at the interchange of I-10 with LA Avenue.

LTRC Test Wall

The LTRC test wall (figure 86) is an instrumented reinforced-soil wall constructed at the PRF site using silty-clay soil backfill. Its purpose is to evaluate the performance of geosynthetic-reinforced walls constructed using low quality backfill material over soft clay foundation. The wall is 6 m high and 48 m long. The main objectives from the wall's construction were to evaluate the effect of reinforcement properties on the deformation and stress distribution in the wall, and to study the soil-geosynthetics interaction mechanism [49]. Another secondary objective of the construction of the test wall was to evaluate its deformation due to the settlement of the soft clay foundation soil. Therefore, the settlement of the soft foundations under the wall was monitored during and after the construction using two horizontal inclinometer pipes installed under the wall in the longitudinal and transverse directions of the wall, as shown in figure 87. The results of the horizontal inclinometers provided the profiles of the soil settlement under the wall.

The vertical settlement profile of the wall along the transverse section was estimated with the laboratory consolidation test results and the PCPT test data by using equation 4. The foundation soil properties and the results of in-situ PCPT and dissipation tests were presented earlier. To calculate the settlement using equation 4, the constrained modulus (M) and the applied load ($\Delta\sigma$) for each soil layer needed to be determined. The constrained modulus (M), for each soil layer was predicted using the Sanglerat [7] method and the proposed correlation ($M = 3.58\ (q_t - \sigma_{vo})$), with q_t representing the average q_t value of the soil layer. The applied stress ($\Delta\sigma$) resulting from wall weight was calculated from the concept of vertical stress distribution due to embankment loading [50]. The vertical coefficient of consolidation (c_v) predicted using the Teh and Houlsby [1] method was used to describe the time rate of consolidation. The PCPT-predicted settlements (using Sanglerat [7] and proposed correlations of M) were compared with laboratory-calculated settlements and field-measured settlements using the transverse horizontal inclinometer as shown in figure 88. The figure shows that the proposed PCPT interpretation method predicted the total consolidation settlement better than the Sanglerat [7] PCPT method and the laboratory methods.

Figure 86
View of the pullout boxes at the vertical facing of the wall [49]

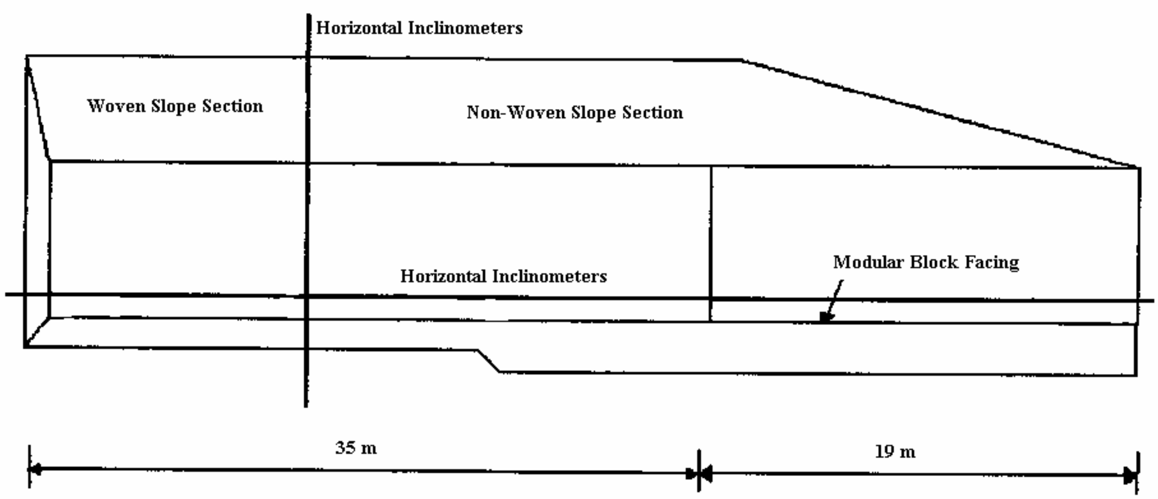


Figure 87
Layout of horizontal inclinometers under the wall [49]

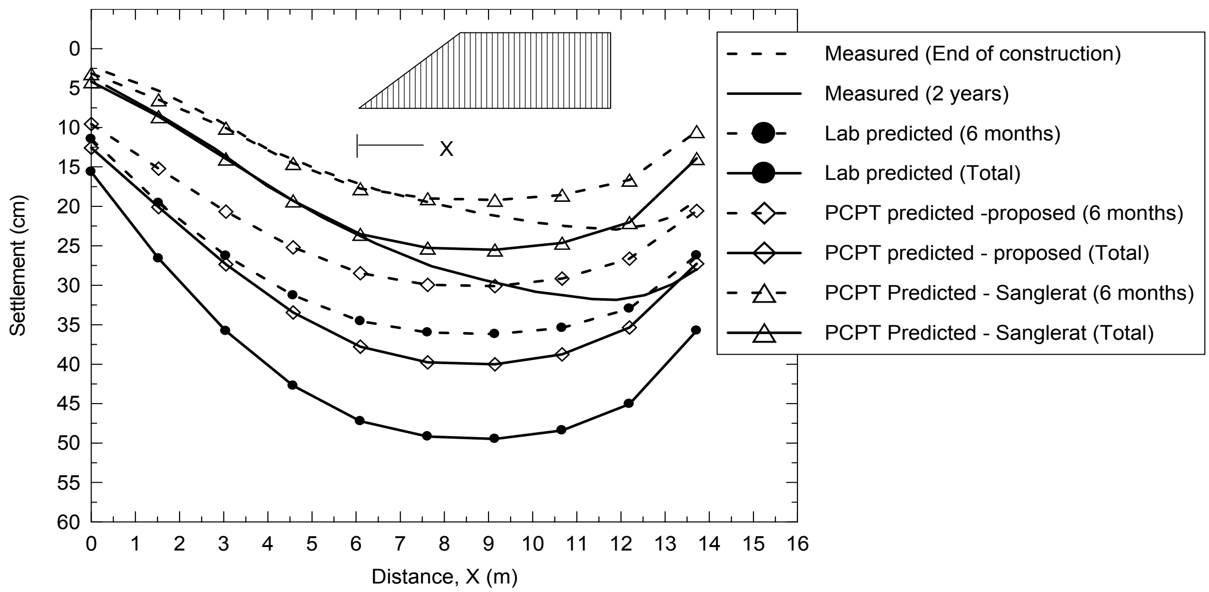


Figure 88
Comparison between PCPT-predicted, laboratory-calculated, and field-measured settlements for the LTRC test wall

John Darnell Site

A 2.56 m (8.4 ft) embankment constructed at the west approach of the John Darnell Road intersection with LA 88 was also used for verification. A surcharge height of 0.91 m (3 ft.) and wick drains with a 1.52 m (5 ft.) triangular spacing were used to accelerate the consolidation settlement. The embankment was instrumented with settlement plates to monitor the settlement with time. Five PCPT tests were conducted around the embankment down to 18m. Three PCPT tests were conducted using the u_1 measurement while the other two PCPT tests were conducted using the u_2 measurement. The profiles of PCPT test results and the corresponding CPT soil classification are presented in figure 89. The CPT soil classification indicates that the soil profile consists of silty clay soils down to about 13.5m. Two of the PCPT tests (with the u_1 measurement) were selected to conduct dissipation tests at different depths one test in each side of the embankment. Figures 90 and 91 depict the results of dissipation tests.

In order to predict the consolidation settlement from the PCPT data, the constrained modulus (M) and the vertical coefficient of consolidation (c_v) need to be evaluated first. The results of the PCPT test data (mainly the profile of q_t) were used to calculate the profile of constrained modulus (M) using the Sanglerat [7] method and the proposed correlation (equation 38). The profile of constrained modulus (M) with depth is presented in figure 92. The Teh and Houlsby

[1] method was used to estimate the vertical coefficient of consolidation (c_v) from dissipation test curves in order to describe the time rate of consolidation. The profile of predicted c_v values with depth is shown in figure 93. The PCPT correlated M and c_v were then used to predicted the consolidation settlement with time from PCPT data. The PCPT predicted consolidation settlements are compared with the measured field settlements from settlement plates and with the laboratory-calculated settlements, as shown in figure 94. The comparison figure of this site shows that the PCPT interpretation methods (Sanglerat [7] and proposed correlations) and the laboratory calculations closely predicted the total consolidation settlement.

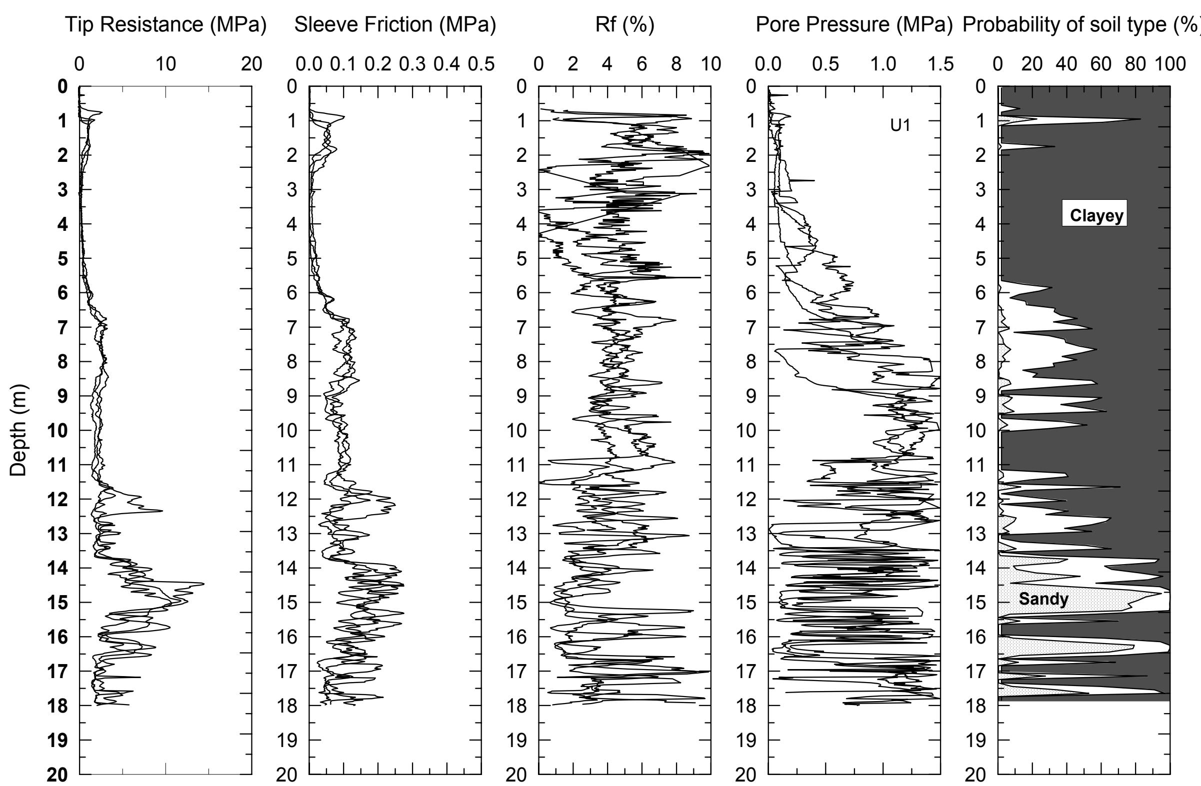


Figure 89
Results of PCPT tests at John Darnell Road – LA 88 site

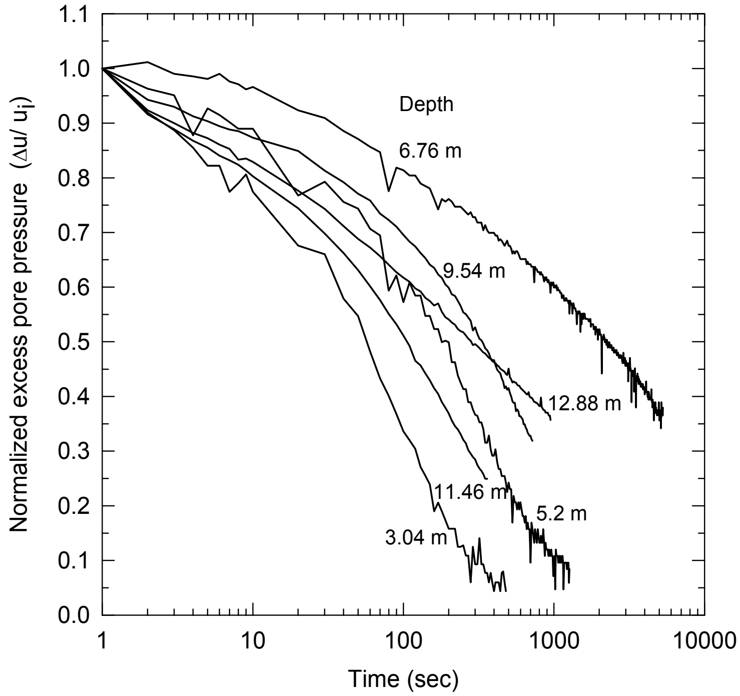


Figure 90
Dissipation test results at John Darnell site – first location

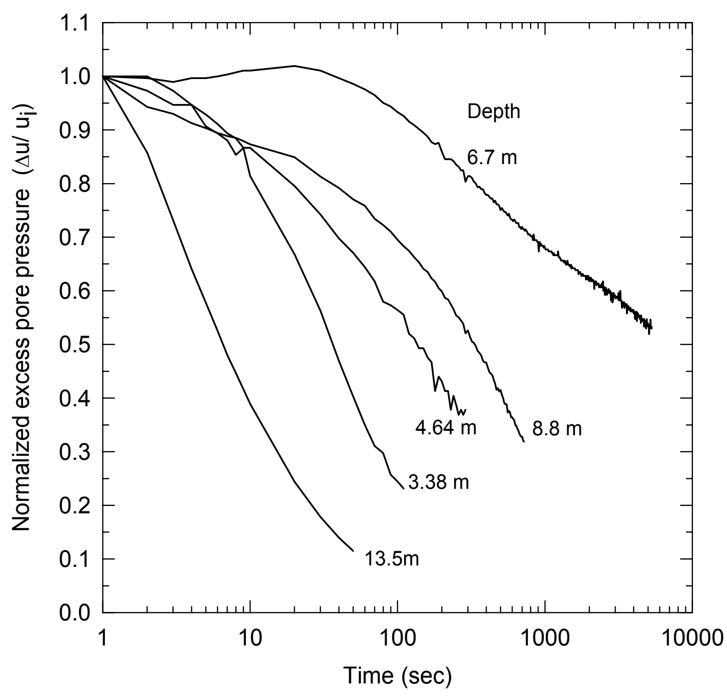


Figure 91
Dissipation test results at John Darnell site – second location

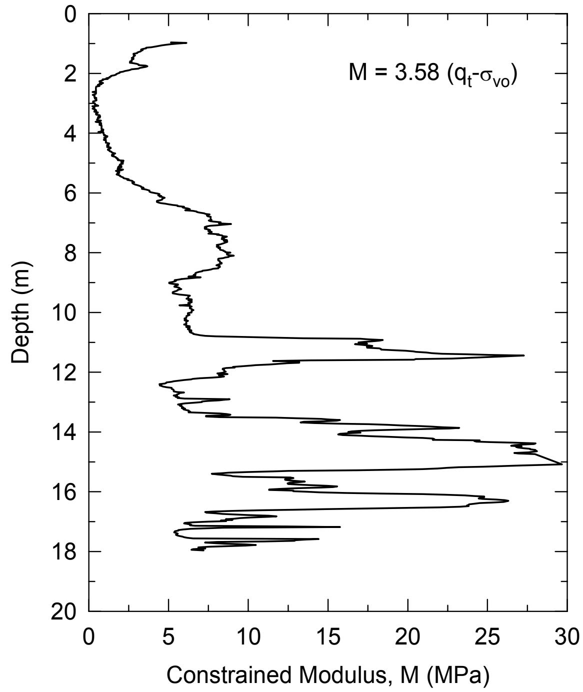


Figure 92
Predicted constrained modulus (*M*) at John Darnell site

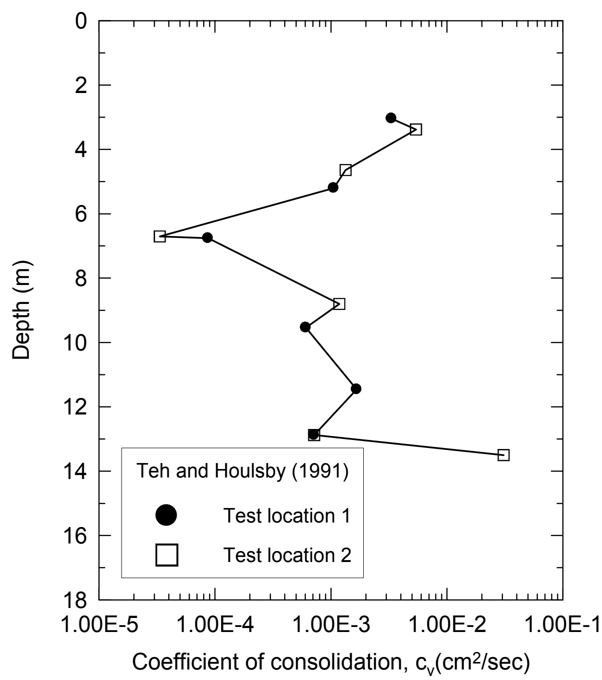


Figure 93
Predicted vertical coefficient of consolidation (c_v) at John Darnell site

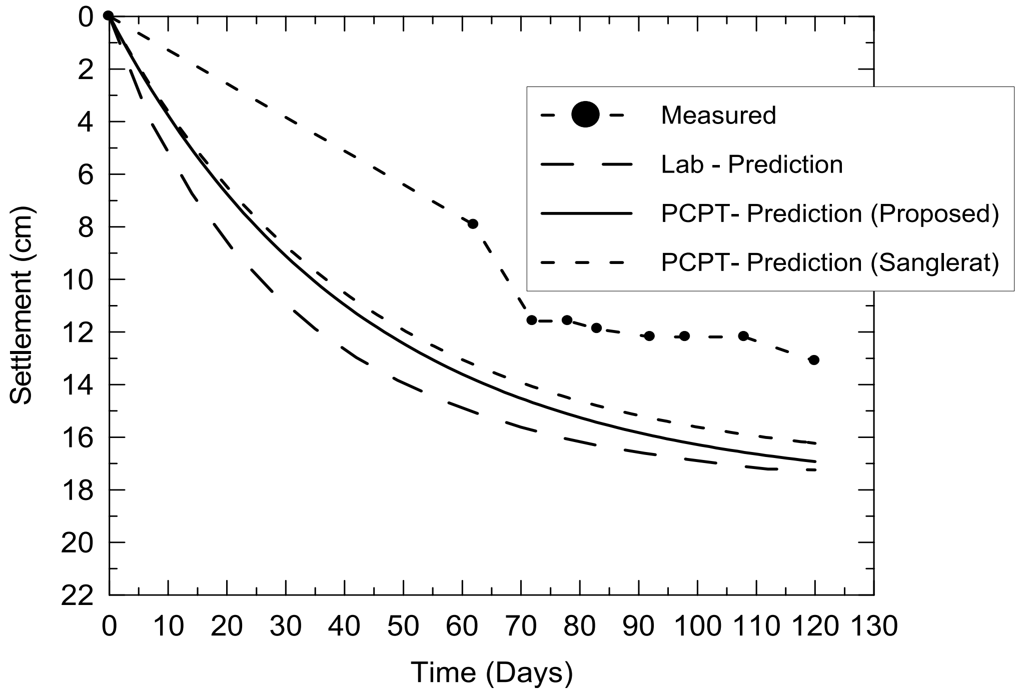


Figure 94
Comparison between PCPT-predicted, laboratory-calculated, and field measured settlements for the embankment at John Darnell Road – LA 88

Louisiana Avenue Site

The third site selected for verification was the east approach of a 6.70 m (22 ft.) high embankment constructed at the intersection of LA Avenue with I-10, near Lafayette. A surcharge of 0.91 m (3 ft.) in height was placed to accelerate the consolidation settlement. In addition, wick drains were also installed with 2.1 m (5 ft.) triangular spacing and 14.63m length. Settlement plates were instrumented to monitor the settlement of the embankment over time. At this site, six PCPT tests were conducted around the embankment down to 18m. Three PCPT tests were conducted using the u_1 measurement and three PCPT tests were conducted using the u_2 measurement. Figure 95 presents the profiles of PCPT test measurements. The CPT soil classification shows that the soil deposit consists of clayey soil from surface to about 12 m. Two of the PCPT tests (with u_1 measurement) were selected to conduct the dissipation tests. One set of dissipation tests was conducted in each side of the embankment. The results of dissipation tests are shown in figures 96 and 97 for the two locations.

The results of the PCPT test data (q_t) were used to calculate the profile of constrained modulus (M) with depth using Sanglerat [7] method and the proposed correlation (equation 38), as shown in figure 98. The results of dissipation tests were also used to predict the profile of the vertical coefficient of consolidation (c_v) with depth using the Teh and Houlsby [1] method. Figure 99 presents the profile of predicted c_v values with depth from two locations. The total consolidation settlement was calculated from the predicted M values and using equation 4, while the time rate of consolidation was evaluated using the predicted c_v values from dissipation tests. Figure 100 depicts the comparison between the PCPT predicted consolidation settlements, the measured field settlements from settlement plates, and the laboratory-calculated settlements. The comparison shows that the proposed PCPT interpretation method can predict the total consolidation settlement better than the Sanglerat [7] PCPT method and the laboratory calculations.

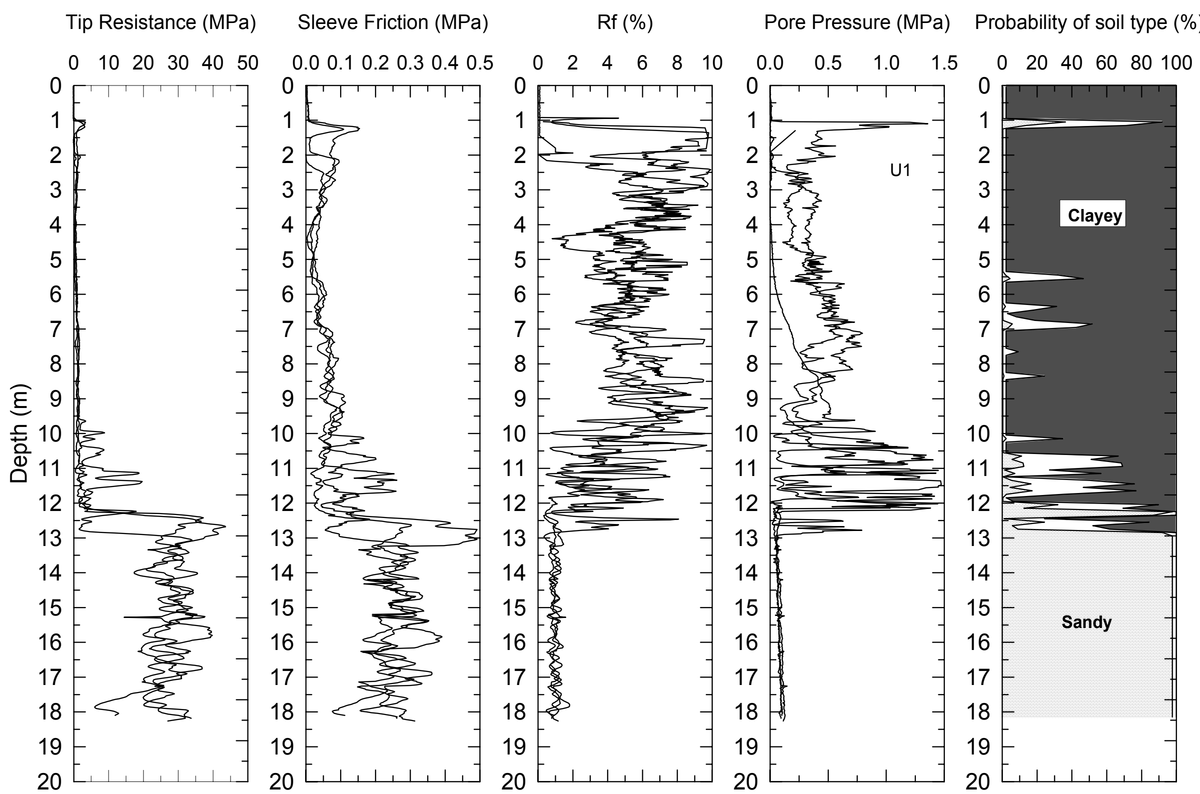


Figure 95
Results of PCPT tests at LA Avenue site

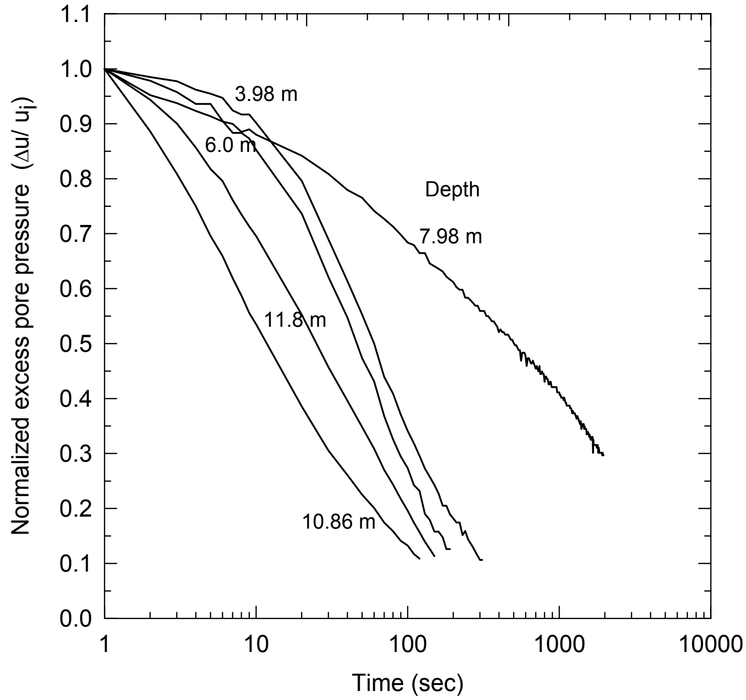


Figure 96
Dissipation test results at LA Avenue site – first location

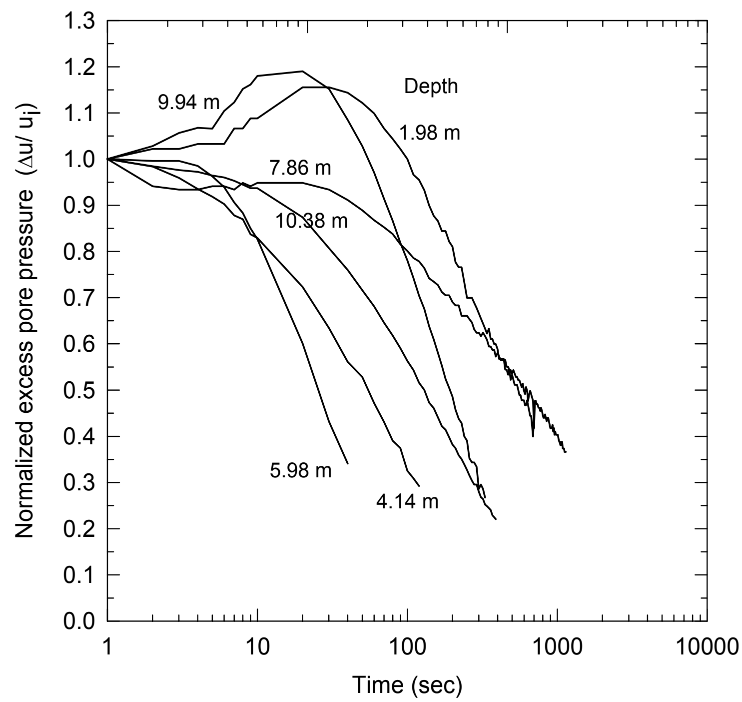


Figure 97
Dissipation test results at LA Avenue site – second location

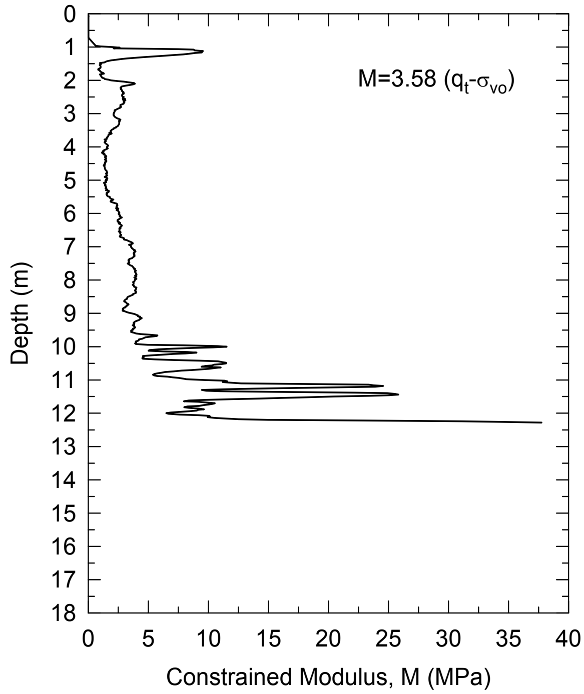


Figure 98
Predicted constrained modulus (*M*) at LA Avenue site

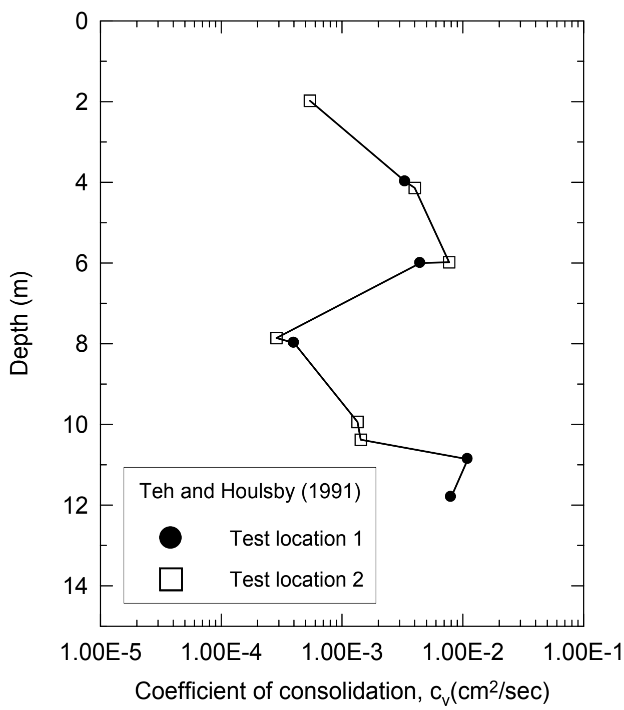


Figure 99
Predicted vertical coefficient of consolidation (c_v)

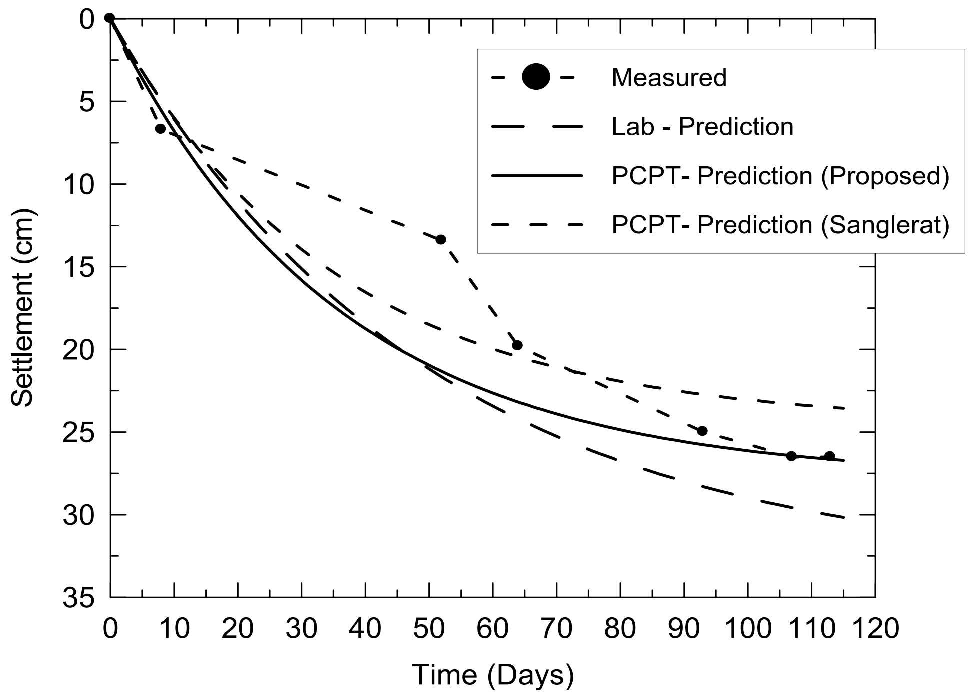


Figure 100
Comparison between PCPT-predicted, laboratory-calculated, and field-measured settlements for the embankment at LA Avenue site

CONCLUSIONS

This report presents the evaluation of the current PCPT interpretation methods' capability to reasonably predict the consolidation parameters of cohesive soils using the piezocone penetration and dissipation tests. These parameters are the constrained modulus (M), the vertical coefficient of consolidation (c_v) and the overconsolidation ratio (OCR). Seven sites in Louisiana were used in this study. At each site, in-situ PCPTs were performed and dissipation tests were conducted at different penetration depths. Undisturbed shelby tube samples collected adjacent to the PCPT tests were used to calculate the laboratory reference soil parameters from the results of one-dimensional oedometer consolidation tests. The predicted values of M, c_v, and OCR obtained from the different PCPT interpretation methods were compared with the laboratory-calculated reference parameters. The reliability of the different interpretation methods was evaluated and new correlations for M and OCR were also developed. The result of this study was verified by comparing settlement predicted from PCPT test data with the measured settlements in three selected sites. Based on this study, the following conclusions can be drawn.

The predicted constrained modulus (M_p) obtained from four PCPT interpretation methods, Kulhawy and Mayne [9], Jones and Rust [10], Sanglerat [7], and Senneset et al. [8], were compared with the measured constrained modulus (M_m) from the oedometer laboratory tests. Then the best fit line of (M_p/M_m) and the corresponding coefficient of determination (R^2) were determined. The results of comparison and arithmetic analysis (mean and standard deviation) showed that the Sanglerat [7] method can predict M better than the other three methods. The Kulhawy and Mayne [9] and Senneset et al. [8] methods, however, overpredict the constrained modulus by a factor of 1.5 to 2.0 times, while the Jones and Rust [10] method underpredicts the constrained modulus by a factor of 0.8 times. Two linear correlations were also developed between M and both the corrected cone tip resistance (q_t) and the net cone tip resistance ($q_t - \sigma_{vo}$). The comparison between the profiles of laboratory-measured M and the predicted M from the different interpretation methods including the proposed correlations for the different sites showed that the two proposed relations can predict M better than the other methods. However, this result is expected since these relations were developed using the current data. Therefore, the proposed relations need to be validated using data from new selected sites.

Four different PCPT methods were selected and evaluated for their capability to predict the overconsolidation ratio (OCR) using the PCPT test data. These methods are the Schmertmann [16] method, the Kulhawy and Mayne [9] method, and the Chen and Mayne [18] methods using pore pressure at the cone tip (u_1) and at the cone base (u_2). The predicted *OCR*s from different methods were then compared with the measured *OCR*s obtained from the oedometer consolidation laboratory tests; then the best fit line and the corresponding R^2 were determined. The results of comparison and arithmetic analysis indicated clearly that all the PCPT interpretation methods overestimated the *OCR* by a factor ranging from 2.0 times for Kulhawy and Mayne [9] to 4.27 times for Chen and Mayne [18] using u_1. Possible correlations between the measured *OCR* and both $(q_t - u_1) / \sigma'_{vo}$ and $(q_t - \sigma_{vo}) / \sigma'_{vo}$ ratios were examined, and linear correlations were obtained. The profile of laboratory-measured *OCR* using oedometer tests for the different investigated sites was compared with the predicted *OCR* using the different PCPT interpretation methods including the proposed two correlations. This comparison indicated that the proposed two relations give better prediction of the measured *OCR* than the other methods. Again, this result is also expected, and the proposed relations need to be validated using data from new sites.

The results of dissipation tests at each site were used to estimate the horizontal coefficient of consolidation (c_h) using the different PCPT interpretation methods. The vertical coefficient of consolidation (c_v) was then calculated from c_h values using the ratios (c_v/c_h, k_v/k_h) from the results of one-dimensional consolidation tests conducted on samples oriented both vertically and horizontally. The logarithmic c_v values predicted from the different interpretation methods were compared with logarithmic c_v values obtained from laboratory consolidation tests. The interpretation methods include the following methods: The Teh and Houlsby [1], Levadoux and Baligh [13], Robertson and Campanella [15], Teh [14], Senneset et al. [12] methods *a* and *b*, and Jones and Rust [10]. For the different methods, the best fit line of the predicted to measured logarithmic c_v and the corresponding R^2 were calculated along with the arithmetic mean and standard deviation of the predicted to measured logarithmic c_v [$Log(c_{v-p}) / Log(c_{v-m})$]. The results of comparison plots show wide scatters between the measured and the predicted Log c_v values, which might question the reliability of these methods to predict the c_v values. This scatter, however, is consistent with other comparisons reported in the literature and can be considered acceptable compared to the variation of c_v values obtained from laboratory tests [48]. The results of the best fit line and the arithmetic analysis indicated that the Teh and Houlsby [1] and Teh [14] methods can predict the vertical coefficient of consolidation (c_v) better than the other methods. Comparing the profiles of the measured c_v values and the predicted c_v values using the different interpretation methods confirmed that the Teh and Houlsby [1] and Teh [14] methods can predict c_v values better than the other methods.

The findings of this study were verified by comparing the predicted settlements using the proposed PCPT correlation, the Sanglerat [7] PCPT method, laboratory-calculated settlement, and the actual field settlements measured using settlement plates at three selected sites. The results of this verification show that the proposed PCPT method can predict the total settlement better than Sanglerat [7] PCPT method and the laboratory-calculated settlement from parameters obtained from the consolidation tests.

RECOMMENDATIONS

The availability of the cone penetration test systems at LA DOTD will eventually make the estimation of the magnitude and time rate of settlement easier, faster, cheaper, and more reliable than the 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 soil parameters in soils where it is impossible to obtain adequate sampling. Therefore, based on the results of this study, it is recommended that the LA DOTD engineers gradually start to implement the PCPT technology, particularly to estimate the consolidation settlement of fine-grained soils, in conjunction with the traditional laboratory calculation of settlements. The comparison between the consolidation settlements predicted from the PCPT data, the calculated settlements from laboratory consolidation parameters, and the field-measured settlements should be continued until the LA DOTD engineers build enough confidence in the PCPT interpretation methods. With increasing confidence and experience, LA DOTD engineers can move toward replacing conventional subsurface exploration with piezocone penetration and dissipation tests with the estimation of consolidation settlement.

To facilitate the use of the PCPT methods to estimate the magnitude and time rate of consolidation settlement of fine-grained soils, it is highly recommended that a friendly computer program be developed to profile the consolidation characteristics of soil layers with depth. The constrained modulus of each soil layer can be estimated using the piezocone penetration test data (q_c, u_1), while the coefficient of consolidation can be estimated from the piezocone dissipation curves obtained at different penetration depths. The development of this program should examine the possibility of using soil properties and measured pore pressures to predict a continuous profile of the vertical coefficient of consolidation (c_v) with depth and to detect thin drainage layers.

It is recommended that the relations proposed in this study to estimate the constrained modulus (*M*) and the overconsolidation ratio (*OCR*) from PCPT data be validated. The Teh and Houlsby [1] method usedto estimate the vertical coefficient of consolidation (c_v) should be validated by comparing the measured values at new selected sites with the predicted values obtained from the proposed relations.

A training manual and workshops will be needed to train the LA DOTD engineers to use the PCPT methods for evaluating the consolidation parameters needed to estimate total fin-grained soil settlement and time rate.

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