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16. Abstract

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Statistical analyses were conducted on the cone soundings and showed that the results are repeatable at each test site within tolerable deviation. Statistical models for predicting the resilient modulus were proposed based on the field and laboratory test results of two soil types and two cases of stresses: in situ conditions and traffic loading. These models correlate the resilient modulus to the cone penetration test parameters, basic soil properties, and in situ stress conditions of the soil. The models for the cohesive soil were validated by predicting the resilient modulus of the other soils that were not used in the development of these models. Predicted and measured values of the resilient modulus are in good agreement. This research provided a preliminary validation of predicting the resilient modulus of subgrade soils utilizing the cone penetration test.

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Investigation of the Applicability of Intrusion Technology to Estimate the Resilient Modulus of Subgrade Soil

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ABSTRACT

An innovative and simple approach is presented for estimation of the resilient modulus of subgrade soils utilizing the cone penetration test. Field and laboratory testing programs were carried out at seven sites that comprise three common soil types in Louisiana. Site characterization was conducted using cone penetration tests, in which continuous measurements of the cone tip resistance and sleeve friction are recorded. Undisturbed and disturbed soil samples were also obtained from different depths at the investigated sites. Laboratory tests were conducted on soil samples to determine the resilient modulus, strength parameters, physical properties, and compaction characteristics. Results of both field and laboratory testing programs were analyzed and critically evaluated.

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IMPLEMENTATION STATEMENT

Currently, the design of flexible pavements is generally conducted based on static properties such as California Bearing Ratio (CBR) and soil support value. These properties do not represent the actual response of the pavement layers under traffic loadings. Recognizing this deficiency, the current and the 2002 American Association of State Highway and Transportation Official's guide for design of pavement structures recommended the use of resilient modulus for characterizing the base and subgrade soil and for the design of flexible pavements.

This report presents the findings from a pilot investigation to assess the applicability of intrusion technology to estimate the resilient modulus of subgrade soils. Models for predicting soil resilient modulus from cone penetration test parameters, basic soil properties, and soil insitu stress conditions were developed.

These models were successfully used in several overlay projects to evaluate the subgrade stiffness along with conventional approach. The evaluation is on-going by identifying field projects in each district and applying this technology during the rehabilitation design stage.

In addition to the above exposure of minicone technology to estimate resilient modulus, workshop sessions are planned for the dissemination of this approach to DOTD design engineers. This will accelerate the implementation of this effective and fundamental approach in pavement design and analysis.

For a successful implementation of this study, it is anticipated that DOTD will provide the necessary budgetary funds required for the acquisition of the Continuous Intrusion Miniature Cone Penetration Test system for each district.

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INTRODUCTION

Characterization of base and subgrade soil is essential for the design and analysis of pavement structures. Design of flexible pavements is generally based on static properties such as California Bearing Ratio (CBR) and soil support value. These properties do not represent the actual response of the pavement layers under traffic loadings. Consequently, the American Association of State Highway and Transportation Official's (AASHTO) guide for the design of pavement structures *[1]* recommended the use of a resilient modulus (M_r) for characterizing the base and subgrade soil and designing flexible pavements. As a result, the use of resilient modulus by highway transportation agencies is becoming increasingly popular.

The resilient modulus is usually determined from laboratory or field nondestructive test methods (NDT). The laboratory procedures are considered laborious, time consuming, and highly expensive. The field nondestructive test procedures have certain limitations with repeatability of test results and the identification of layer properties underlained by soft layers. The shortcomings of these test methods signify the need for an in situ technology that determines the resilient characteristics of subgrade and base soils underneath a pavement.

Among the present in situ methods, cone penetration testing (CPT) is considered the most frequently used method for charactering geomedia because the CPT method is economical, fast, and provides repeatable and reliable results. The CPT advances a cylindrical rod with a cone tip into the soil and measures the tip resistance and sleeve friction from the intrusion. The resistance parameters are used to classify soil strata and to estimate strength and deformation characteristics of soils such as Young's modulus (E) and shear modulus (G). It is expected that the CPT method, if properly calibrated, can also be used to determine the resilient characteristics of subgrade soils.

This report presents the results of the research effort undertaken at the Louisiana Transportation Research Center (LTRC) to investigate the applicability of CPT technology in determining the resilient modulus of subgrade soils. For this purpose, laboratory and field testing programs were conducted at seven sites that comprise common soil types in Louisiana (cohesive soil such as clay and silty clay and cohesionless soil such as sand).

Field tests consisted of cone penetration tests and undisturbed soil sampling (using thin-walled Shelby tubes) next to the cone penetration tests. Cone penetration tests were conducted using the 2 cm^2 miniature friction cone penetrometer and the 15 cm^2 friction cone penetrometer. Laboratory tests consisted of repeated load triaxial tests on undisturbed soil samples to evaluate the resilient characteristics of these soils. Other soil tests were conducted to characterize the soils' physical properties and compaction and strength characteristics.

Analyses were conducted to assess the reliability and repeatability of the miniature cone penetrometer. Statistical analyses were performed to correlate the cone penetration test data and the resilient characteristics of the investigated soils. Models were proposed in which the effects of soil type, moisture content, unit weight, and stresses on the predicted resilient modulus by the cone penetration test were investigated. The resilient modulus was investigated under in situ stress conditions and traffic loading. These models were calibrated using the test results of two soils and were used to predict the resilient modulus of other soils.

This report presents the results of a pilot investigation on the applicability of the cone penetration test to evaluate the resilient modulus of subgrade soil.

Background

The design and evaluation of pavement structures on base and subgrade soils need a significant amount of supporting data such as traffic loading characteristics, material properties (base, subbase, and subgrade), environmental conditions, and construction procedures. Currently, empirical correlations developed between field and laboratory material properties are used to obtain highway performance characteristics *[2]*. These correlations do not satisfy design and analysis requirements because they neglect to address all possible failure mechanisms in the field. Also, most of these methods, which use California Bearing Ratio and soil support values, do not represent the conditions of a pavement subjected to repeated type traffic loading. After several discussions by peers and pioneers in this field, it was decided that a dynamic property is needed to characterize the pavement materials *[3]*. Recognizing this, in 1986, the AASHTO design guide for pavement structures recommended the use of a dynamic resilient modulus for the mechanistic analysis and design of pavement structures.

The resilient modulus (M_r) in a repeated load test is defined as the ratio of the maximum deviator stress (σ_d) and the recoverable elastic strain (ε_r)

$$M_r = \frac{s_d}{e_r} \tag{1}$$

The definition of the resilient modulus is illustrated in figure 1. Investigators have endeavored a variety of in situ and laboratory methods to evaluate the dynamic resilient properties of subgrade soil. Laboratory methods are mainly conducted using triaxial systems, simple shear, resonant column, gyratory, and the hollow cylinder testing device. Mohammad et al. *[4]* provided a summary of the research results conducted using the above equipment. Mohammad et al. *[4], [5]* also evaluated the influence of testing procedures and measurement systems on the resilient modulus test results for granular and cohesive soils. Several transportation agencies, including the Louisiana Department of Transportation and Development (DOTD), have already started implementing the 1986 AASHTO design procedure. A state of the art laboratory technique to
determine the resilient modulus of soils utilizing a modified AASHTO T-294 test procedure and an internal (in-cell) measurement system was adopted by Mohammad et al. *[4], [5]*.

Different factors affect the resilient modulus of soils. These include the moisture content, dry unit weight, seasonal variation, confining and deviator stresses, size of the specimen, stress pulse shape, duration, frequency and sequence of stress levels, testing equipment and specimen preparation, and conditioning methods.

Several in situ methods have also been developed to determine the resilient modulus. Both laboratory and in situ methods are improving with developments in hardware technologies, particularly in areas such as data acquisition systems and computer technology. However, the laboratory methods are rather laborious, time consuming, and expensive, and they require a highly sophisticated testing system. Moreover, these methods are still being modified to make them more accurate and reliable. In situ nondestructive testing (NDT) using Dynaflect and Falling Weight Deflectometer (FWD) has also been subjected to a lot of scrutiny. Deflections of pavement materials are measured in the field by this equipment and these deflections are used with back calculation subroutines for estimating resilient properties. Several types of back calculation software are already available and reported in the literature.

$$\text{Resilient modulus, } M_r = \frac{\sigma_d}{\varepsilon_r}$$

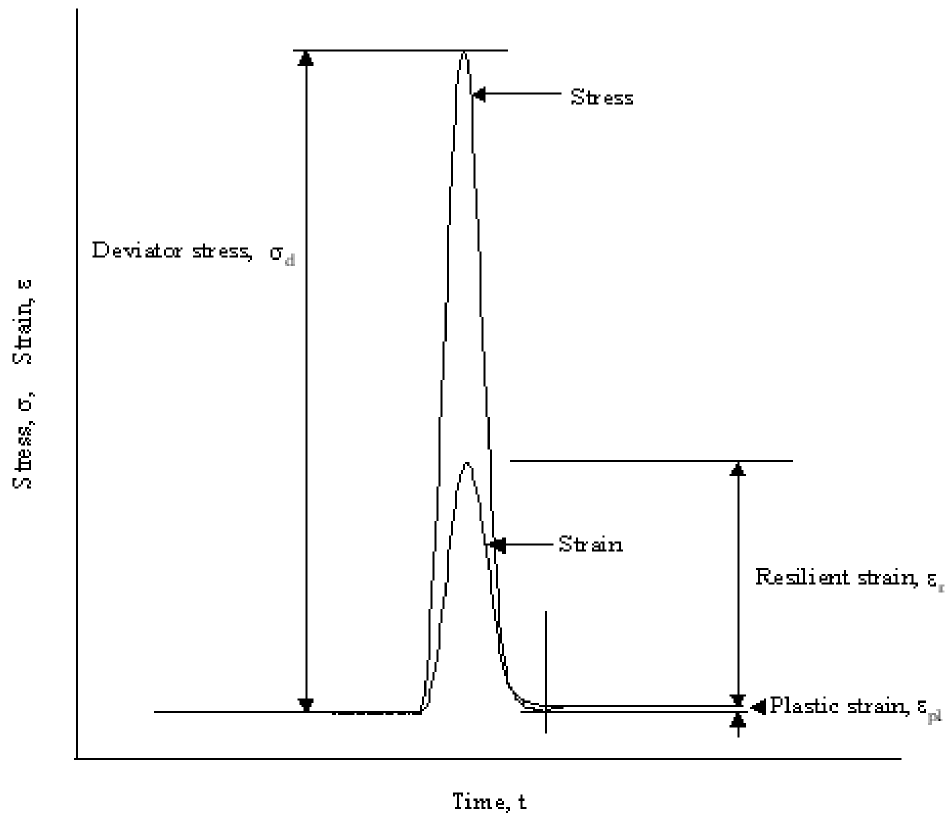


Figure 1
The definition of the resilient modulus in a repeated loading triaxial test

Results obtained from this software are not repeatable and appear to be affected by factors such as the testing load, the relative stiffness between layers, and environmental conditions.

These limitations in laboratory and in situ NDT methods signify the need to develop a more realistic, reliable, and economical in situ method for determining the resilient properties of subgrade soils. The in situ method should be able to save a significant amount of time and money, which would have been spent on sampling and laboratory testing. Several types of in situ testing equipment have been used in geo-technical investigations for the past two decades. The cone penetration test (CPT) has been recognized as one of the most widely used in situ tests. In the U.S., cone penetration testing has gained rapid popularity in the last decade and is currently replacing the traditional standard penetration test (SPT) *[6]*. Cone penetration testing is an in situ method used for classification and interpretation of engineering properties of soils in the field of geo-technical engineering. The cone penetration test consists of advancing a cylindrical rod with a conical tip into the soil and measuring the forces required to push this rod. There are two forces or resistances measured during the CPT: the tip resistance (q_c), the soil resistance to advance the cone tip, and the friction resistance (f_s), the sleeve friction developed between the soil and the sleeve of the cone because of the advancement of the cone tip. Friction ratio (R_f) is defined as the ratio between the friction resistance and tip resistance and is expressed in a percentage. These measured resistances are used to identify soils and determine their properties.

There are significant developments in cone penetration testing. Different sizes and shapes of cones are used *[7]*. As a result of these investigations, the ASTM Standard D3441 recommended using a standard cone with an apex angle of 60 degrees, tip area of 10 cm^2, and a sleeve area of 150 cm^2. Cones smaller than the standard cone are generally used where shallow depths need to be explored, such as pavement and subgrade explorations and are also used for finer soil classification of the strata. Larger cones are generally used to penetrate harder strata when the standard cone cannot be used.

The CPT parameters are used for the following applications in geo-technical engineering *[7]*: continuous soil stratification and identification, assessment of the undrained shear strength, stress history or over consolidation ratio (OCR), consolidation parameters and conductivity characteristics of cohesive soils, assessment of relative density, drained strength parameters and compressibility characteristics of cohesionless soils, evaluation of liquefaction potential of cohesionless soils, determination of pile foundation capacities, assessment of ground water pressures, and settlement calculations of footings in soils.

Applications of cone testing are spreading to other fields of engineering. Cone tests are used to identify contaminants in the ground in environmental engineering, determine seismic properties in earthquake engineering and for various applications in geological investigations of petroleum engineering. Applications of CPT in the field of pavement engineering, particularly related to subsoils, have also been attempted. Badu-Tweneboah et al. *[8]* conducted CPT tests on various highway pavements in Florida. They correlated the cone test results with M_r results from NDT methods. Inaccuracies and uncertainties involving NDT back calculation subroutines may affect the reliability of these correlations. In spite of this limitation, this study revealed the potential of CPT in determining resilient properties.

Another type of cone penetration testing equipment known as the dynamic cone penetrometer (DCPT) has been used by the researchers in this area. This equipment is portable and tests can be conducted by operators with minimal training. The test results are often shown by the number of blows required to penetrate a given distance, which is similar to that of SPT. The method is fast, economical, repeatable and reliable, and has been successfully used for developing correlations between these parameters, CBR, and unconfined compression strength. However, the test results cannot be accurately used for the quantitative analysis of soil properties *[6]*. Furthermore, the results are significantly influenced by soil type, size and shape of grains, relative density, degree of saturation, cementation, penetration rate, diameter of penetrometer, operational errors due to operational procedures, and site conditions.

The cone penetration test is among the popular in situ tests conducted by the Louisiana Transportation Research Center and the Louisiana Department of Transportation and Development due to the soft nature of most soil deposits in southern Louisiana. A variety of cone penetration test systems operate for LTRC and DOTD. The availability of these systems and the nature of soil deposits were among the encouraging factors to initiate a study for investigating the applicability of the cone penetration test in evaluating the resilient characteristics of subgrade soil.

Limitation

The cone penetration test parameters were used (in geo-technical literature) to determine the static strength and deformation properties of soils such as Young's modulus and shear modulus. The concern regarding the use of the cone penetration test method to determine the resilient modulus of subsoils is with respect to the differences in the testing modes used. The tip resistance and sleeve friction are obtained from the cone penetration test, which is considered a quasi-static test method, whereas the resilient modulus is a property obtained from a dynamic repeated load test. It is often assumed that test parameters obtained from different test backgrounds may not provide reasonable correlations with one another. However, this is not always the case.

Earlier studies *[6], [9]* showed the potential of the quasi-static CPT method in determining the low strain dynamic shear modulus and liquefaction of soil. The dynamic shear moduli and CPT parameters are less influenced by stress and strain history. In fact, these parameters are controlled by the same soil variables, which may have led to the development of better correlations between them. The resilient modulus is considered analogous and also related to the shear modulus. Therefore, the influence of stress and strain behavior on resilient modulus will be similar to that of shear modulus. Previous studies also indicated that the resilient property of subgrade soil is less dependent on stress and strain history *[4]*. The strain history influence is also expected to be insignificant in a nondestructive repeated load triaxial test. Furthermore, the cone penetration tests and repeated load resilient modulus tests were conducted on soil under identical environmental conditions. This implies that both test parameters were subjected to similar environmental variables such as density, moisture content, and geo-material fabric. In such conditions, the cone penetration test and resilient moduli parameters depend on the same soil variables. Therefore, it is reasonable to assume that a correlation is possible between cone penetration test and resilient moduli parameters. The current study provided a preliminary assessment of the validity of this assumption.

OBJECTIVES

The main objective of this study is to assess the applicability of the intrusion technology in evaluating the resilient characteristics of subgrade soils. To accomplish this objective, common soil types in Louisiana were selected for the field and laboratory investigations in order to:

1. Investigate the influence of soil type and soil characteristics on resilient behavior of undisturbed soil samples obtained from the site near the cone test locations. Laboratory resilient modulus tests were conducted on the soil samples to examine this aspect.

2. Develop a statistical correlation between cone resistance parameters, soil characteristics, and laboratory determined resilient modulus.

3. Validate this correlation by predicting the resilient modulus of subgrade soils, which were not included in the development of the model.

SCOPE

Field testing using the 2 cm^2 miniature friction cone penetrometer as well as the 15 cm^2 friction cone penetrometer were performed on eight soils which comprise a wide spectrum of Louisiana soils. These soils include cohesive soil (fine-grained) such as silty clay, heavy clay, and overconsolidated clay and cohesionless soil (coarse-grained) such as sand. Repeated load triaxial tests were conducted on undisturbed soil samples obtained from the sites next to the CPT soundings to evaluate their resilient characteristics. The results of the 2 cm^2 miniature friction cone penetrometer were used to develop the correlation between the resilient modulus and the CPT output. For the fine-grained soils (clay, silty clay), the results of the 15 cm^2 friction cone penetrometer were used to calibrate the miniature friction cone in cohesive soils.

METHODOLOGY

Field and laboratory testing programs were conducted on eight soils, which comprise common soil deposits in Louisiana. These soils are silty clay, heavy clay, fissured overconsolidated clay, and sand. The field tests carried out at the selected sites consisted of cone penetration testing and disturbed/undisturbed soil sampling. The laboratory testing program was conducted at the Engineering Materials Characterization Research Facility (EMCRF) at LTRC. Undisturbed and disturbed soil samples were subjected to different tests to determine their resilient modulus, physical properties, strength parameters, and compaction characteristics.

Description of the Investigated Soils

An arrangement with DOTD professionals was made to select the test sites. Seven test sites were identified which comprise common soils occurring in Louisiana. The selected sites vary from man-made embankment to subgrade soil, which is suitable for the subject of the current research. A brief description of the soils at the test sites is presented in table 1. As presented in table 1, six of the investigated soils are considered fine-grained soils and two are classified as coarse-grained soils. Characteristics and detailed descriptions of the investigated soils is covered in the section "Analysis of Results." Figure 2 shows a map of Louisiana with the locations of the test sites.

Field Testing Program

Equipment for Field Testing

Two cone penetration test systems were utilized to execute the field testing program: the Research Vehicle for Geotechnical In-situ Testing and Support (REVEGITS) and the Continuous Intrusion Miniature Cone Penetration Test (CIMCPT). Undisturbed soil sampling was conducted using the drilling rig of DOTD/materials division and REVEGITS. Figure 3 depicts a photograph for the cone penetration test systems utilized in the investigation.

Research Vehicle for Geo-technical In situ Testing and Support (REVEGITS). REVEGITS, which was developed with sponsorship of the National Science Foundation by Tumay *[10]*, is an in situ testing and support system mounted on 20-ton vehicle powered by a Caterpillar 210 HP diesel engine on a model G-744 6×6 chassis. REVEGITS has the

Table – 1
Properties of the investigated soils

Property	PRF- Silty clay	PRF-Heavy clay	I-10/ LA-42 Clay	LA-15 Clay	LA-1 Sand	LA-28 Sand	LA-89 Silty clay loam	Siegen Lane clay
Description	Embankmen t	Natural deposit	Natural deposit	Mississippi river levee	Embankment	Natural deposit	Natural deposit	Natural deposit
Passing sieve #200 (%)	93	98	90	98	5	30	91	98
Clay (%)	23	84	42	44	5	12	26	33
Silt (%)	70	14	48	54	0	18	65	65
Undrained shear strength, S_u (kPa)	NA	51.5	90.4	28.5	NA	NA	46.0	136.0
Soil classification (USCS)	CL-ML (Silty clay)	CH (Fat clay)	CH (Fat clay)	CH (Fat clay)	SP-SM (Poorly graded sand with silt)	SM (Silty sand)	CL (Lean clay)	CL (Lean clay)
Soil classification (AASHTO)	A-4 (Silty soil)	A-7-6 (Clayey soil)	A-7-6 (Clayey soil)	A-7-6 (Clayey soil)	A-3 (Fine sand)	A-2-4 (Silty sand)	A-6 (Silty clay loam)	A-6 (Silty clay)

NA: Not available

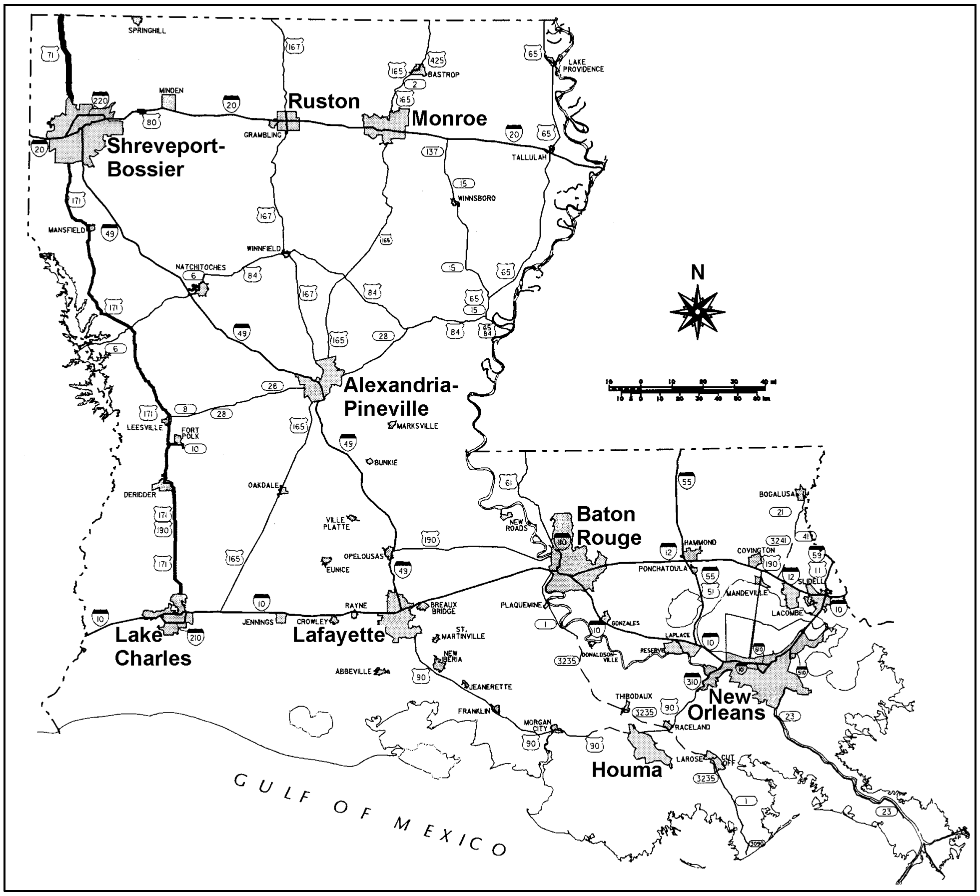


Figure 2
Locations of the investigated sites

Figure 3
The cone penetration test systems used in the study: the CIMCPT on the right and REVEGITS on the left.

capability to carry out cone penetration tests utilizing a variety of cones for different applications. A variety of cones are available including the 10 and 15 cm^2 friction cone penetrometers. The 15 cm^2 friction cone used in the current study is the cone developed by Fugro-McClelland Engineers B.V., the Netherlands, with a cone projected cross-sectional area of 15 cm^2, friction sleeve area of 200 cm^2, and a 60 degree cone apex angle. Figure 4 depicts a photograph for the hydraulic thrust system in addition to other units of REVEGITS.

Continuous Intrusion Miniature Cone Penetration Test (CIMCPT) System. The CIMCPT was developed at LTRC *[11]* for site characterization of subgrade soils, construction control of embankments, and assessment of the effectiveness of ground modification. The system is mounted on a 4-wheel drive, all terrain truck with a crew cab. The cone is attached to a coiled push rod, which allows a continuous penetration of the cone without segmental push rods. The coil is approximately 0.75 m in diameter and is mechanically straightened as the cone is pushed into the soil. The continuous push device of the CIMCPT system is shown in figure 5. The miniature cone was designed to measure soil properties at shallow depths (upper 5 to 10 m). The CIMCPT system and the miniature cone penetrometers were fabricated at SAGE Engineering, in Houston, Texas. The cone cross sectional area of the miniature cone penetrometer is 2 cm^2, the friction sleeve area is 40 cm^2, and the cone apex angle is 60 degrees.

Cone Penetrometers. Tumay and de Lima *[12]* have shown that the 10 and 15 cm^2 cone penetrometers give the same results. The 15 cm^2 cone penetrometer was used in this study as the reference cone since its results are *similar* with the results of the 10 cm^2 cone penetrometer. The 15 cm^2 friction cone has a projected cross-sectional area of 15 cm^2, friction sleeve area of 200 cm^2, and 60 a degree cone apex angle. A miniature cone penetrometer was also used in this study. The cone cross sectional area of the miniature cone penetrometer is 2 cm^2, the friction sleeve area is 40 cm^2, and the cone apex angle is 60 degrees. Figure 6 shows the size of the miniature cone penetrometer with respect to the 10 and 15 cm^2 cones.

Field Tests Procedure

The testing procedure described here was used at all sites. Field testing consisted of cone penetration tests (CPT) using the 15 cm^2 friction cone, miniature cone penetration tests (MCPT) using the 2 cm^2 miniature friction cone, and undisturbed soil sampling using

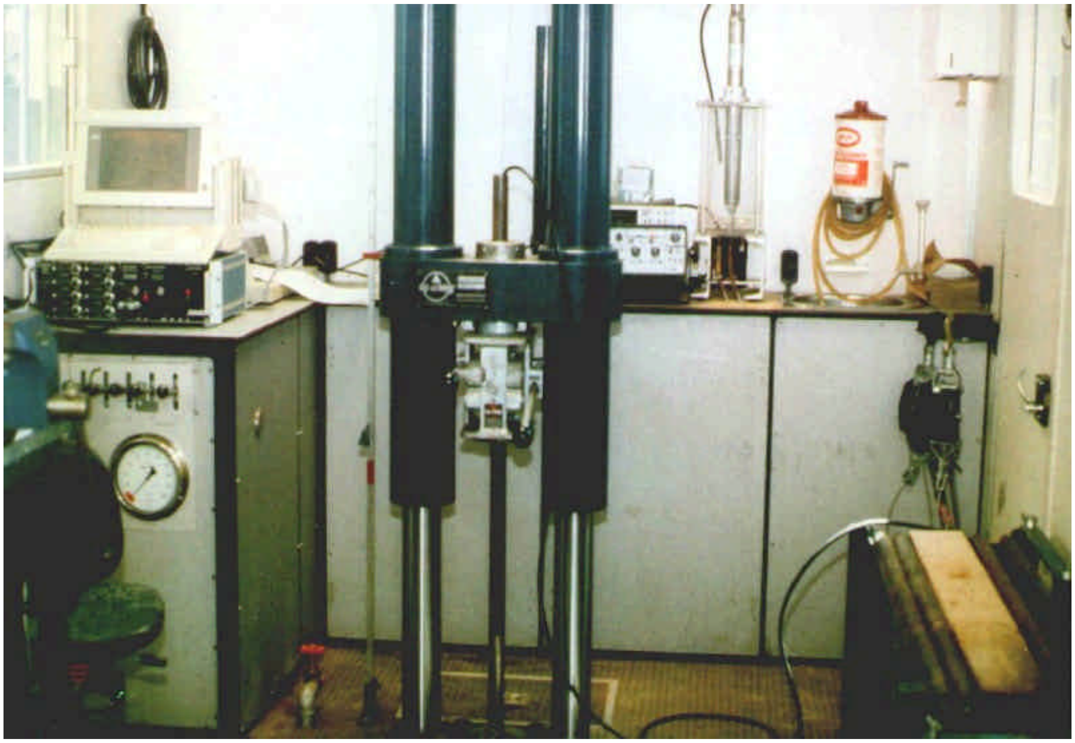


Figure 4
The interior of the 20-ton cone truck, REVEGITS.

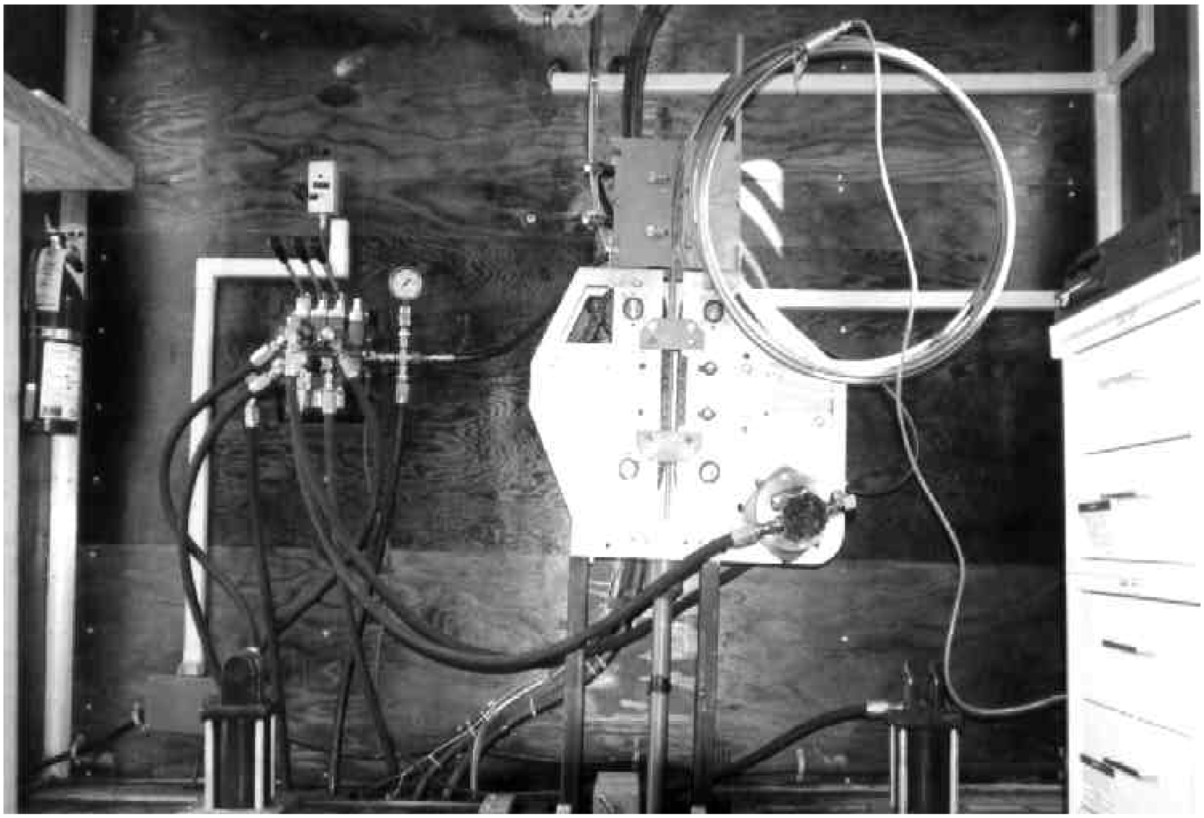


Figure 5
The cone penetration test systems used in the study, the caterpillar-type continuous push device of the CIMCPT system.

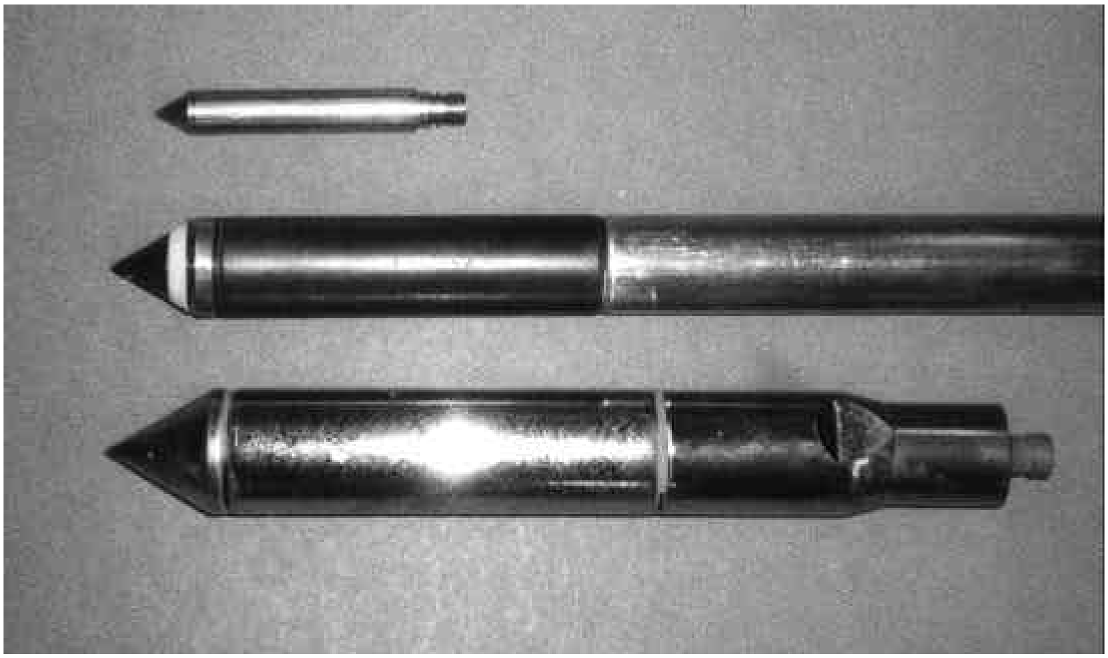


Figure 6
Miniature versus 10 and 15 cm^2 cone penetrometers.

thin-walled Shelby tubes. The CIMCPT system is mounted on a 1-ton truck compared to a 20-ton truck (REVEGITS) that hauls the cone penetration test system. Therefore, the continuous intrusion miniature cone penetration tests (MCPT) were conducted first to minimize the disturbance effects on the soil then monitor the MCPT test results. Finally the cone penetration tests were conducted, followed by Shelby tube sampling.

During the cone penetration tests (CPT and MCPT), the cone was advanced into the ground at a rate of 2 cm/sec. Continuous measurements of the tip resistance (q_c) and the sleeve friction (f_s) were obtained. Cone penetration tests were conducted around the borehole where the laboratory samples were obtained for resilient modulus determination. This was to ensure that the cone penetration soundings represent the soil tested in the laboratory. A cone penetration test plan was set to evaluate the reliability and the repeatability of the miniature cone penetration tests at each test site. A typical cone penetration test plan at the investigated sites is presented in figure 7.

Undisturbed and disturbed soil samples were obtained from the each test site, upto a depth of 2.0 m. Soil samples were extracted, sealed, and kept in a moisture controlled humidity room. Soil sampling and cone penetration tests were carried out in the same day to ensure similar in situ conditions.

Laboratory Testing Program

Undisturbed soil samples were trimmed and prepared for the laboratory resilient modulus testing. Repeated loading triaxial tests were conducted, using the MTS test system, to determine the resilient modulus of the investigated soil, following the AASHTO T-294 procedure *[13]*. Some of the investigated soils (e.g., the PRF-heavy clay) are very soft with a low unconfined compressive strength. These soil specimens could not be tested at high stress levels. In such cases, AASHTO T 294 specifies that the maximum deviator stress be limited to less than half of the unconfined compressive strength of the specimen.

Soil samples were also subjected to different laboratory tests to determine their physical properties and to provide complete material characterization. Atterberg limits, natural water content, in situ unit weight, grain size distribution, and specific gravity were among the tests

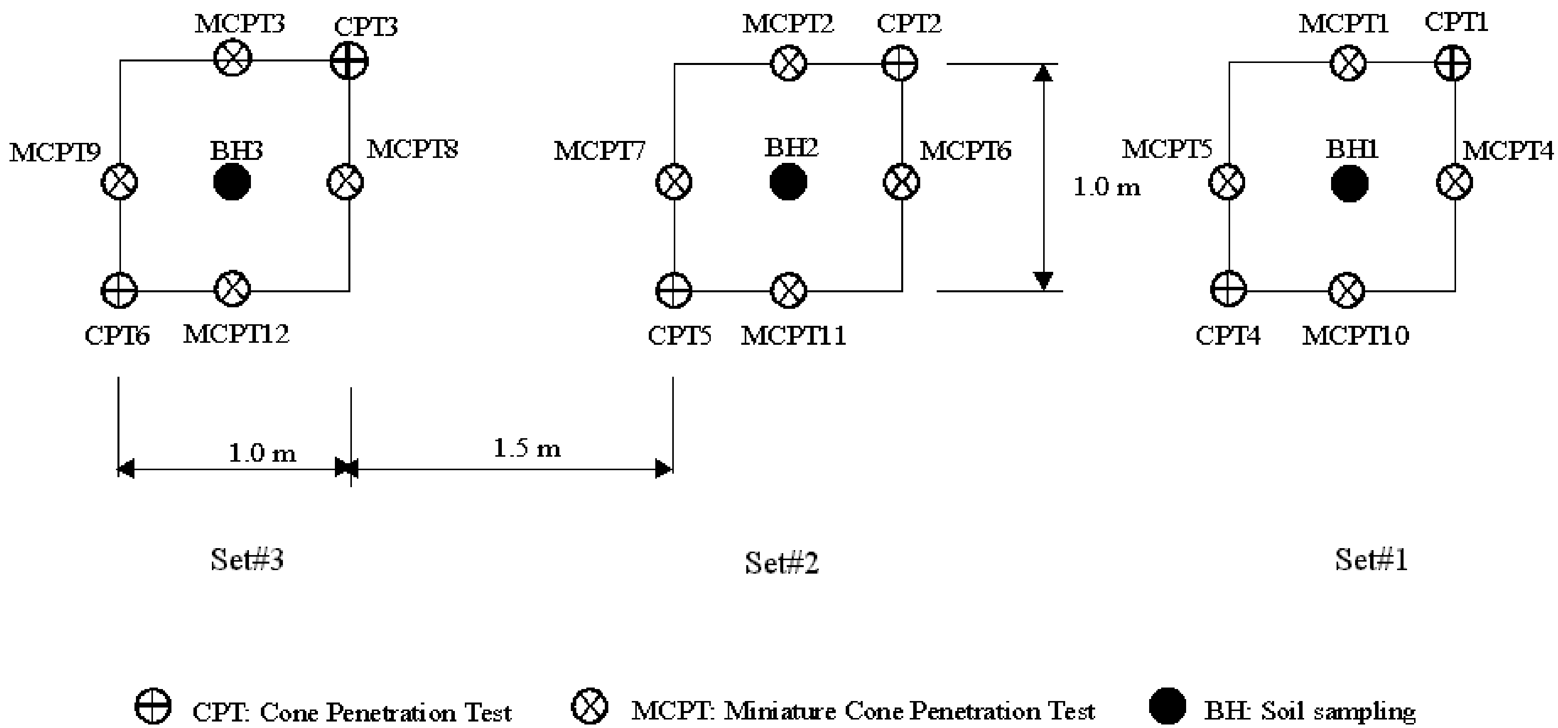


Figure 7
A typical field testing layout for the soil at a selected test site.

conducted. Some of the investigated soils (e.g., PRF-silty clay and PRF-heavy clay) have been subjected to many studies since they comprise the soil at the Louisiana Transportation Research Center/Pavement Research Facility experimentation site. Tests such as physical properties and compaction characteristics have been conducted on these soils *[4], [14]*. Description of this equipment is given below.

Equipment for Resilient Modulus Testing

An MTS model 810 closed loop servo-hydraulic material testing system is used to apply repeated loading. The major components of this system are the loading system, digital controller, and load unit control panel.

Loading System. The MTS loading system consists of a load frame and hydraulic actuator. The dynamic force and displacement ratings of this system are 100 kN (22 kips) and ±75 mm (±3 in.), respectively. Figure 8 presents a photograph of the loading system. The load frame is a free-standing, self-supporting, two column type unit with a moveable crosshead. The hydraulic actuator is supported on this crosshead. The load cell is attached to the end of the piston rod of the actuator and located inside the triaxial chamber.

Digital Controller. Figure 8 also shows the digital controller of the equipment. This acts as an interface between the computer and the rest of the system. The interface includes a machine control either in displacement or force mode that has sense conditioners and connections for external equipment. It also provides control of the hydraulic power supply and the hydraulic service manifold and provides 16 channels for analog inputs and outputs. The computer using the machine software, TESTSTAR, downloads the program code to the digital controller. This provides the digital controller with the code that controls the whole system.

The test equipment uses a closed loop control system for applying forces to the specimen. The digital controller acts as a stabilizing unit in this operation. Figure 9 shows the schematic of the closed loop control action. A simplified closed loop control consists of a controlling element, which in this case is the computer and digital controller, and a controlled element, which consists of a servo valve, hydraulic actuator, and test specimen.

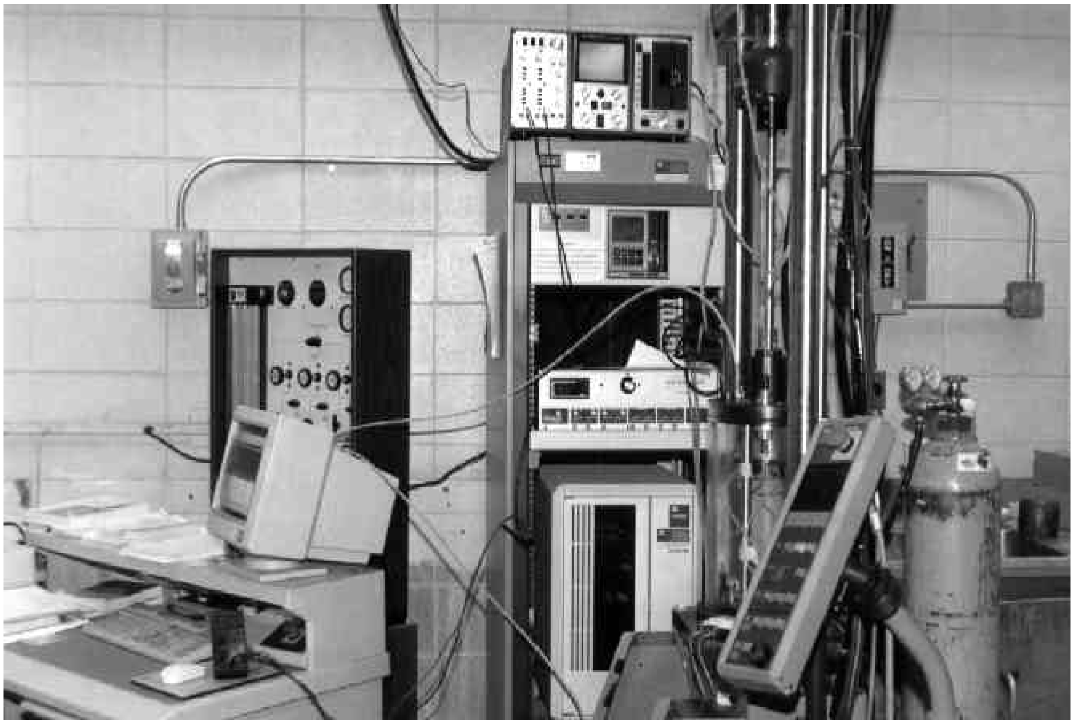


Figure 8
Photograph of the MTS equipment.

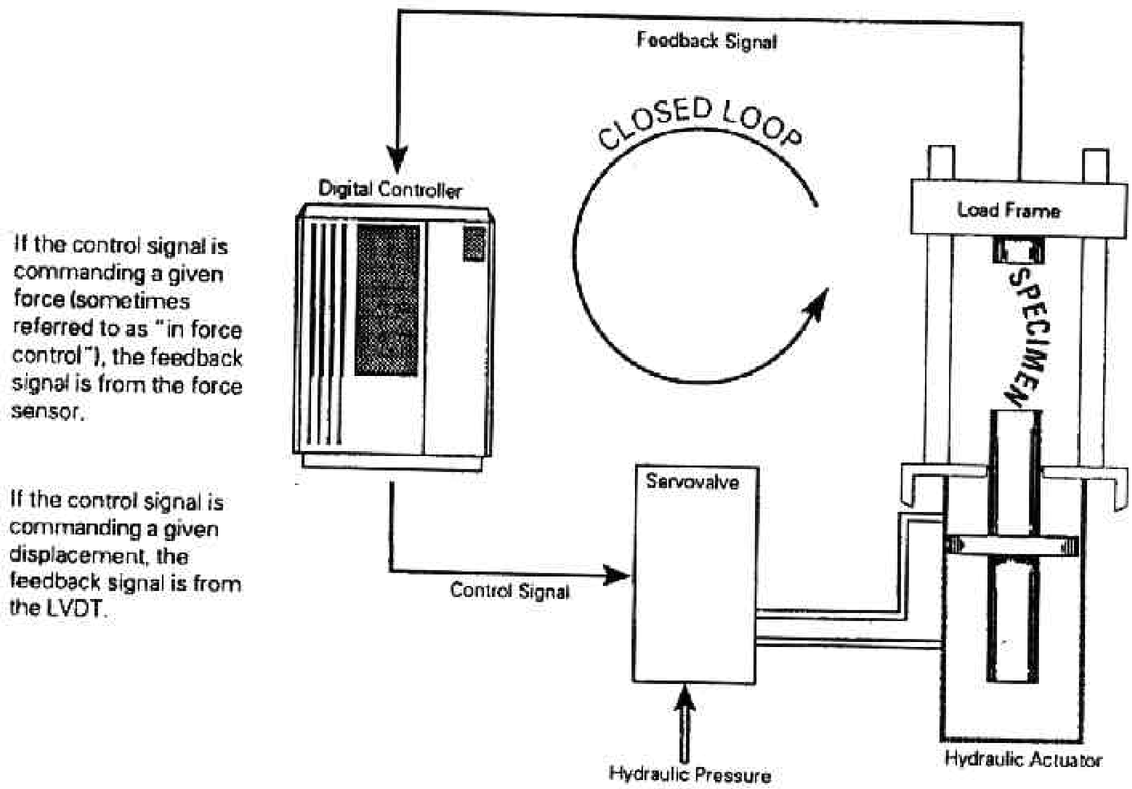


Figure 9
Schematic of the closed loop control system in the MTS testing device.

Load Unit Control Panel. This panel allows the users to control the load hydraulics while placing the specimen inside the triaxial cell. It has a 13 line by 40 characters long LCD display, which shows machine status and custom messages. A switch and control knob on this panel can be used to manage the actuator position for specimen installation. It also has switches to control program start, stop, hold, and resume operations. Other accessories used in the experimental program are listed below.

Triaxial Cell. The plexiglas triaxial cell, manufactured by research engineering, is 203 mm (8 in.) in diameter and 330 mm (13 in.) in height. This cell has features that allow in-cell axial displacement measurements and can accommodate samples of 71.1 mm (2.8 in.) in diameter. Confining pressures of up to 700 kPa (98 psi) can be applied in this cell. Compressed air is used as the medium since it is easy to work with and requires no special insulation for in-cell measurement devices and other electrical connectors.

Pressure Control Panel. The control or pressure panel is used in applying the confining pressures to the specimen. Pressure regulators on the panel are used for this purpose. The minimum pressure that can be applied with this system is 0.35 kPa (0.05 psi). This control panel also has a venturi type pressure pump, which will provide suction pressures. Figure 10 shows a photograph of the triaxial cell and control panel.

LVDTs and Load Cell. The measurement system has two diametrically placed internal LVDTs. The LVDTs of this system have a full scale stroke of ± 6.35 mm (± 0.25 in.) with a non-linearity of ± 0.0158 mm (±0.000625 in.). An internal load cell with a capacity of 1.36 kN (300 lbs) is used. This allows the researchers to apply even small loads without much disturbance.

Data Acquisition and Equipment Control. The data acquisition system plays a major role in determining the resilient modulus. Since the test is of a repeated loading type, an accurate and faster sampling data acquisition system is required to capture material response. The data acquisition system consists of a signal conditioner, data acquisition board, and software for equipment control and data analysis.

Signal Conditioner. A signal conditioner provides excitation signals for the LVDTs and the pressure transducers while amplifying the low level output signals from these measurement devices to high level signals. High level signals can be carried long distances without

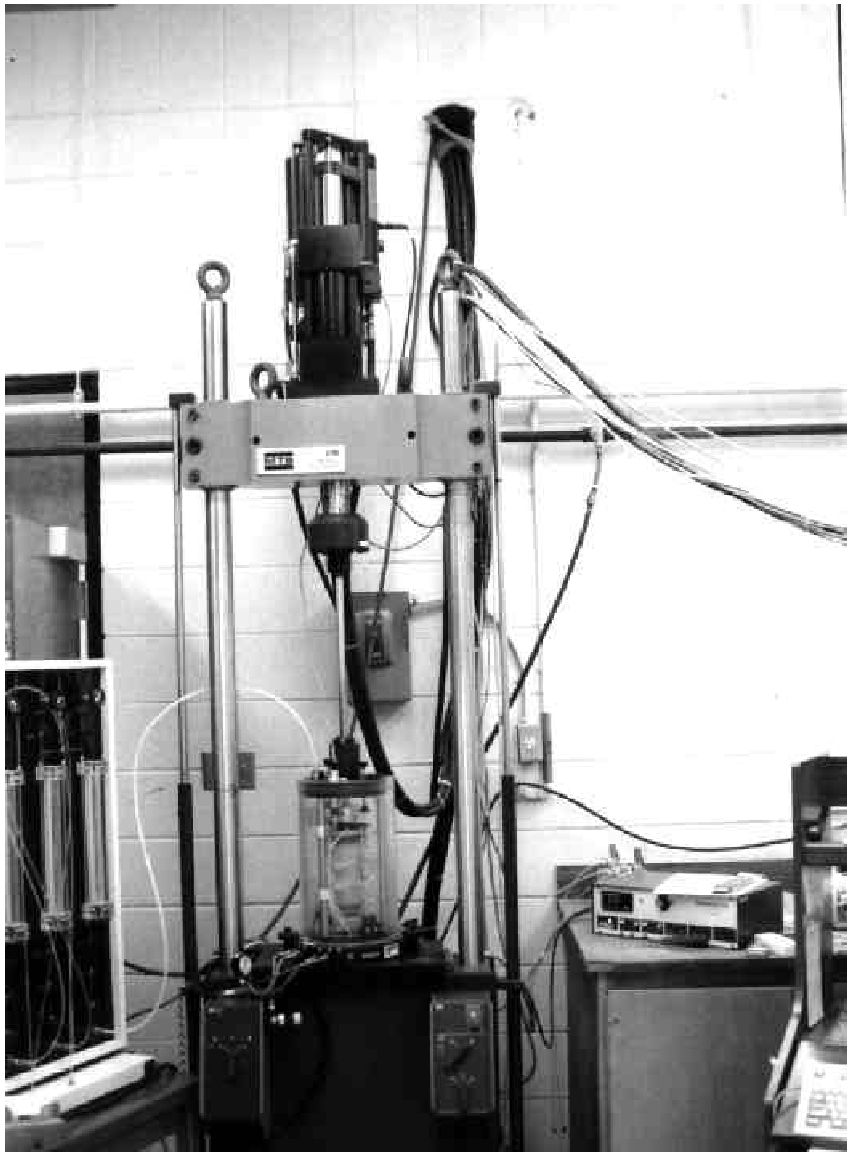


Figure 10
Photograph showing the triaxial cell and control panel.

causing much noise. Hence, a signal conditioner is kept close to the testing equipment. A ± 5 volt range is used for all sensors. The loads are monitored from the MTS system and have a fixed output level of ± 10 Volts for ± 1.36 kN (300 lbs).

Data Acquisition Board. A computer is used, together with a 12-bit interfacing board from Metrabyte, to collect, store, and analyze the data. Servo valve, strain guages, and LVDT signals are interfaced to the Metrabyte board through the signal conditioner. Custom application software was developed using the drivers and routines supplied by this board. This board has adjustable gain settings, which can be used for achieving the required resolution. The minimum values, which can be read from this system for LVDTs, pressure transducer, and load cell, are 0.00309 mm (0.00012 in.), 0.00104 mm (0.000041 in.), 3.3 N (0.73 lb), and 0.35 kPa (0.05 psi), respectively.

Software. The TESTSTAR software, provided with the equipment, is used for data acquisition and equipment control. The testing templates or procedures are written using the features in TESTSTAR software. Templates are made separately for conditioning and testing phases of M_r tests. For accurate measurements of high frequency dynamic loading and micro deformations, a sampling rate of 500 Hz per signal was used.

Data analysis for each confining pressure and deviatoric stress is performed by scanning the test data and analyzing it to determine the peak loads, resilient or elastic, permanent deformations, and resilient modulus properties. The software also provides a mechanism for verifying the sample alignment. This is done by comparing the deformations of both LVDTs from the end and middle measurement systems. Any significant differences in the deformations suggest that the sample is not properly aligned.

Resilient Modulus Test Procedure

Resilient modulus tests were performed on undisturbed soil samples from the investigated sites, according to the AASHTO T 294-92 “Resilient Modulus of Unbound Granular Base/Subbase Materials and Subgrade Soils - SHRP Protocol P46.” The tests on soils were performed at the confining and deviator stress levels recommended in the AASHTO T-294-92. The soil samples were conditioned by applying 1,000 repetitions of a specified deviator stress at a certain confining pressure. Conditioning eliminates the effects of specimen disturbances from sampling, compaction, and specimen preparation procedures and minimizes the imperfect contacts between end platens and the specimen. The specimen is then subjected to different stress sequences. The stress sequence is selected to cover the expected in-service range that a pavement or subgrade material experiences because of traffic loading.

Physical Properties

Standard laboratory tests were conducted on undisturbed and disturbed soil samples to characterize the soil at the investigated sites. Particle size analysis, Atterberg limits, specific gravity, natural moisture content, organic content, standard compaction test, unconfined compression test, and consolidated undrained conventional triaxial compression (CU-CTC) test were among the tests conducted. Some of the investigated soils (e.g., PRF-silty clay and PRF-heavy clay) have been subjected to many studies since they comprise the soil at the Louisiana Transportation Research Center/Pavement Research Facility experimentation site. Tests such as strength properties and compaction characteristics for these soils were obtained from the previous investigations. Table 2 summarizes the standard tests conducted on the investigated soils.

Table 2
Tests conducted on the investigated soil

Standard Test methods	PRF-Silty clay	PRF-Heavy clay	I-10/ LA-42 Clay	LA-15 Clay	LA-1 Sand	LA-28 Sand	LA-89 Silty clay loam	Siegen Lane clay
DOTD TR 407-89 Mechanical analysis of soils	✓	✓	✓	✓	✓	✓	✓	✓
DOTD TR 403-92 Determination of moisture content	✓	✓	✓	✓	✓	✓	✓	✓
DOTD TR 413-71 Organic material in soil	✓	✓	✓	-	-	-	-	-
DOTD TR 428-67 Determining the Atterberg limits of soils	✓	✓	✓	✓	-	-	✓	✓
ASTM D854-92 Test method for specific gravity of soils	✓	✓	✓	✓	✓	✓	✓	✓
ASTM D4767-88 Test method for consolidated undrained triaxial compression test on cohesive soils	*	*	*	✓	-	-	✓	✓
DOTD TR 418-93 Moisture-density relationships (standard Proctor test)	*	*	✓	✓	✓	✓	-	✓
ASTM D2487-93 Test method for classification of soil for engineering purposes (unified soil classification system)	✓	✓	✓	✓	✓	✓	✓	✓
DOTD TR423-89 Classification of soil and soil-aggregate mixtures for highway construction purposes	✓	✓	✓	✓	✓	✓	✓	✓

Legend: ✓- test done, *- obtained from previous studies

DISCUSSION OF RESULTS

This section presents the results of the field and laboratory testing programs, analysis of these results, and critical evaluation of the test results. First, site characterization of the investigated soils is presented, followed by an analysis of the cone penetration test results. Second, the results and elaborated analyses are given for the repeated triaxial loading to evaluate the resilient modulus of the investigated soils. Third, statistical analysis is presented and an empirical model is proposed for predicting the resilient modulus from the cone penetration test results and basic soil properties. Finally, a critique for the model and discussion of the results are presented.

Characterization of the Investigated Soils

Site characterization and evaluation of basic soil properties is necessary to accomplish the current research project. Physical soil properties and identification of soils are the main variables, since both the resilient modulus and the cone penetration test results are highly dependent on these variables. Eight common Louisiana soils were identified at seven different test sites across Louisiana. These soils comprise man-made embankments as well as natural soil deposits.

LTRC/Pavement Research Facility

The first site selected was the LTRC/Pavement Research Facility test site (LTRC/PRF), in Port Allen, Louisiana. The LTRC/PRF is the experimentation test site of the Louisiana Transportation Research Center. The site is located on six acres of a natural soil deposit of heavy clay (CH) with 84 percent clay and 14 percent silt. This site also houses the Louisiana Accelerated Loading Facility. A 1.52 m thick embankment was constructed of silty clay (CL-ML) with 23 percent clay and 70 percent silt at the LTRC/PRF site to investigate the response of flexible pavement structures under accelerated loading conditions. The LTRC/PRF site was selected to perform the field investigation due to the availability of two different types of cohesive soils used in the flexible pavement structure. Also, the field and laboratory tests database for these soils were available. Figure 11 depicts a map for the LTRC/PRF site showing the field tests conducted.

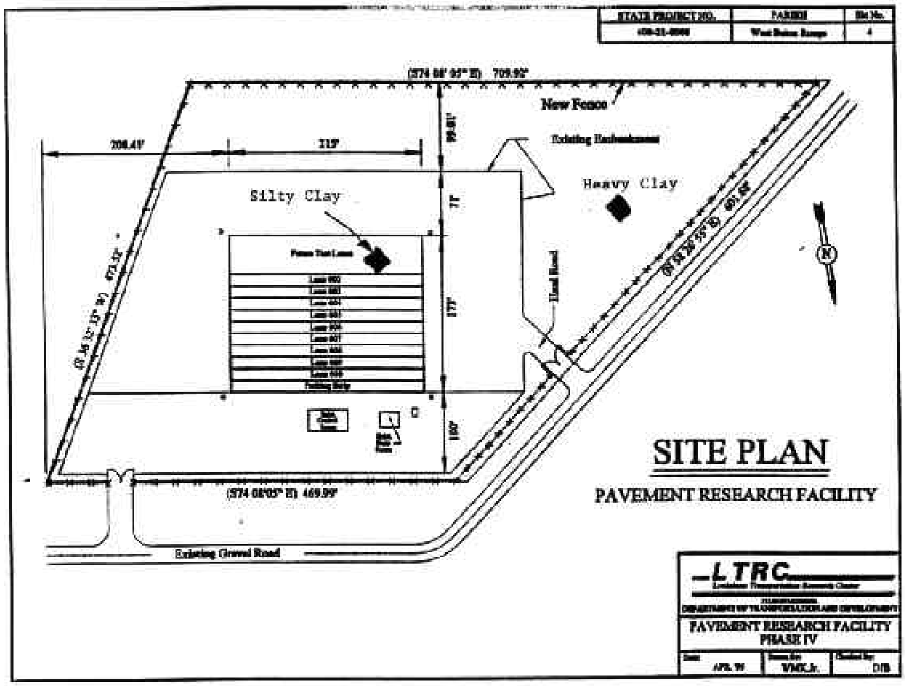


Figure 11

Location of the field test at the LTRC/PRF experimentation site, Port Allen, LA.

PRF-Silty Clay. The PRF-silty clay is a soil composed of 23 percent clay and 70 percent silt constructed in a 1.52 m thick embankment at the LTRC/PRF site. The main purpose was to investigate the response of flexible pavement structures under accelerated loading conditions. The embankment consists of a 12 in. top layer of limestone base with a moisture content of 4.2 percent and an in situ unit weight of 22.1 kN/m^3. The underlying layer is the 1.52 m silty clay layer with a 93 percent passing #200 sieve and 4.7 percent organic content. The PRF-silty clay is classified as CL-ML (silty clay) according to USCS and A-4 (silty soil) according to the AASHTO classification system. A geo-textile fabric was installed prior to construction of the silty clay layer to separate the embankment from the subgrade soil, which is the PRF-heavy clay.

The compaction characteristics of the PRF-sily clay showed that the optimum moisture content of the soil (w_{opt}) is 16.5 percent and the corresponding maximum dry unit weight (γ_{dmax}) is 17.0 kN/m^3. The properties of the PRF-silty clay are summarized, together with the properties of the investigated soils, in table 3.

PRF-Heavy Clay. Laboratory tests on undisturbed soil samples showed that the PRF site consists of medium gray soft normally consolidated clay with traces of organic materials and iron oxide. The top soil layer, with an average depth of 0.5 m, is mainly soft clay mixed with organic materials and traces of roots. This layer is underlain by approximately a 6.0 m deep soft normally consolidated clay layer. The PRF clay consists of 2 percent sand, 14 percent silt, 84 percent clay and colloids. The water table level is located at the ground surface. The soil possesses high moisture content with an average of 51 percent (average LL = 93 percent and average PL = 27). The average unit weight of the soil is 17.16 kN/m^3. The PRF-clay is classified as CH (fat clay) using the USCS and A-7-6 (clay), according to the AASHTO classification system. In this report, this clay is called PRF-heavy clay.

Standard Proctor test, using the PRF-heavy clay, showed that the optimum moisture content of the soil (w_{opt}) is 31.4 percent and the corresponding maximum dry unit weight (γ_{dmax}) is 13.6 kN/m^3. Unconsolidated undrained triaxial (UU) tests were conducted to evaluate the undrained shear strength of the PRF-heavy clay. Test results showed that the average undrained shear strength is 51.5 kPa. The properties of the PRF-heavy clay are summarized, together with the properties of the investigated soils, in table 3.

Table 3
Properties of the investigated soils

Property	PRF-Silty clay	PRF-Heavy clay	I-10/ LA-42 Clay	LA-15 Clay	LA-1 Sand	LA-28 Sand	LA-89 Silty clay loam	Siegen Lane clay
Description	Embank-ment	Natural deposit	Natural deposit	Mississippi river levee	Embankment	Natural deposit	Natural deposit	Natural deposit
Passing sieve #200 (%)	93	98	90	98	5	30	91	98
Clay (%)	23	84	42	44	5	12	26	33
Silt (%)	70	14	48	54	0	18	65	65
Organic content (%)	4.7	9.2	8	NA	NA	NA	NA	NA
Liquid limit (LL) (%)	28	93	50	52	NA	NA	34	35
Plastic limit (PL) (%)	22	27	16	25	NA	NA	23	23
Plasticity index (PI)	6	66	34	27	NA	NA	11	12
Specific gravity	2.67	2.68	2.69	2.70	2.69	2.68	2.69	2.69
Angle of internal friction(deg.)	22.0	14.0	28.5	14.0	28-38*	28-38*	17.0	19.2
Optimum water content (w_{opt}) (%)	16.5	31.4	18.1	28.1	14.4	11.4	20.2	17.5
Maximum dry unit weight (γ_{dmax}) (kN/m^3)	17.0	13.6	16.8	15.1	16.3	18.3	16.5	17.0
Soil classification (USCS)	CL-ML (Silty clay)	CH (Fat clay)	CH (Fat clay)	CH (Fat clay)	SP-SM (Poorly graded sand with silt)	SM (Silty sand)	CL (Lean clay)	CL (Lean clay)
Soil classification (AASHTO)	A-4 (Silty soil)	A-7-6 (Clayey soil)	A-7-6 (Clayey soil)	A-7-6 (Clayey soil)	A-3 (Fine sand)	A-2-4 Silty sand	A-6 (Silty clay loam)	A-6 (Silty clay)

State Route LA-42 (Highland Road) @ I-10

The second site is located at the intersection of Interstate 10 and the State Route LA-42 (Highland Road), Baton Rouge, Louisiana. Field and laboratory test results for the upper 10 m of soil indicated that the soil at this site consists of a brownish gray fissured stiff overconsolidated clay of low moisture content. The clay layer is interbedded with a 1 m thick silty clay layer at 3.5 m depth. The soil at this site consists of 42 percent clay and 48 percent silt. The average organic content of the soil is 11 percent. The moisture content values of the soil vary between 22 and 45 percent and are generally close to the plastic limit. The soil is primarily classified as CH (fat clay) using USCS and A-7-6/A-7-5 (clayey soil) using the AASHTO classification system. The average clay unit weight is 19.1 kN/m^3. The soil properties described are generally consistent with previous field and laboratory investigations at the same site by DOTD *[14]* and Chen and Mayne *[15]*.

Standard Proctor test using this clay showed that the optimum moisture content of the soil (w_{opt}) is 18.1 percent and the corresponding maximum dry unit weight (γ_{dmax}) is 16.8 kN/m^3. The undrained shear strength (S_u) of the clay obtained from an unconfined compression test varies from 75 to 121 kPa with an average of 90.4 kPa. Chen and Mayne [15] conducted isotropically consolidated undrained triaxial compression tests (CICU) on this clay. They reported that S_u range was from 60 to 120 kPa and the angle of internal friction (ϕ') is 28.5^o. Properties of LA-42 clay are summarized in table 3.

State Route LA-15

The embankment of State Route LA-15 in Concordia Parish is located approximately 15 miles south of Vidalia. The embankment at this location is part of the Mississippi River levee. LTRC has conducted several field and laboratory tests at the failure site; therefore this site was selected in the current study. The soil considered in this study is located at the up slope of the roadway on the levee. The soil consists of 44 percent clay and 54 percent silt. The average Liquid Limit (*LL*) is 52 percent and the average Plasticity Index (*PI*) is 27. The soil is classified as CH (fat clay) using the USCS and A-7-6 (clayey soil) using the AASHTO classification system.

The optimum moisture content of the soil is 28.1 percent while the corresponding maximum dry unit weight is 15.1 kN/m^3. Properties of LA-15 clay are summarized in table 3.

State Route LA-1/Larose

An embankment, under construction to relocate the State Route LA-1 at Larose, was selected as a test site for the current study. The embankment consists of 95 percent sand and 5 percent fines (passing sieve #200). According to the USCS the soil classification is poorly graded sand with silt (SP-SM) and fine sand (A-3) using AASHTO soil classification.

Compaction characteristics of the soil showed that the optimum moisture content of the soil is 14.4 percent and the corresponding maximum dry unit weight is 16.3 kN/m^3. Shear strength parameters for Larose sand was estimated from Das *[16]*. The properties of Larose sand are presented in table 3.

State Route LA-28/ Simpson

This site is located near the intersection of the State Route LA-28 and Highway 465 near Simpson, Vernon Parish. The site is used by the DOTD as a borrow pit for construction of a roadway embankment. The soil at this site consists of 60 percent sand, 18 percent silt, and 12 percent clay. Consequently, the soil is classified silty sand (SM) according USCS and silty sand (A-2-4) according to the AASHTO soil classification system.

Standard Proctor test using this soil showed that the optimum moisture content of the soil is 11.4 percent and the corresponding maximum dry unit weight is 18.3 kN/m^3.

State Route LA-89/New Iberia

An embankment located on State Route LA-89, New Iberia was selected as a test site. The embankment consists of lime treated recycled soil-cement base. The soil considered in this study is the subgrade soil (silty clay loam), which consists of 26 percent clay and 65 percent silt. The USCS classification for this soil is lean clay (CL) and AASHTO classification is silty clay loam (A-6).

Siegen Lane/Baton Rouge

The test site is located at the intersection of Siegen Lane and the Industriplex in Baton Rouge. The soil at this site consists of 33 percent clay and 65 percent silt. The soil is classified as lean clay (CL) according to the USCS and silty clay (A-6) according to the AASHTO classification. The average Liquid Limit of the soil is 35 percent and the average Plasticity Index is 12. Compaction characteristics of the soil showed that the optimum moisture content is 17.5 percent and the corresponding maximum dry unit weight is 17.0 kN/m^3.

Cone Penetration Tests

Since the CIMCPT is a newly developed system, a comprehensive database of the miniature cone penetration test results in soft Louisiana soils is still in the process of compilation.

Therefore it is essential to conduct further field testing programs in parallel with laboratory confirmation schemes using the miniature cone as well as the 10 and 15 cm^2 cones in different soil types. The test data will provide the means to calibrate the miniature friction cone with respect to the standard cone penetrometers. It will also be used to establish correlations between the miniature cone penetration test results and soil parameters determined in the laboratory, utilizing them for roadway design and construction control of highway embankments. The development of such a database will foster confidence in using the miniature friction cone for shallow-depth site characterization. Cone penetration tests were conducted according to the plans presented in table 4 to ensure the reliability and repeatability of the miniature cone penetration test data.

The cone penetration tests and the soil sampling plan at the LTRC/PRF site are presented in figure 12. A total of 15 cone penetration tests were conducted on the PRF-silty clay (12 MCPT and three CPT tests). The penetration tests were conducted in three sets; each set included four MCPT tests and one CPT test, as shown in figure 12a. The cone tip resistance and the sleeve friction were recorded up to a depth of 2.0 m, which is satisfactory for the purpose of investigating the resilient modulus of the 1.52 m deep silty clay embankment.

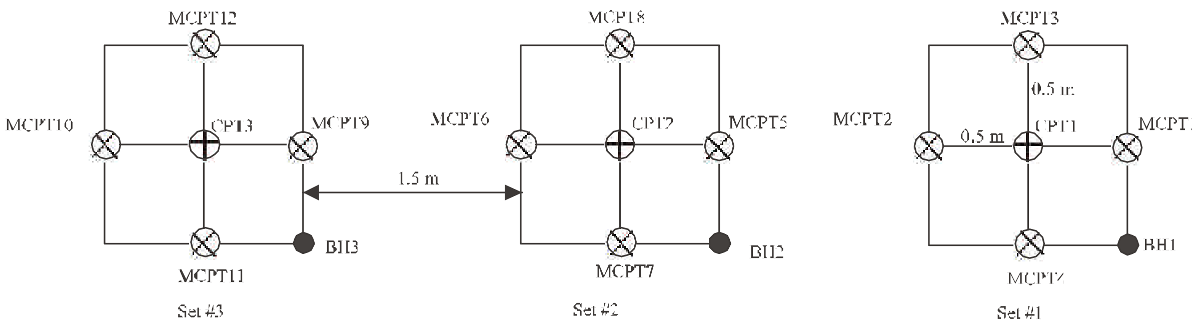
Eight MCPT and two CPT tests were conducted on the PRF-heavy clay, as shown in figure 12 b. Only five MCPT tests were considered in the analyses (MCPT 1, 3, 5, 6, and 8). Malfunction in the data acquisition system occurred while conducting MCPT 2, 4, and 7 tests. Both the MCPT and the CPT tests were conducted to a depth of about 10 m.

Due to the high resolution of the miniature friction cone penetrometer output and to the fact that measurements are recorded approximately every 4 mm of soil depth, a computer program was developed to analyze the miniature cone output. The program performs

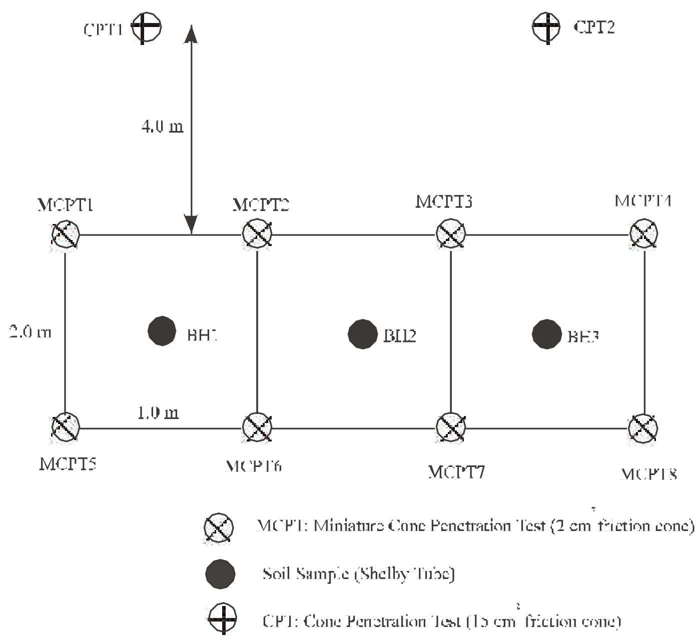
Table 4
Summary of the field testing program at the selected testing sites

Soil site	Field testing	Test set			
		1	2	3	4
PRF-Silty clay	CIMCPT: CPT: Soil sampling:	MCPT 1, 2, 3, 4 CPT 1 BH1	MCPT 5, 6, 7, 8 CPT 2 BH2	MCPT 9,10,11,12 CPT 3 BH3	- - -
PRF-Heavy clay	CIMCPT: CPT: Soil sampling:	MCPT 1, 5, 6 CPT 1 BH1	MCPT 3, 6 CPT 1, 2 BH2	MCPT 3, 8 CPT 2 BH3	- - -
I-10/LA-42 clay	CIMCPT: CPT: Soil sampling:	MCPT 5, 6 CPT 2, 4 BH1	MCPT 2, 6, 7, 11 CPT 2, 5 BH2	MCPT 7, 8 CPT 3, 5 BH3	MCPT 3, 8, 9, 12 CPT 3, 6 BH4
LA-15 clay	CIMCPT: CPT: Soil sampling:	MCPT 1, 4, 5, 10 CPT 1,4 BH1	MCPT 2, 6, 7, 11 CPT 2, 5 BH2	MCPT 3, 8, 9, 12 CPT 3, 6 BH3	- - -
LA-1 sand	CIMCPT: CPT: Soil sampling:	NA CPT 1, 2, 7, 8 BH1	NA CPT 3, 4, 9,10 BH2	NA CPT 5, 6, 11, 12 BH3	- - -
LA-28	CIMCPT: CPT: Soil sampling:	MCPT 1, 4, 5, 10 CPT 1, 4 BH1	MCPT 2, 6, 7,11 CPT 2, 5 BH2	MCPT 3, 8, 9, 12 CPT 3 , 6 BH3	- - -
LA-89 clay	CIMCPT: CPT: Soil sampling:	MCPT 1, 2, 7, 8 CPT 1, 4 BH1	MCPT 3, 4, 9, 10 CPT 2, 5 BH2	MCPT 5, 6, 11, 12 CPT 3, 6 BH3	- - -
Siegen Lane clay	CIMCPT: CPT: Soil sampling:	MCPT 1, 4, 5, 10 CPT 1, 4 BH1	MCPT 2, 6, 7, 11 CPT 2, 5 BH2	MCPT 3, 8, 9, 12 CPT 3, 6 BH3	- - -

Legend: CIMCPT- Continuous Intrusion Miniature Cone Penetration Test, CPT- Cone Penetration Test, MCPT- Miniature Cone Penetration Test, BH- Bore Hole, NA- not available.



(a) Field tests on the PRF-silty clay.



(b) Field tests on the PRF-heavy clay.

Figure 12
Layout of the field tests at the PRF experimentation site.

statistical analyses on the CPT and the MCPT test results. For each MCPT test, the soil was divided into layers of identified thickness, where the tip resistance and the sleeve friction values were averaged along the layer depth. Each MCPT test set was analyzed separately where the mean, standard deviation, and coefficient of variation were determined. In this program, the Gaussian kernel was used to smooth the measured tip resistance and sleeve friction. The friction ratio was also calculated using the same program.

Figure 13a depicts the MCPT and CPT soundings on the PRF-silty clay for field test set 1. Considering the silty clay layer, the tip resistance and sleeve friction measured by the CIMCPT system at field test set 1 show a small variation among the four MCPT tests and reflect similar patterns. In addition, the CIMCPT soundings are consistent with the CPT soundings presented in figure 13a. Statistical analyses were also conducted to quantitatively evaluate the reliability of the CIMCPT soundings. The silty clay layer at the PRF site was divided into small layers of 20 mm, which is the layer thickness of the CPT field measurements, and 50 mm each. Then the tip resistance and the sleeve friction were averaged along these layers and the standard deviation and the coefficient of variation along the soil depth were determined. Considering the soil layer thickness of 50 mm in figure 13b, results indicate that the CIMCPT soundings are consistent. The coefficient of variation of the tip resistance measurements ranges between 0 and 34 percent and generally remains under 20 percent for most of the silty clay layer. Considering the soil variability from one spot to another and the high sensitivity of the CIMCPT system, these results are considered satisfactory. The analyses were also performed for the sleeve friction and the field test set 1, as shown in figure 13b. The analyses were conducted on each field test set at all sites. The results, as shown for PRF-silty clay field test set 1, are satisfactory, and the CIMCPT soundings are considered repeatable and consistent within each set and with the 15 cm^2 CPT.

Figure 14a depicts the results of three MCPT and two CPT tests conducted with the PRF-heavy clay up to about 10 m in depth. Examination of the figure indicates that the tip resistance and sleeve friction measurements, using the miniature friction cone penetrometer and the 15 cm^2 cone, are consistent. Considering the effects of soil variability and the differences between the two friction cones, the miniature friction cone soundings are considered reliable. Analysis of the MCPT test results on the heavy clay was conducted. The variation of the tip resistance with depth for MCPT test sets is shown in figure 14b. The coefficient of variation for the tip resistance ranges between 0 and 43 percent.

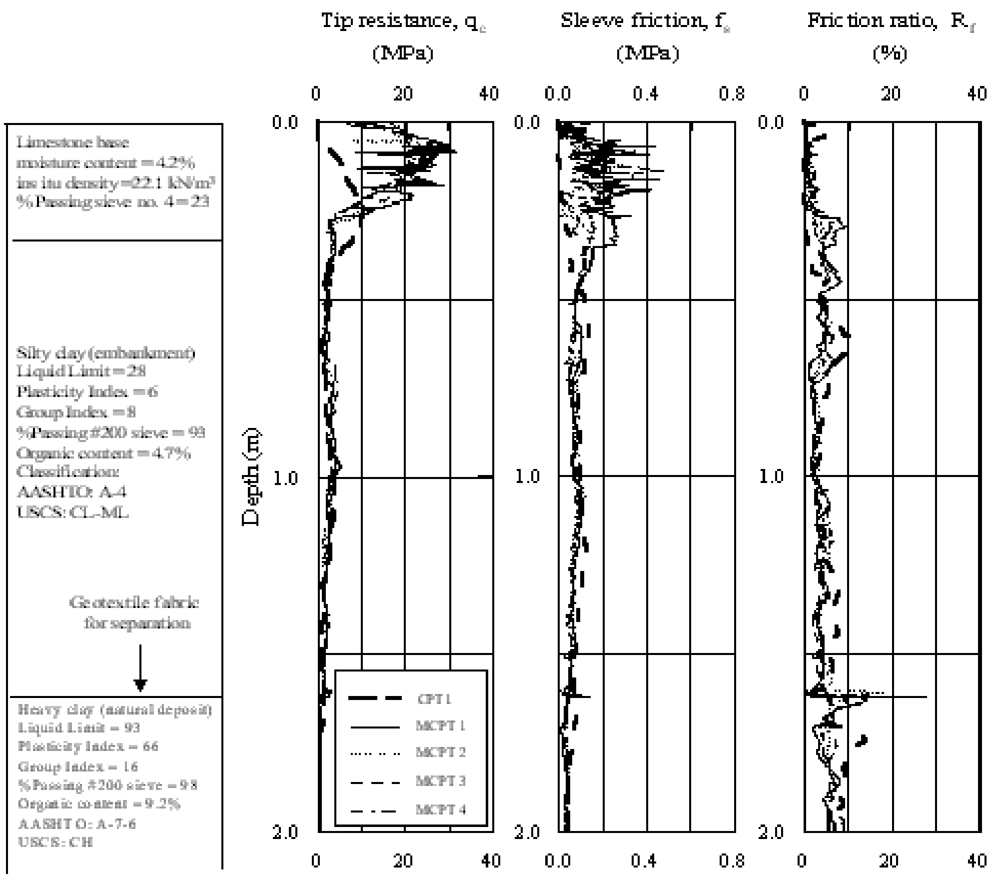


Figure 13:
(a) Cone penetration test results (MCPT and CPT soundings) on the silty clay

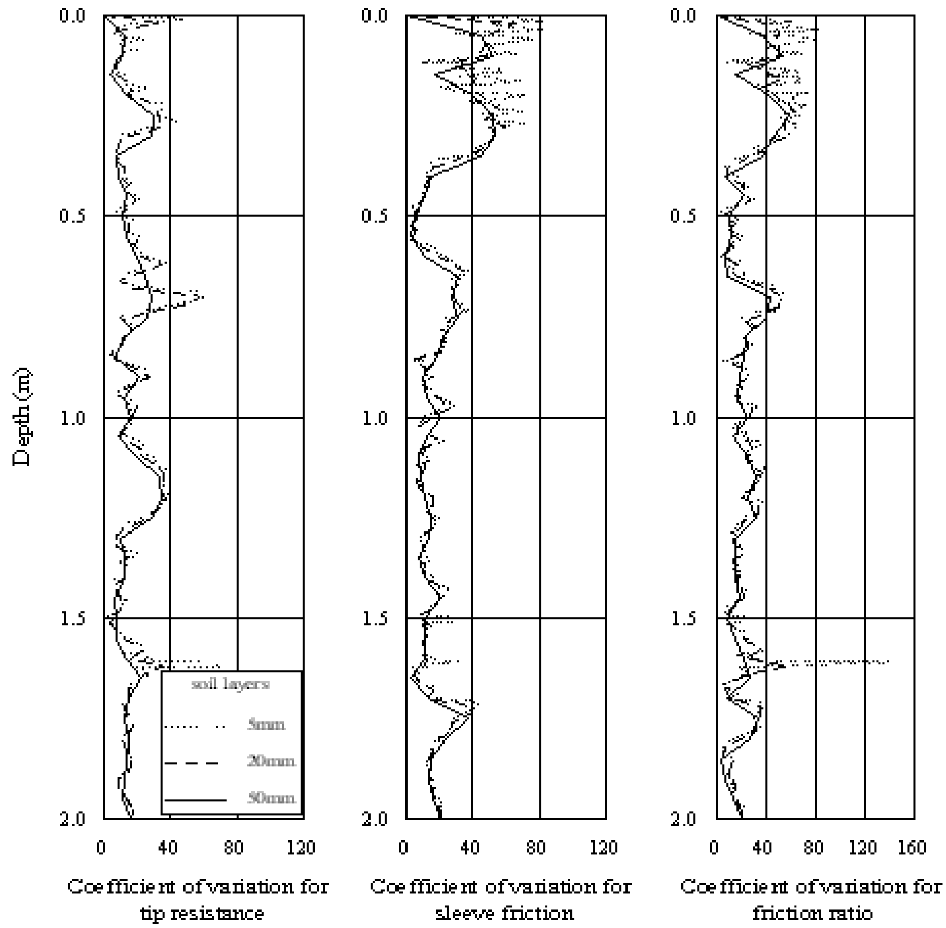


Figure 13:
(b) Statistical analyses of the MCPT soundings on silty clay (set 1).

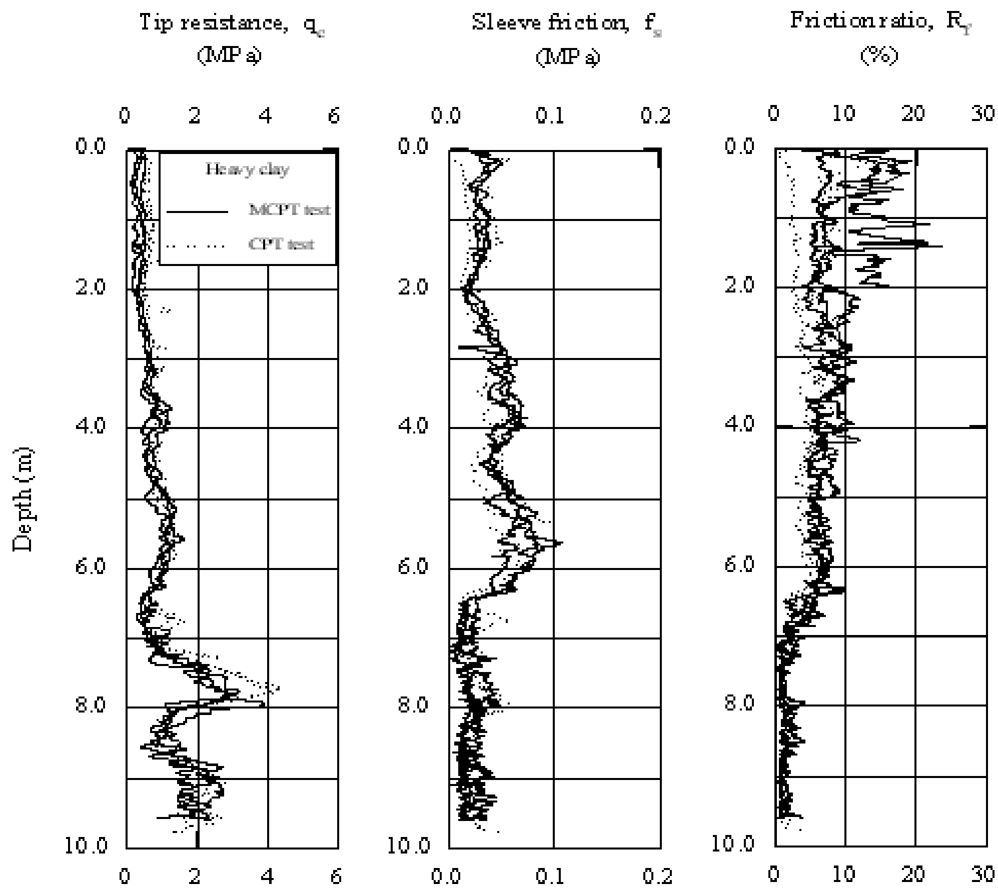


Figure 14:
(a) Cone penetration tests (MCPT and CPT soundings) on the heavy clay.

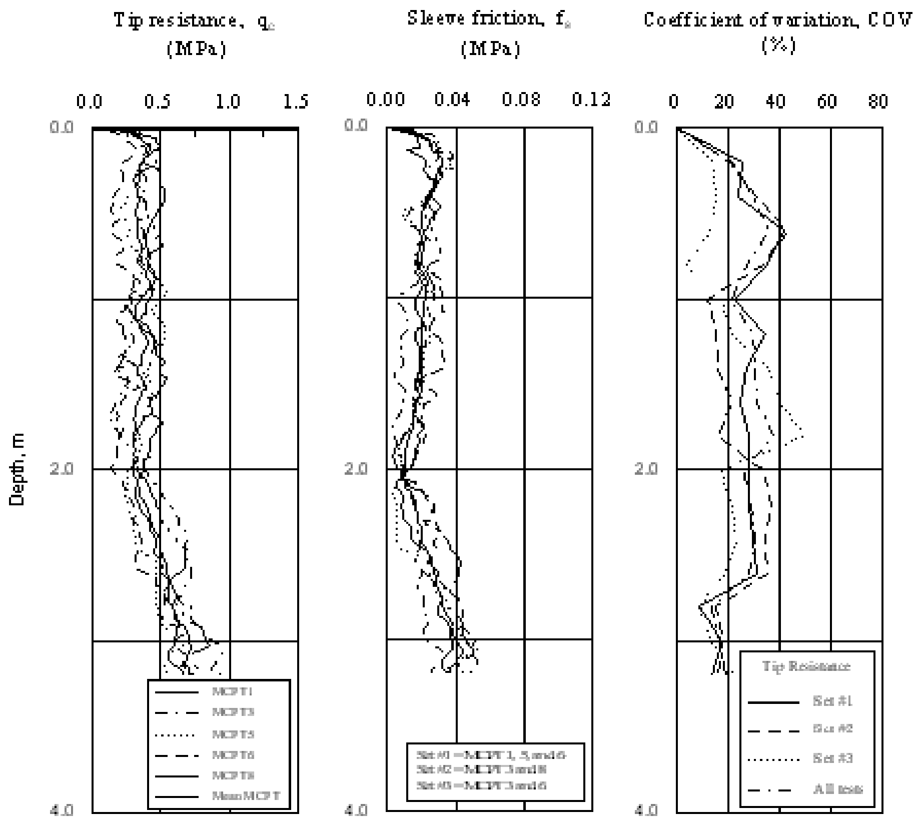


Figure 14:
(b) Variation of the MCPT soundings on the heavy clay.

The test plan for the LA-42 (Highland Road) site is shown in figure 15. The results of selected MCPT and CPT tests at the Highland Road site are depicted in figure 16. An inspection of figure 16 indicates that the MCPT and CPT soundings show very good compliance, are consistent within the same group, and reflect similar patterns. Statistical analyses were conducted to obtain a quantitative interpretation for both the MCPT and CPT test results. Figure 17a depicts a comparison of the means for 14 MCPT and six CPT soundings. The mean profiles for tip resistance (q_c), sleeve friction (f_s), and friction ratio (R_f) show consistent patterns between the MCPT and CPT tests. Figure 17b depicts the COV profiles for f_s and q_c from the MCPT and CPT tests. The tip resistance profiles for the top soil layer (0 to 1 m) and the silty clay layer (3.5 to 4.5 m) show large deviations for the MCPT and CPT tests results. For the remaining soil, the deviation is small and the different MCPT and CPT soundings show similar and consistent patterns. The average COV of the tip resistance is 18.5 percent for the 2 cm^2 miniature cone and 12.5 percent for the 15 cm^2 cone penetrometer. The average COV for the sleeve friction is 18.2 percent for the 2 cm^2 miniature cone and 16 percent for the 15 cm^2 cone penetrometer. The COV variation within the MCPT soundings is slightly higher than that for the CPT, particularly at the top soil layer and the silty clay layer, where thin layers and lenses/pockets of silt were observed. This is due to the high sensitivity of the miniature cone penetrometer and its capability to recognize thin soil layers. The pressure bulb generated in the soil, due to the miniature cone penetration, is smaller than that of the 15 cm^2 cone because of the difference in cone diameters. Therefore, the miniature cone will detect small (local) details/variations within the soil layer. These variations within the soil layers may not be recognized by the 15 cm^2 cone due to the globalization effect on the soil.

The soil at Larose test site consists of dense (compacted) fine sand. In such soil, sand particles will rearrange their orientation during penetration to let the state of compactness increase around the cone. The increase of the density of the sand, due to the penetration of the cone, results in an increase in the soil resistance to the cone penetration. Therefore a larger thrust is needed to push the cone into the ground. At Larose site, the maximum allowable thrust to push the miniature cone was reached with very little advancement of the cone. Therefore, no MCPT tests were conducted at this particular site.

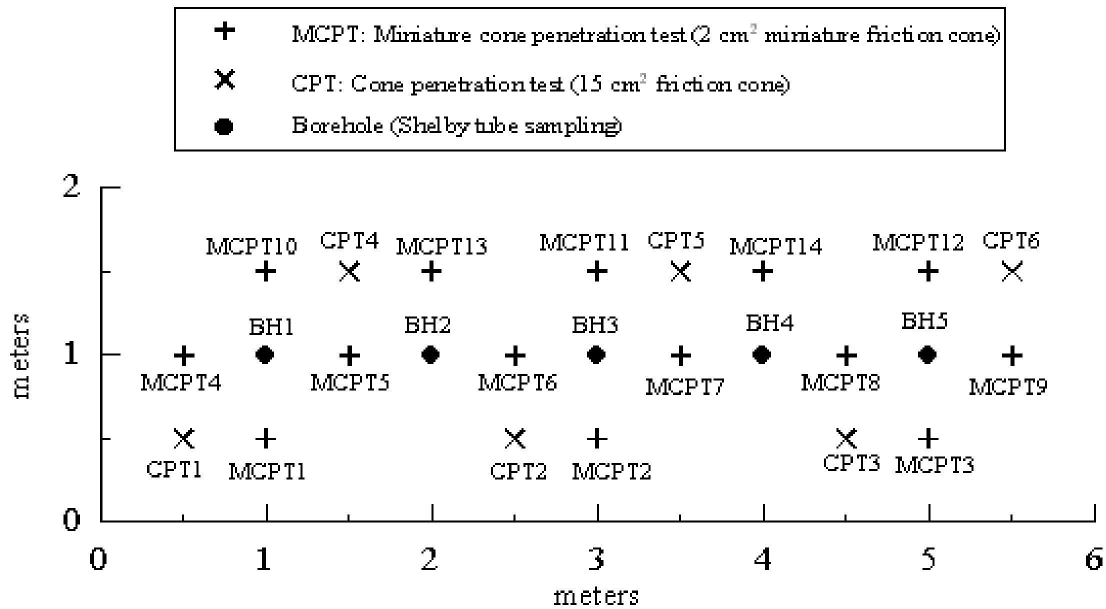


Figure 15
Field testing plan at the (a) I-10 @ Highland Road (LA State Route 42) test site

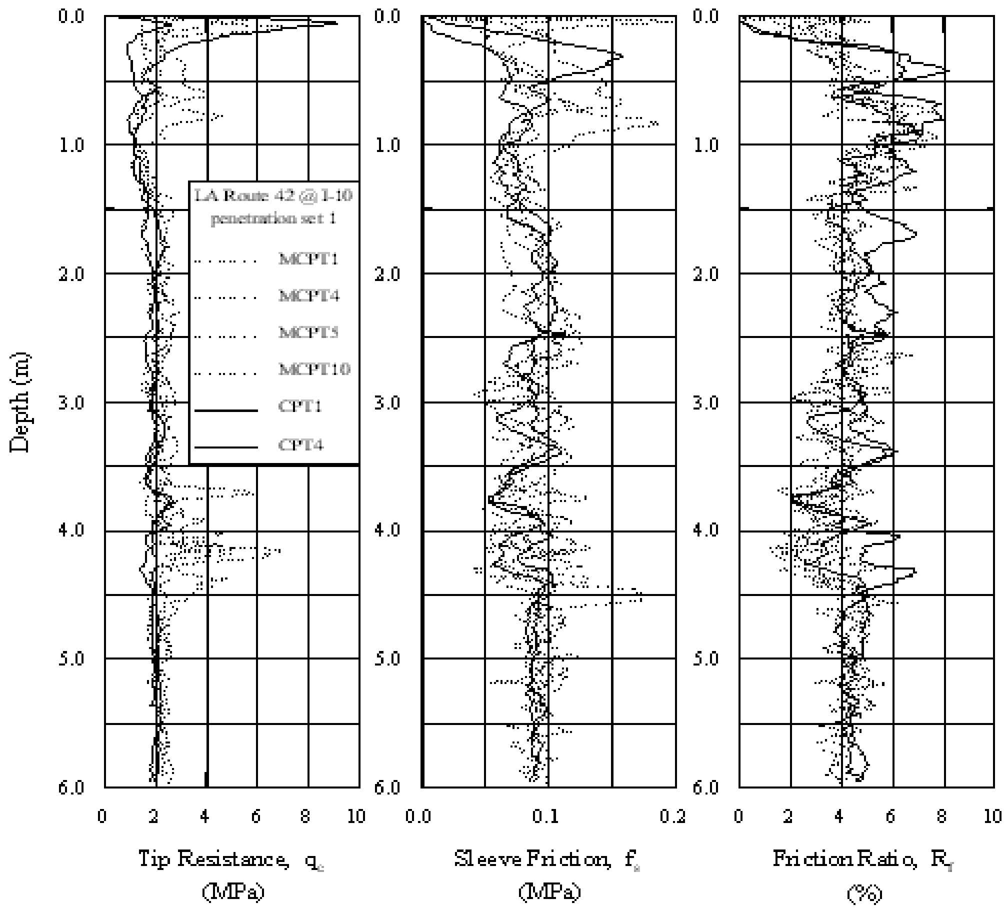


Figure 16
CPT and MCPT tests at the LA-42 test site (test set 1).

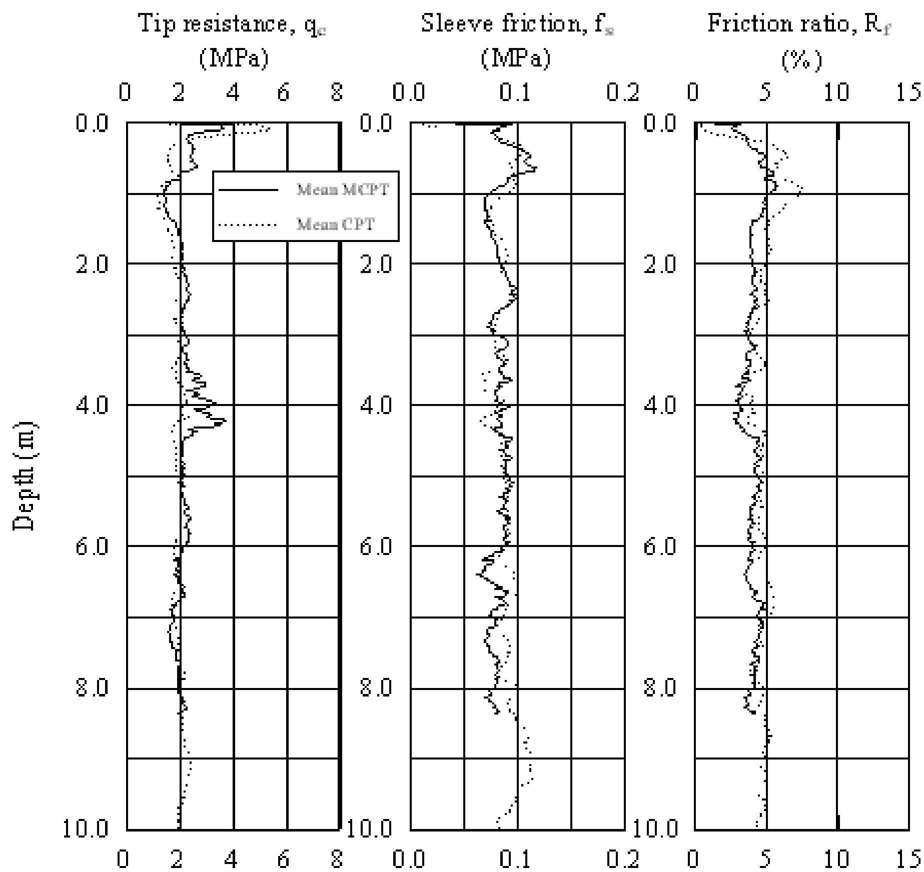


Figure 17
Statistical analysis for the MCPT and CPT soundings at Highland Road site.
(a) Comparison of means of MCPT and CPT tests on Highland Road clay.

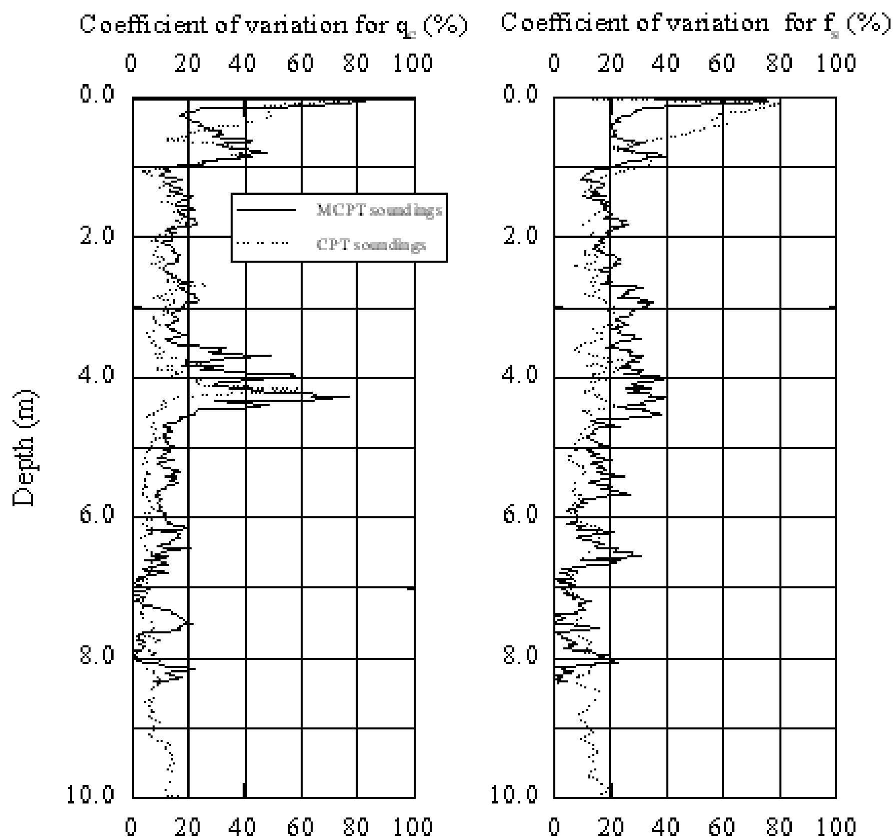


Figure 17
Statistical analysis for the MCPT and CPT soundings at Highland Road site.
(b) Comparison of COV profiles with depth for the MCPT and CPT tests.

Miniature CPT Versus Reference CPT

Analysis of the MCPT and CPT soundings at the different sites was conducted. It is evident from the output profiles that the miniature cone captured the same soil pattern obtained by the 15 cm^2 cone penetrometer. Based on the statistical analysis conducted on the fine-grained soil layers, the following relationships were established between the output of the miniature friction cone penetrometer and the 15 cm^2 friction cone penetrometer**.**

Cone tip resistance:
$$q_c(MCPT) = 1.11 q_c(CPT) \tag{2}$$

Sleeve friction:
$$f_s(MCPT) = 0.91 f_s(CPT) \tag{3}$$

These relations are consistent with the findings of de Lima and Tumay *[12]*, Tumay *[11]* and Titi et al. *[17]*, on the scale effects of cone penetrometers where smaller cones recorded higher tip resistance. The output of the miniature friction cone penetrometer is considered in the modeling part of this report without reference to the 15 cm^2 cone. The 15 cm^2 friction cone penetrometer is used only to calibrate and establish relationship with the miniature friction cone penetrometer.

Resilient Modulus

The results of the repeated load triaxial test on the LTRC/PRF soils (silty clay and heavy clay) are presented in tables 5 and 6.

The variations of the resilient modulus (M_r) with deviator stress at different confining pressures for the PRF-silty clay and PRF-heavy clay are shown in figures 18 and 19, respectively. The resilient modulus at a constant confining pressure decreases as the deviator stress increases, whereas, the resilient modulus at a constant deviator stress increases as the confining pressure increases. This reflects a typical behavior of the effect of stresses on the resilient modulus. Inspection of figure 18 indicates that the PRF-silty clay exhibited high resilient modulus values. The in situ moisture content of PRF-silty clay ranges from 20.8 to 25.4 percent and the unit weight varies between 19.9 and 20.8 kN/m3. These values are close to the optimum moisture content (w=16.5 percent) and the corresponding unit weight

Table 5
Results of the resilient modulus test for the PRF-silty clay.

σ_c (kPa)	σ_d (kPa)	BH1-top			σ_c (kPa)	σ_d (kPa)	BH2-top			σ_c (kPa)	σ_d (kPa)	BH3-top		
		M_r (MPa)	STD (MPa)	COV (%)			M_r (MPa)	STD (MPa)	COV (%)			M_r (MPa)	STD (MPa)	COV (%)
41.8	26.4	37.8	0.2	0.4	41.6	26.2				41.3	26.9	45.7	0.1	0.2
41.4	13.1	45.0	0.1	0.1	41.6	13.0	52.3	0.2	0.4	41.3	13.4	53.3	0.4	0.8
41.5	26.3	38.5	0.1	0.3	41.6	26.3	45.3	0.1	0.3	41.3	26.9	46.1	0.1	0.2
41.5	38.4	33.6	0.1	0.2	41.6	38.5	39.7	0.1	0.2	41.3	39.4	39.6	0.1	0.2
41.5	51.0	29.9	0.0	0.1	41.5	51.2	35.0	0.1	0.2	41.3	52.3	34.0	0.1	0.2
41.5	63.9	27.9	0.0	0.1	41.5	63.9	31.9	0.0	0.1	41.3	65.6	30.2	0.1	0.2
20.7	13.1	40.5	0.1	0.3	21.1	13.0	46.3	0.2	0.3	20.7	13.4	49.2	0.2	0.5
20.7	25.8	32.1	0.1	0.2	21.2	25.8	36.6	0.1	0.2	20.7	26.5	38.1	0.0	0.1
20.7	37.9	28.3	0.0	0.1	21.2	37.9	32.2	0.1	0.2	20.7	39.0	32.4	0.0	0.1
20.7	50.8	26.7	0.0	0.1	20.6	50.8	30.0	0.1	0.2	20.7	52.2	29.5	0.0	0.1
20.7	63.9	26.1	0.0	0.1	20.5	63.7	29.1	0.0	0.2	20.7	65.4	28.0	0.0	0.1
0.4	12.8	33.8	0.1	0.2	0.1	12.7	35.7	0.1	0.3	0.3	13.0	39.1	0.1	0.3
0.4	25.2	26.7	0.0	0.2	0.1	25.1	29.1	0.1	0.4	0.3	25.7	31.7	0.0	0.1
0.4	37.4	24.2	0.0	0.0	0.1	37.3	26.6	0.1	0.2	0.3	38.6	27.8	0.0	0.2
0.4	50.6	23.5	0.0	0.2	0.1	50.7	25.7	0.0	0.2	0.3	51.8	26.1	0.0	0.2
0.4	63.7	23.7	0.0	0.2	0.1	63.8	25.5	0.0	0.2	0.3	65.2	25.4	0.0	0.1
BH1-bottom					BH2-bottom					BH3-bottom				
41.3	26.1	43.7	0.2	0.5	41.1	27.2	69.3	0.3	0.4	41.3	25.9	59.2	0.0	0.4
41.3	13.0	50.4	0.2	0.4	41.1	13.6	81.4	0.6	0.7	41.3	13.1	67.7	0.1	0.4
41.3	26.2	44.1	0.1	0.3	41.1	27.2	70.2	0.2	0.3	41.3	25.5	59.9	0.1	0.3
41.4	38.2	37.7	0.1	0.3	41.1	40.0	59.8	0.1	0.2	41.3	37.7	52.9	0.1	0.2
41.3	50.8	32.5	0.1	0.2	41.1	53.2	50.9	0.1	0.1	41.3	50.3	46.9	0.1	0.2
41.3	63.5	30.1	0.0	0.1	41.1	66.3	45.1	0.0	0.1	41.3	62.7	42.7	0.0	0.1
20.6	12.9	42.9	0.2	0.4	20.9	13.6	75.2	0.6	0.1	20.7	12.5	55.5	0.3	0.5
20.6	25.6	34.1	0.1	0.2	20.8	27.1	57.3	0.2	0.3	20.7	25.5	45.7	0.1	0.1
20.7	37.7	30.0	0.0	0.1	20.9	39.7	48.5	0.1	0.2	20.7	37.2	40.9	0.1	0.2
20.6	50.7	28.4	0.1	0.2	20.9	53.0	44.3	0.1	0.2	20.7	49.6	38.5	0.1	0.2
20.6	63.7	27.9	0.1	0.2	20.9	66.4	42.0	0.0	0.2	20.7	62.7	37.6	0.1	0.1
0.3	12.7	35.1	0.3	0.1	0.3	13.6	62.3	0.1	0.1	0.0	12.1	38.6	0.3	0.1
0.3	25.1	28.4	0.2	0.1	0.3	27.1	47.1	0.3	0.2	0.0	24.1	33.6	0.2	0.1
0.3	37.3	25.6	0.2	0.1	0.3	39.7	40.7	0.2	0.1	0.0	36.4	31.7	0.1	0.0
0.3	50.5	24.9	0.1	0.0	0.3	53.0	38.0	0.2	0.1	0.0	48.9	31.1	0.1	0.0
0.3	63.5	25.1	0.1	0.0	0.3	66.4	37.0	0.2	0.1	0.0	61.3	31.2	0.1	0.0

Legend: σ_c =Confining stress, σ_d =Deviator stress, M_r=Resilient modulus, COV= Coefficient of variation, STD=Standard deviation

Table 6

Results of the resilient modulus test for the PRF-heavy clay.

σ_c (kPa)	σ_d (kPa)	BH1-top			σ_c (kPa)	σ_d (kPa)	BH2-top			σ_c (kPa)	σ_d (kPa)	BH3-top		
		M_r (MPa)	STD (MPa)	COV (%)			M_r (MPa)	STD (MPa)	COV (%)			M_r (MPa)	STD (MPa)	COV (%)
41.6	12.0	6.2	0.1	1.0	41.2	13.1	9.8	0.0	0.2	41.3	13.1	10.6	0.0	0.4
41.7	5.8	8.3	0.0	0.2	41.1	6.4	12.5	0.1	0.4	41.3	6.2	13.0	0.1	0.6
41.7	11.8	6.3	0.0	0.4	41.1	13.1	9.9	0.0	0.3	41.3	13.0	9.8	0.0	0.4
41.7	17.4	5.5	0.0	0.3	41.1	18.9	7.8	0.0	0.6	41.3	19.2	8.3	0.0	0.3
20.9	5.8	8.2	0.0	0.2	20.6	6.3	12.2	0.1	0.6	20.7	6.2	12.9	0.1	0.4
20.9	11.7	6.2	0.0	0.4	20.6	12.9	9.0	0.0	0.1	20.7	13.1	10.3	0.0	0.3
21.0	17.4	5.4	0.0	0.1	20.7	18.9	7.5	0.0	0.2	20.7	18.6	8.1	0.0	0.3
0.6	5.7	7.9	0.2	2.0	0.4	6.2	11.1	0.1	0.6	0.0	6.2	11.9	0.1	0.9
0.5	11.6	6.1	0.1	1.0	0.4	12.7	8.4	0.0	0.4	0.0	12.4	8.9	0.4	4.1
0.6	17.2	5.3	0.0	0.3	0.3	18.8	7.1	0.0	0.3	0.0	18.6	7.7	0.0	0.5
BH1-bottom					BH2-bottom					BH3-bottom				
41.4	12.1	7.0	0.1	2.0	41.5	13.7	15.5	0.0	0.3	41.3	13.8	15.2	0.0	0.2
41.5	5.9	8.6	0.2	2.0	41.5	6.5	18.5	0.1	0.4	41.3	6.6	18.0	0.1	0.8
41.5	12.3	7.1	0.1	1.0	41.5	13.7	15.6	0.0	0.2	41.3	13.8	15.3	0.0	0.2
41.4	17.2	5.8	0.0	0.3	41.5	20.3	13.1	0.1	0.4	41.3	20.3	12.8	0.0	0.1
20.9	5.9	8.8	0.1	1.0	20.5	6.5	17.5	0.1	0.4	20.7	6.3	16.7	0.0	0.2
20.9	12.1	6.8	0.1	1.0	20.5	13.6	14.5	0.0	0.1	20.7	13.8	13.9	0.0	0.3
20.9	17.3	5.8	0.0	0.2	20.5	20.2	12.5	0.0	0.3	20.7	20.0	12.1	0.0	0.1
0.6	5.8	8.2	0.0	0.4	0.5	6.4	16.0	0.2	1.0	0.0	6.2	15.2	0.0	0.3
0.6	11.9	6.5	0.0	0.4	0.5	13.4	13.2	0.0	0.2	0.0	13.1	12.6	0.0	0.3
0.6	17.2	11.7	0.0	0.2	0.4	20.0	11.5	0.0	0.3	0.0	20.0	10.9	0.0	0.4

Legend: σ_c =Confining stress, σ_d =Deviator stress, M_r=Resilient modulus, COV= Coefficient of variation, STD=Standard deviation

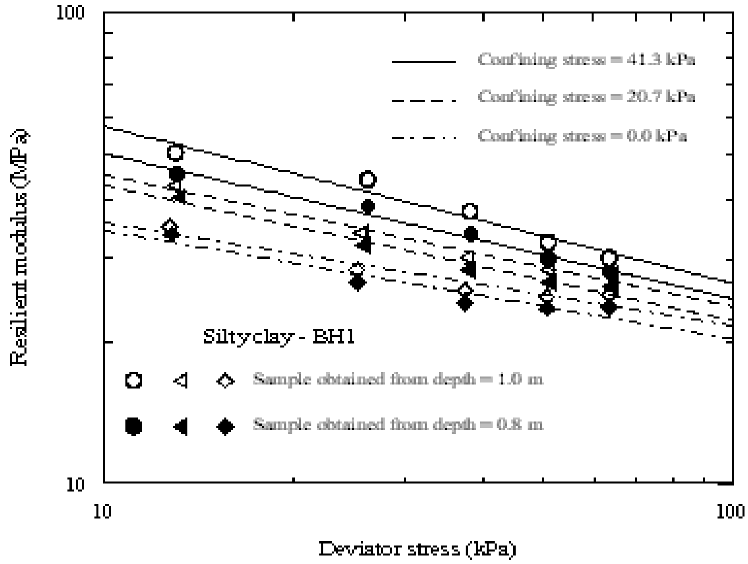


Figure 18
(a) Variation of the resilient modulus with deviator stress at different confining pressures for silty clay obtained from borehole BH1

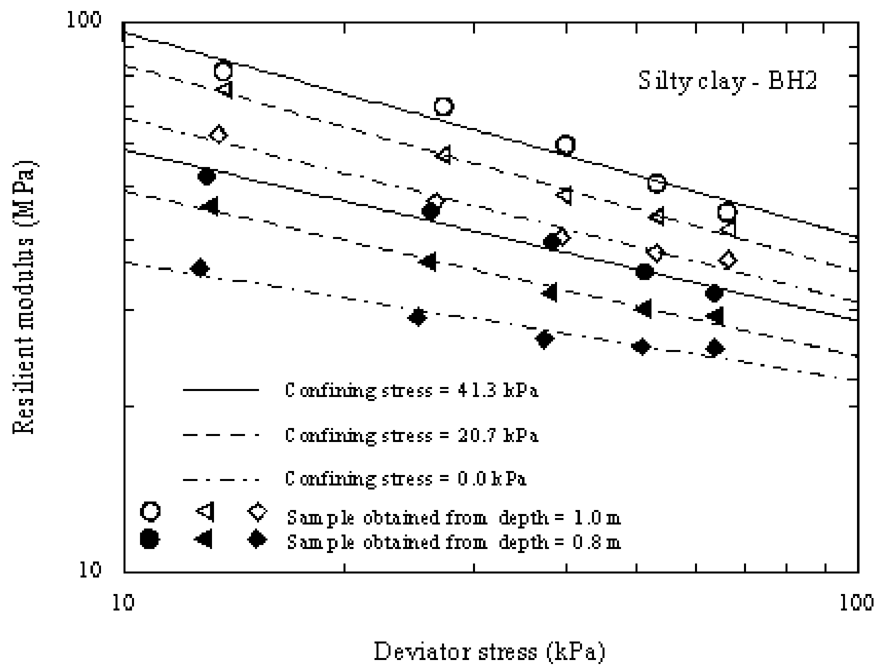


Figure 18
(b) Variation of the resilient modulus with deviator stress at different confining pressures for silty clay obtained from borehole BH2

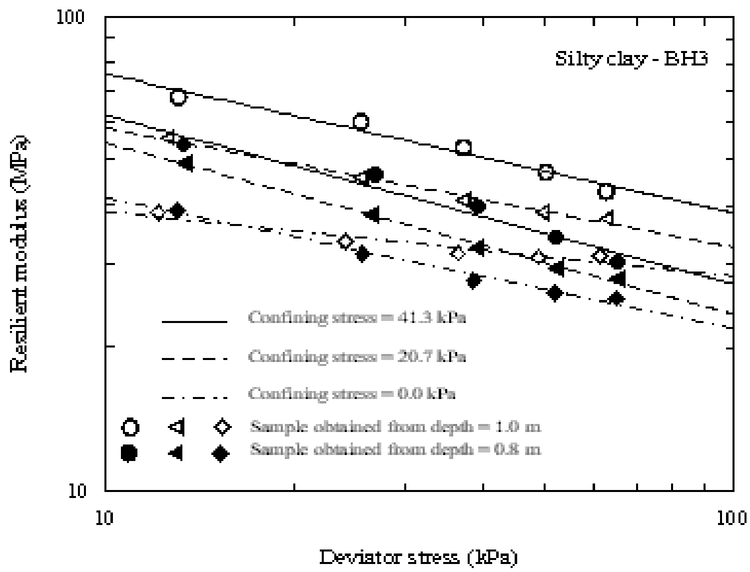


Figure 18
(c) Variation of the resilient modulus with deviator stress at different confining pressures for silty clay obtained from borehole BH3

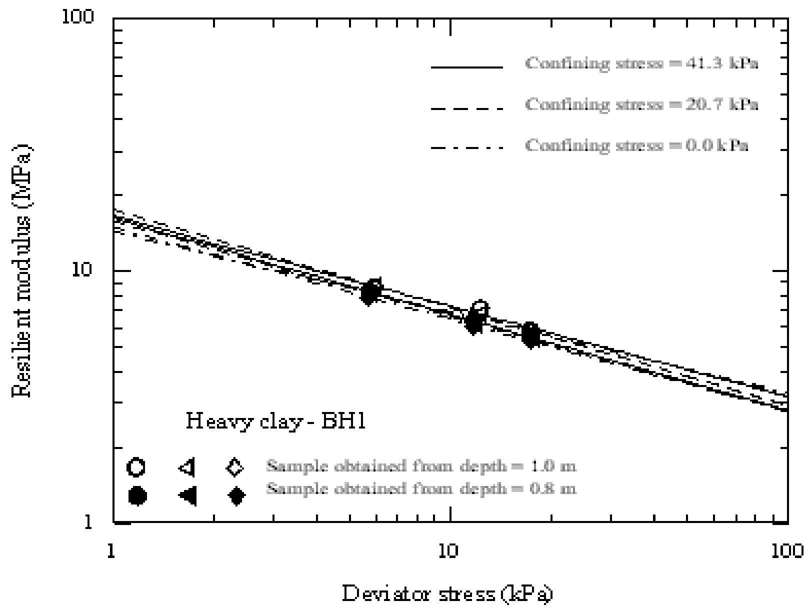


Figure 19
(a) Variation of the resilient modulus with deviator stress at different confining pressures for heavy clay obtained from borehole BH1

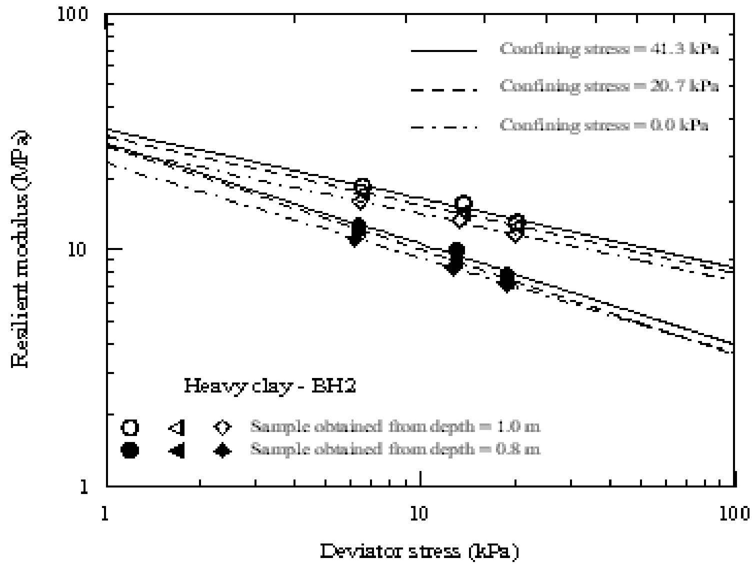


Figure 19
(b) Variation of the resilient modulus with deviator stress at different confining pressures for heavy clay obtained from borehole BH2

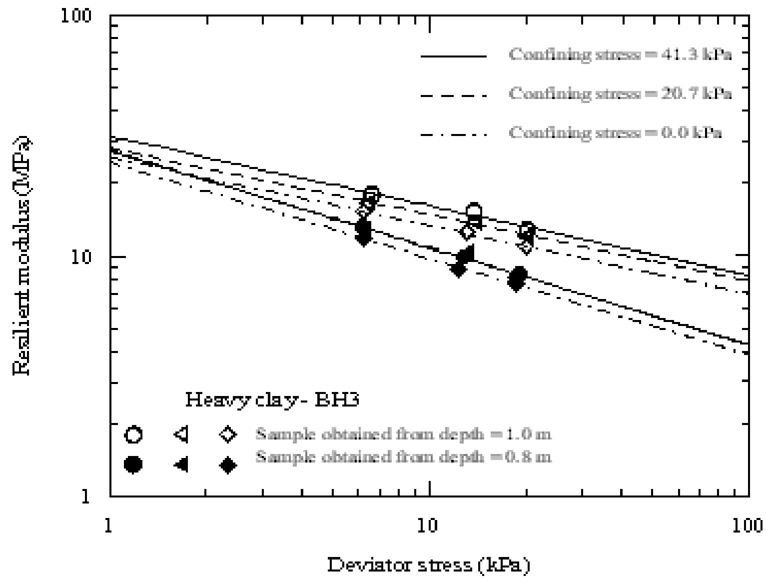


Figure 19
(c) Variation of the resilient modulus with deviator stress at different confining pressures for heavy clay obtained from borehole BH3

(γ=19.8 kN/m^3) obtained from the laboratory compaction test. The resilient modulus of the PRF-heavy clay is low compared to the PRF-silty clay. The PRF-heavy clay is a soft soil with a range of in situ moisture content between 59 and 65.1 percent and the unit weight ranges from 16 to 16.4 kN/m^3. The optimum moisture content of the PRF-heavy clay obtained in the laboratory is 31.4 percent and the corresponding unit weight is 17.8 kN/m3. The high amount of moisture in the PRF-heavy clay is the main reason for the lower resilient modulus of this soil.

The resilient modulus of coarse-grained (cohesionless) soils increases with the increase in the bulk stress. This behavior is shown in figure 20 for Larose sand. Similar behavior was also observed for the silty sand obtained from LA-28 test site.

The resilient modulus of the investigated fine-grained soils reflects a typical behavior where the values of the resilient modulus at constant confining pressure decrease with the increase of the deviator stress. In addition, the resilient modulus values at a constant deviator stress increase with the increase of the confining pressure.

CPT-Resilient Modulus Correlation

The objective of this study is to establish a correlation between the cone penetration test output and the resilient modulus of subgrade soil and to provide a validation for this correlation. Therefore, an experimental program was carried out in which cone penetration tests were conducted near boreholes from which undisturbed soil samples were tested to determine their resilient modulus. Analyses for both the resilient modulus and the cone penetration tests assured the reliability of the test results.

In order to establish a correlation between the cone penetration output and the resilient modulus, the variables affecting both tests are identified. The cone tip resistance (q_c), sleeve friction (f_s), and resilient modulus (M_r) are affected by the soil type, unit weight (γ), moisture content of the soil (w), and state of stress (σ). Therefore the attempt made in this study accounts for the effects of the cone resistance, sleeve friction, soil properties, and stresses on the prediction of resilient modulus.

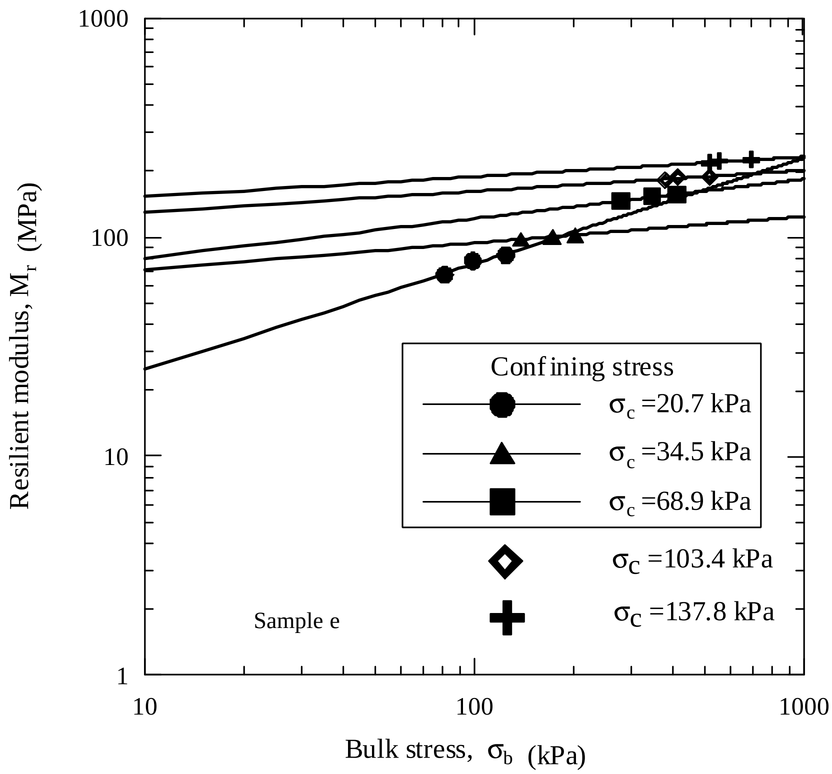


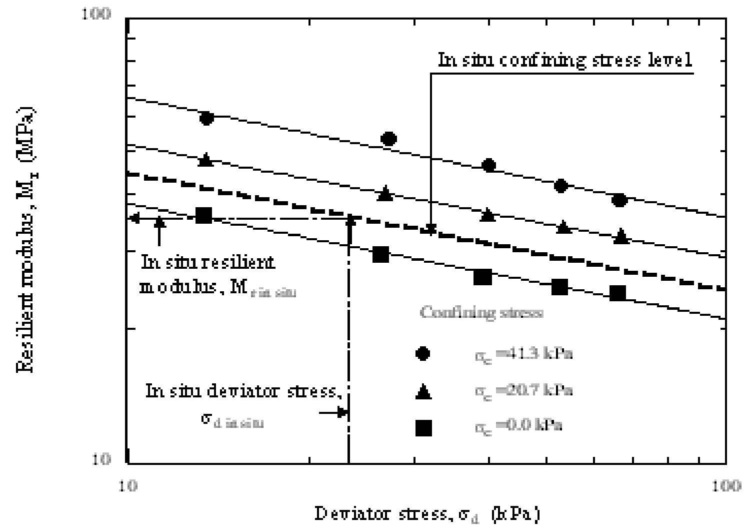
Figure 20

Variation of the resilient modulus with bulk stress at different confining pressures for Larose sand

The resilient modulus obtained from the laboratory repeated loading triaxial test vary with deviator stress. Therefore, it is necessary to identify a single value of the resilient modulus from the laboratory test corresponding to the in situ stress conditions. This value of the resilient modulus is interpolated from the laboratory test and is denoted as the field resilient modulus value. A procedure was developed to obtain the field resilient modulus values for the investigated soils and is illustrated in figure 21. The average depths of the undisturbed soil samples (i.e. 0.8 and 1.0 m) were considered to determine the stresses acting on the soil element. Then, the resilient modulus values were interpolated from the laboratory test results based on the stresses of each soil sample, as illustrated in figure 21. In order to obtain a representative range of values for the resilient modulus, two cases for stresses were considered for each test sample. These cases comprise the maximum and minimum possible values for the resilient modulus: (a) the soil is under no traffic loading, and (b) the soil is under a standard single wheel loading of 20 kN (4.5 kips). The first case considers the soil under in situ (K_o) condition, where the in situ stresses were calculated from the soil unit weight and the depth of the soil element under consideration. In the second case, the traffic loading is added to the in situ stresses acting on the soil element under consideration. The stresses due to traffic loadings were determined using the computer code for the analysis of linear-elastic pavement systems, ELSYM5 *[18]*. The configuration of the different pavement layers considered in the elastic analysis is presented in figure 22. The elastic soil parameters (modulus of elasticity E and Poisson's ratio ν) for the investigated soils are presented in table 7. The modulus of elasticity values were determined from the repeated load triaxial test and Poisson's ratio for the soils, which were estimated based on similar case histories.

Proposed CPT-Resilient Modulus Correlations

A summary of the field and laboratory test results for the fine-grained soils is presented in table 8. These results represent the resilient modulus values corresponding to the in situ stress conditions. The results of the analysis to obtain the resilient modulus corresponds to the 20 kN (4.5 kips) standard single wheel loading, presented in table 9. The variables presented in tables 8 and 9 are considered in the analysis to correlate the resilient modulus and the cone penetration test output. Statistical analyses (multiple regression) were performed using the Statistical Analysis System (SAS) program. Forward selection, backward elimination, all possible regression, and stepwise procedures were used to select the variables in these correlations. The output of the miniature friction cone penetrometer (which is developed mainly for pavement design) is considered in the current study.



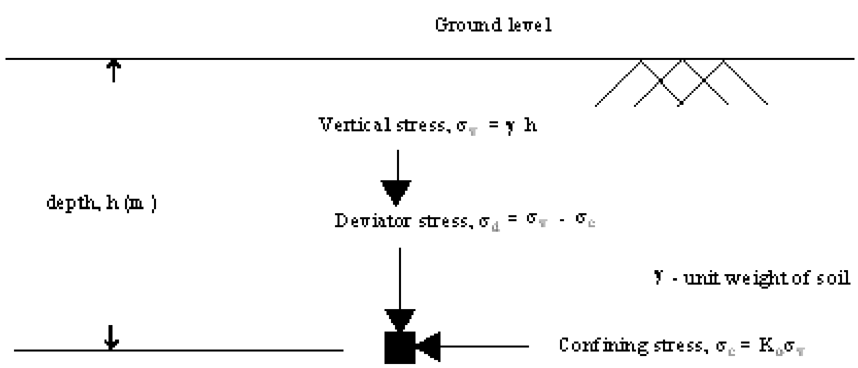


Figure 21
Estimation of in situ resilient modulus from the laboratory resilient modulus test results

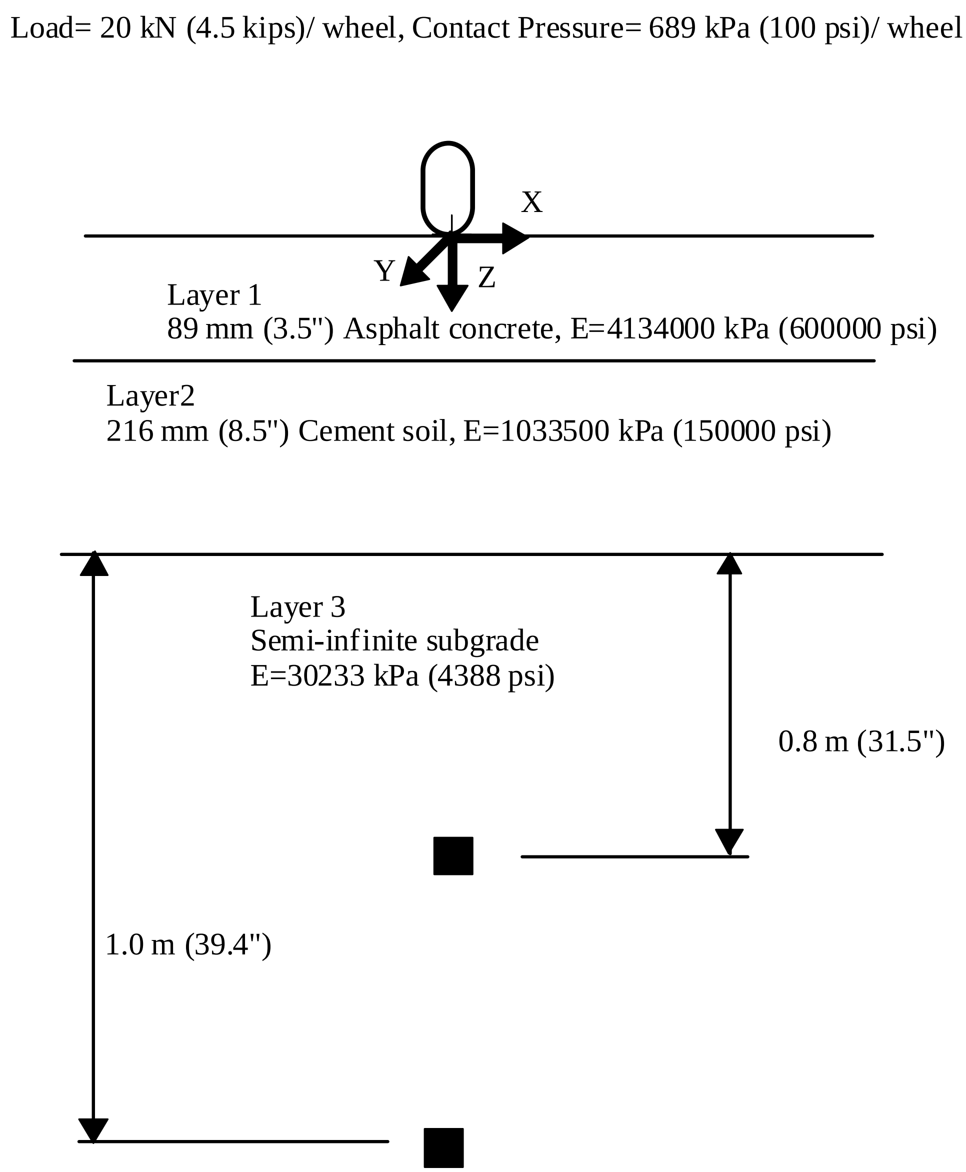


Figure 22
A typical pavement section for traffic stress analysis.

Table 7(a)
Elastic properties of the investigated soil sites

		PRF-Silty clay	PRF-Heavy clay	I-10/ LA-42 Clay	LA-15 Clay	LA-1 Sand	LA-28 Sand	LA-89 Silty clay loam	Siegen Lane clay
Embankment	**E (MPa)**	**49.2**	-	-	-	-	-	-	-
	n	**0.45**	-	-	-	-	-	-	-
Subgrade	**E (MPa)**	-	**21.5**	**30.2**	**63.6**	**NA**	**NA**	**37.6**	**41.4**
	n	-	**0.35**	**0.35**	**0.35**	**NA**	**NA**	**0.35**	**0.35**

Legend: NA: not available, n**: Poisson's ratio, E: Modulus of elasticity**

For all soil sites, elastic properties for the assumed pavement configuration as shown in figure 22 are given in table 7 (b).

Table 7(b)
Elastic properties for the assumed pavement layers

Asphalt concrete surface 89 mm (3.5")	**E (MPa)**	**4134.0**
	n	**0.35**
Base 216 mm (8.5")	**E (MPa)**	**1034**
	n	**0.35**

Table 8
Summary of field and laboratory tests on the investigated soils (M_r under in situ condition)

Soil Sample	Depth (m)	M_r (MPa)	q_c (MPa)	f_s (MPa)	Water content(%)	Dry unit weight (kN/m^3)
PRF-Silty clay						
BH1-top	0.8	48.4	2.50	0.0662	25.4	15.9
BH1-bot	1	50.4	3.20	0.0714	23.0	16.5
BH2-top	0.8	54.1	2.69	0.0905	20.8	16.8
BH2-bot*	1	100.2	3.66	0.1028	21.0	17.0
BH3-top	0.8	63.3	2.82	0.0733	23.2	16.9
BH3-bot	1	60.7	3.15	0.0921	21.5	17.0
PRF-Heavy clay						
BH1-top	0.8	14.3	0.28	0.0185	61.6	9.9
BH1-bot	1	14.6	0.31	0.0201	65.1	9.9
BH2-top	0.8	24.7	0.32	0.0229	60.4	10.2
BH2-bot	1	26.2	0.40	0.0229	62.5	10.0
BH3-top	0.8	24.8	0.39	0.0185	59.0	10.2
BH3-bot	1	24.5	0.38	0.0178	59.5	10.3
I-10/LA-42 clay						
BH1	0.8	43.3	2.08	0.1040	21.5	16.9
BH1	1	32.2	1.88	0.1122	19.6	17.2
BH2	0.8	19.6	1.13	0.0556	23.0	16.5
BH2	1	33.4	2.01	0.1194	21.4	16.3
BH3	0.8	34.9	1.82	0.0943	20.8	16.8
BH3	1	18.0	1.24	0.0623	22.5	16.4
LA-15 clay						
BH1-top	0.8	77.4	2.85	0.1509	24.1	17.3
BH1-bot	1	58.3	2.08	0.1141	23.0	16.2
BH2-top	0.8	52.8	2.07	0.1233	28.4	16.8
BH2-bot	1	53.0	2.14	0.0968	27.3	15.3
BH3-top	0.8	83.3	3.07	0.1345	18.8	17.8
BH3-bot	1	56.9	2.05	0.1095	31.4	15.2
LA-89 clay						
BH1-bot	0.6	45.6	1.74	0.0990	24.9	18.1
BH2-top	0.8	36.1	1.36	0.1076	26.8	16.1
BH2-bot	1.6	14.9	0.50	0.0619	28.6	15.9
BH3-bot	0.6	53.8	1.79	0.1043	24.6	17.1
Siegen Lane clay						
BH1-top	0.6	54.6	3.10	0.1241	9.5	18.3
BH1-bot	1.2	35.9	1.32	0.1560	22.5	17.1
BH2-top	0.8	61.1	3.36	0.1134	16.7	17.1
BH2-bot	1.2	63.1	3.66	0.1166	21.9	17.3
BH3-bot	1.3	33.2	1.61	0.1050	23.1	15.4

Legend: M_r -Resilient modulus, q_c -Cone tip resistance, f_s -Cone sleeve friction, BH -Borehole, * -Excluded

Table 9
Summary of the stress analysis on the investigated soils

Soil sample	Depth (m)	In situ			In situ + traffic		
		σ_c (kPa)	σ_d (kPa)	M_r (MPa)	σ_c (kPa)	σ_d (kPa)	M_r (MPa)
PRF-silty clay							
BH1	0.8	12.4	4.5	48.4	15.8	10.6	40.0
BH1	1	15.7	5.3	50.4	18.9	11.1	42.9
BH2	0.8	12.4	4.5	54.1	15.8	10.6	45.2
BH3	0.8	12.4	4.5	63.4	15.8	10.6	49.9
BH3	1	15.7	5.3	60.7	18.9	11.1	54.7
PRF-heavy clay							
BH1	0.8	11.9	1.3	14.3	17.1	5.7	8.2
BH1	1	14.9	1.6	14.6	20.0	5.7	8.9
BH2	0.8	11.9	1.3	24.7	17.1	5.7	12.6
BH2	1	14.9	1.6	26.2	20.0	5.7	18.2
BH3	0.8	11.9	1.3	24.8	17.1	5.7	13.4
BH3	1	14.9	1.6	24.5	20.0	5.7	17.3
I-10/LA-42 clay							
BH1	0.8	8.6	7.8	43.3	12.2	14.0	38.0
BH2	0.8	8.5	7.8	32.2	12.2	14.0	29.7
BH2	1	10.7	9.7	19.6	14.2	15.6	17.3
BH3	0.8	8.3	7.6	33.4	11.9	13.8	28.2
BH4	0.8	8.4	7.7	34.9	12.1	13.9	30.9
BH4	1	10.5	9.6	18.0	14.1	15.4	16.0
LA-15 clay							
BH1	0.8	13.0	4.1	77.4	18.1	9.4	70.9
BH1	1	15.1	4.8	58.3	20.2	9.5	51.8
BH2	0.8	13.1	4.1	52.8	18.2	9.4	54.5
BH2	1	14.8	4.7	53.0	19.8	9.4	50.8
BH3	0.8	12.9	4.1	83.3	18.0	9.4	77.7
BH3	1	15.2	4.8	56.9	20.2	9.5	46.4
LA-89 clay							
BH1	0.6	9.1	3.7	45.6	13.9	9.5	43.3
BH2	0.8	11.6	4.8	36.1	16.5	9.9	26.7
BH2	1.6	23.2	9.5	14.9	27.8	13.5	13.7
BH3	0.6	9.1	3.7	53.8	13.9	9.5	36.8
Siegen Lane clay							
BH1	0.6	8.1	4.0	54.6	12.7	10.2	59.8
BH1	1.2	16.8	8.3	35.9	21.2	13.0	35.1
BH2	0.8	10.7	5.3	61.1	15.3	10.8	71.9
BH3	1.3	16.3	8.0	33.2	20.7	12.6	32.3

Legend: σ_c - Confining stress , σ_d - Deviator stress, M_r - Resilient modulus, BH- Borehole

Fine-Grained (Cohesive) Soils - In situ Conditions. A model is proposed for the fine-gained soils under in situ conditions. The variables for the PRF-silty clay and PRF-heavy clay were only used to develop the model, since they represent the stiff and the soft soil types, respectively. The following model is proposed for the fine-grained soils:

$$\frac{M_r}{s_c^{0.55}} = \frac{1}{s_v}\left(31.79q_c + 74.81\frac{f_s}{w}\right) + 4.08\frac{g_d}{g_w} \qquad (4)$$

Where, M_r is the resilient modulus (MPa), q_c is the cone resistance (MPa), f_s is the sleeve friction (MPa), s_c is the confining stress (kPa), s_v is the vertical stress (kPa), w is the water content in decimal number format, g_d is the dry unit weight (kN/m^3), and g_w is the unit weight of water (kN/m^3). The root of the mean squared error for this model is *RMSE*=1.37 and the coefficient of determination is R^2 = 0.99.

In this regression model, the coefficient of determination, R^2 is 0.99. Therefore, a successful statistical correlation between the resilient modulus and the cone data was developed for different types of soils.

This model was developed and calibrated based on the field and laboratory tests on the PRF-silty clay and PRF-heavy clay. This model was then used to predict the resilient modulus of the investigated cohesive soils from LA-42, LA-15, LA-89, and Siegen Lane. The results of the predicted versus measured resilient modulus are shown in figure 23. The predicted and measured values of resilient modulus are in good agreement.

Fine-Grained (Cohesive) Soils - In situ Conditions and Traffic Loading.
In this analysis, a fictitious pavement configuration with a standard 20 kN (4.5 kips) wheel loading was placed on each type of soil site as shown in figure 22. Then for this traffic loading, the stresses in the soil at a particular depth were computed by elastic analysis program, ELSYM5 *[18]*. Then, the in situ stresses and traffic stresses were superimposed to compute the combined major and minor (confining stress) principal stresses, at this soil element. From the major and minor principal stresses, deviator stress was calculated. From the laboratory resilient modulus test at these confining stress and deviator stress, resilient modulus under this traffic loading was estimated. The results are presented in table 9. These

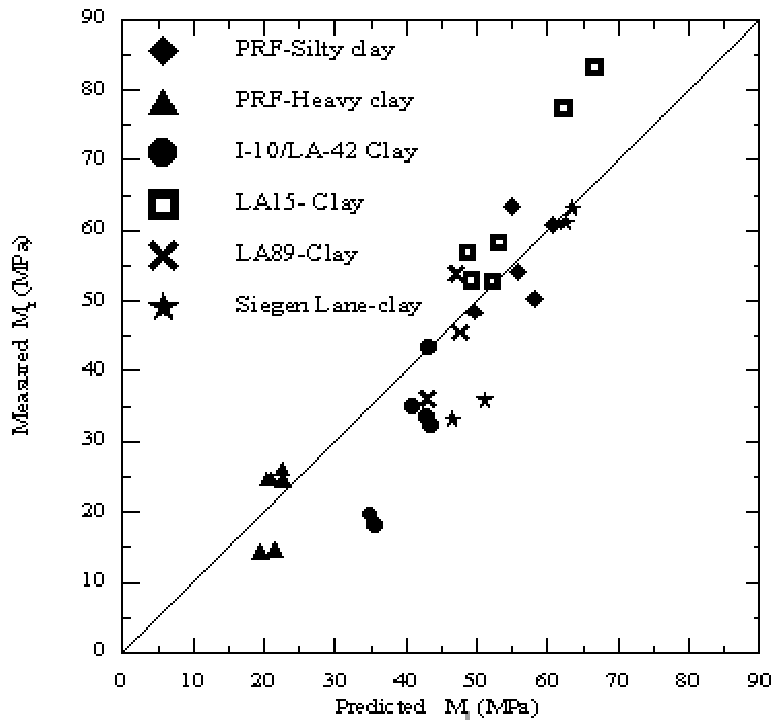


Figure 23
Prediction of in situ resilient modulus by cone penetration test results

results were used to correlate cone penetration test parameters. The miniature cone penetration tests were used in the analysis.

Appendix A presents an illustration of using ELSYM5 [18] to determine the stresses within the pavement layers.

The following nondimensional correlation between the cone data and the resilient modulus was developed from the PRF-silty clay and PRF-heavy clay:

$$\frac{M_r}{s_3^{0.55}} = \frac{1}{s_1}\left(47.03 q_c + 170.40 \frac{f_s}{w}\right) + 1.67 \frac{g_d}{g_w} \qquad (5)$$

s_1- is the major principal stress and s_3 is the minor principal stress. For this correlation, R^2 =0.99, RMSE=0.80.

This model was developed and calibrated based on the field and laboratory tests on the PRF-silty clay and PRF-heavy clay. The model was then used to predict the resilient modulus of the investigated cohesive soils from LA-42, LA-15, LA-89, and Siegen Lane under in situ and traffic loadings. The results of the predicted versus measured resilient modulus are shown in figure 24. The predicted and measured values of the resilient modulus are consistent.

Coarse-Grained (Cohesionless) Soils. A summary of the field and laboratory test results for the coarse-grained soils is presented in table 10. These results represent the resilient modulus values corresponding to the in situ stress conditions. Traffic loading was not considered, due to the lack of data. For sand, CPT data was considered in the analysis because of the difficulties encountered in penetrating dense sand using the miniature friction cone. The cement stabilized soil at LA-89 showed a behavior similar to the sand under repeated loading triaxial tests and is therefore analyzed in this category.

As the resilient modulus increases with the bulk stress in sand, the following nondimensional correlation between the cone data and the resilient modulus was developed, using data from LA-28 and LA-89 cement stabilized soil:

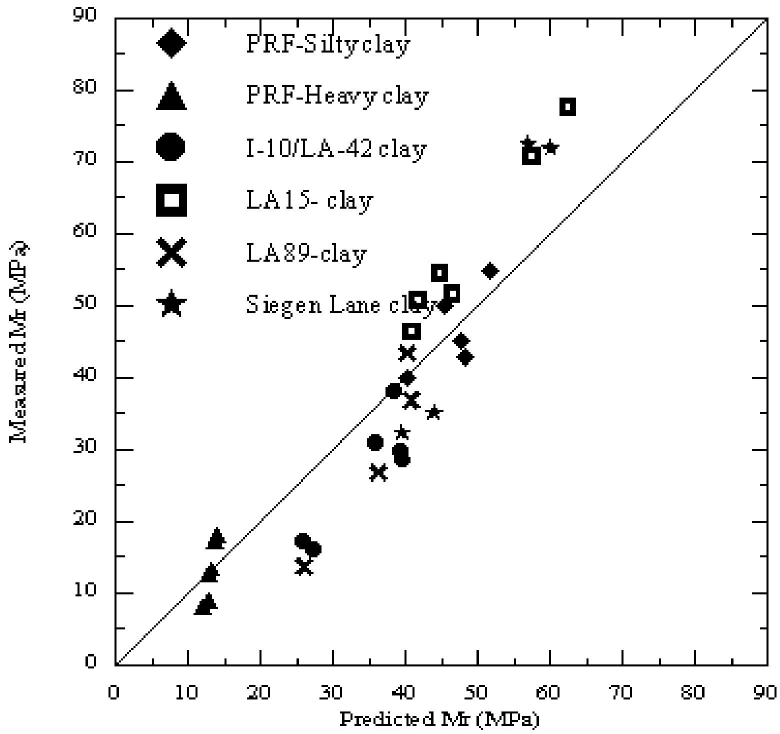


Figure 24
CPT-Resilient modulus of cohesive soil under traffic loading

Table 10

Summary of the field and laboratory tests on the LA-28 sand and LA-89 lime treated recycled cement soil (In-situ condition).

	Depth (m)	M_r (MPa) In-situ	σ_c (kPa)	σ_d (kPa)	q_c (MPa)	f_s (MPa)	w (%)	γ_d (kN/m^3)
LA-28								
BH1	0.8	34.6	9.13	8.09	2.3	0.0148	11.0	19.4
BH1	1	24.0	10.61	9.41	2.1	0.0250	16.3	17.2
BH2	0.8	40.1	10.44	9.26	2.3	0.0217	11.1	17.7
BH3	0.8	32.4	8.18	7.26	2.4	0.0238	10.3	17.5
LA-89								
BH1	0.4	43.2	5.55	1.95	6.5	0.3240	18.8	15.8
BH3	0.4	53.9	6.0	2.2	8.1	0.3690	17.9	17.2

Legend: M_r - Resilient modulus, q_c - Cone tip resistance, f_s - Cone sleeve friction,
σ_c - Confining stress, σ_d - Deviator stress, γ_d - Dry unit weight

$$\frac{M_r}{\sigma_c^{0.55}} = 6.66\frac{q_c \sigma_b}{\sigma_v^{2}} - 32.99\frac{f_s}{q_c} + 0.52\frac{\gamma_d}{w\gamma_w} \tag{6}$$

Where, M_r is the resilient modulus (MPa), q_c is the cone resistance (MPa), f_s is the sleeve friction (MPa), σ_c is the confining stress (kPa), σ_v is the vertical stress (kPa), σ_b is the bulk stress (kPa), w is the water content in decimal number format, γ_d is the dry unit weight (kN/m^3), and γ_w is the unit weight of water (kN/m^3). The root of the mean squared error for this model is RMSE=0.96 and the coefficient of determination R^2 = 0.99. In this regression model, the coefficient of determination, R^2, is as high as 0.99. Therefore, the statistical correlation between the resilient modulus and the cone data is considered successful.

The model, given in equation (6), was developed using data from LA-28 and LA-89 cement stabilized soil. Figure 25 depicts the comparison of the predicted resilient modulus and the measured resilient modulus. The measured and predicted resilient moduli are in good agreement. Due to the lack of data on resilient modulus and cone penetration tests for sand, validation of this model to predict the behavior of other soil was not conducted.

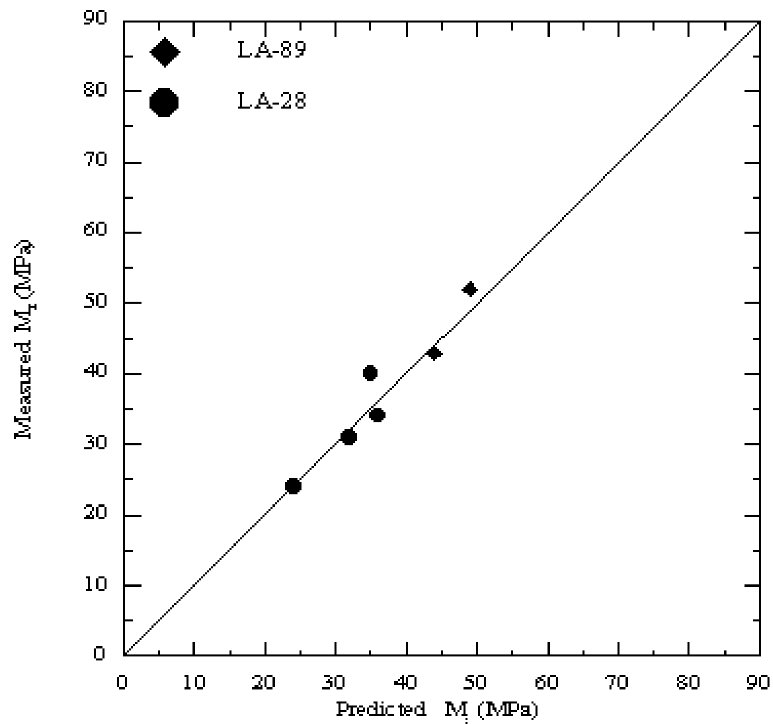


Figure 25
Predicted versus measured resilient modulus of the investigated cohesionless soils.

CONCLUSIONS

This report presented a pilot investigation to assess the applicability of the intrusion technology for estimating the resilient modulus of subgrade soils. Field and laboratory testing programs were carried out at seven sites that comprise three common soil types in Louisiana. Site characterization was conducted using cone penetration tests in which continuous measurements of the cone tip resistance and sleeve friction were recorded. Undisturbed and disturbed soil samples were also obtained from different depths at the investigated sites. Laboratory tests were conducted on soil samples to determine the resilient modulus, strength parameters, physical properties, and compaction characteristics. Results of both field and laboratory testing programs were analyzed and critically evaluated.

Statistical analyses were conducted on the cone soundings and showed that the results are repeatable at each test site within tolerable deviation. Statistical models for predicting the resilient modulus were developed, based on the field and laboratory test results for cohesive as well as cohesionless soils. These models correlate the resilient modulus to the cone penetration test parameters, basic soil properties, and in situ stress conditions of the soil. The model for cohesive soil was calibrated based on the test results of two soil types and used to predict the resilient modulus of the other soils. Predicted and measured values of the resilient modulus are in good agreement. This research provided a preliminary validation of predicting the resilient modulus of subgrade soils utilizing the cone penetration test.

As a result of the current research, the following conclusions were reached:

1. Despite the fact that the CPT and resilient modulus are fundamentally different, the resilient modulus was predicted from the CPT results within acceptable tolerance.

2. The continuous intrusion miniature cone penetration test system is a robust and reliable system, when used in cohesive soils; increased thrust is needed to conduct tests on sandy soils.

3. The cone tip resistance and sleeve friction of the miniature cone penetrometer were calibrated with respect to the 15 cm^2 friction cone, where the tip resistance of the miniature cone was found 11 percent higher than the tip resistance of the 15 cm^2 friction cone and the sleeve friction of the miniature cone were found 9 percent lower than that of 15 cm^2 friction cone. This calibration is only valid for cohesive soils.

4. The proposed model for cohesive soils predicts the resilient modulus of subgrade soils by CPT and soil properties. Two cases were considered which comprise the in situ stresses and traffic loading. The model was calibrated and then used to predict the measured resilient modulus of subgrade soils. Predicted and measured values are in agreement. The coefficient of determination for the predicted resilient modulus values under in situ stresses is 0.99 and the RMSE is 1.19. When traffic loading was considered, the coefficient of determination is 0.99 and RMSE is 0.72. These values indicated a good predictive capability of the cohesive soil model.

5. The proposed model for the cohesionless soils was not validated, due to the lack of data. Most of the investigated sites are mainly cohesive soils (which is the common soil types in Louisiana).

RECOMMENDATIONS

Currently, the design of flexible pavements is generally conducted based on static properties such as California Bearing Ratio (CBR) and soil support value. These properties do not represent the actual response of the pavement layers under traffic loadings. Recognizing this deficiency, the current and the 2002 American Association of State Highway and Transportation Official's guide for design of pavement structures recommended the use of resilient modulus for characterizing the base and subgrade soil and for the design of flexible pavements.

This report presents the findings from a pilot investigation to assess the applicability of intrusion technology to estimate the resilient modulus of subgrade soils. Models for predicting soil resilient modulus from cone penetration test parameters, basic soil properties, and soil insitu stress conditions were developed.

These models were successfully used in several overlay projects to evaluate the subgrade stiffness along with conventional approach. The evaluation is on-going by identifying field projects in each district and applying this technology during the rehabilitation design stage.

In addition to the above exposure of minicone technology to estimate resilient modulus, workshop sessions are planned for the dissemination of this approach to DOTD design engineers. This will accelerate the implementation of this effective and fundamental approach in pavement design and analysis.

For a successful implementation of this study, it is anticipated that DOTD will provide the necessary budgetary funds required for the acquisition of the Continuous Intrusion Miniature Cone Penetration Test system for each district.

LIST OF ACRONYMS / ABBREVIATIONS / SYMBOLS

AASHTO: American Association of State Highway and Transportation Officials
ALF: Accelerated Load Facility
ASTM: American Society for Testing and Materials
BH: Borehole
c: Cohesion intercept
CBR: California Bearing Ratio
CICU: Isotropically Consolidated Undrained Triaxial Compression Test
COV: Coefficient of Variation
CPT: Cone Penetration Test
CIMCPT: Continous Intrusion Miniature Cone Penetrometer Test System
E: Modulus of elasticity (Young's modulus)
EASL: Equivalent Single Axial Loading
EMCRF: Engineering Materials Characterization Research Facility
FWD: Falling Weight Deflectometer
G: Shear modulus
G_s: Specific gravity
k_0: Coefficient of lateral earth pressure
DOTD: Louisiana Department of Transportation and Development
LCD: Liquid Crystal Display
LL: Liquid Limit
LTRC: Louisiana Transportation Research Center
LVDT: Linear Variable Differential Transducer
MTS: Material Testing System
M_r: Resilient modulus
MCPT: Miniature Cone Penetrometer Test
NA: Not Available
NDT: Nondestructive Test
PI: Plasticity Index
PRF: Pavement Research Facility
REVEGITS: Research Vehicle for Geo-technical In situ Testing and Support
RMSE: Root Mean Squared Error
R^2: Coefficient of determination
SAS: Statistical Analysis System
SHRP: Strategic Highway Research Program
STD: Standard deviation
S_u: Undrained shear strength
USCS: Unified Soil Classification System
UU: Unconsolidated Undrained Triaxial Test
w: Water content in decimal number format
w_{opt}: Optimum water content
f_s: Sleeve friction
q_c: Cone tip resistance

R_f: Friction ratio
ε_r : Axial strain
σ_1: Major principal stress
σ_3: Minor principal stress
σ_c: Confining stress
σ_d: Deviator stress
σ_h: Horizontal stress
σ_v: Vertical stress
ϕ: Angle of internal friction
γ_d: Dry unit weight
γ_{dmax}: Maximum dry unit weight
γ_w: Unit weight of water
υ: Poisson's ratio

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APPENDIX A

Calculation of pavement stresses using ELSYM

Calculation of pavement stresses using ELSYM5

Pavement stresses for LA-42/ I-10

In situ stresses at depth 0.8m

From table 3 and table 8,

$$\phi' = 28.5$$
$$\gamma = 20.52 kN/m^3$$
$$h = 0.8m$$
$$K_0 = 1 - \sin\phi' = 0.52$$

Vertical stress,

$$\sigma_v = \gamma h = 20.52 x 0.8 = 16.42 kN/m^2$$

Horizontal stress,

$$\sigma_c = K_0 \sigma_v = 0.52 x 16.42 = 8.54 kN/m^2$$

Deviator stress,

$$\sigma_d = \sigma_v - \sigma_h = 16.42 - 8.54 = 7.88 kN/m^2$$

Stresses due to traffic loading
(See ELSYM5 output)

Minor principal stresses,

$$\sigma_{xx} = \sigma_{yy} = 0.0342 psi = 0.24 kPa$$

Major principal stresses,

$$\sigma_{zz} = 0.482 psi = 3.32 kPa$$

Shear stresses,

$$\sigma_{xy} = \sigma_{yz} = \sigma_{xz} = 0$$

Table A1
Pavement dead load

Thickness (in)	Unit weight, pcf (pci)	Stresses (psi)
3.5	150 (0.087)	0.30
8.5	130 (0.075)	0.64
Total stress due to dead load= 0.94psi (6.50kPa)		

Table A2
In situ and traffic stresses

	In situ stress (kPa)	Traffic stress (kPa)	Pavement dead load (kPa)	Total stress (kPa)
σ_{xx} (σ_c)	8.54	0.24	3.38	12.16
σ_{zz} (σ_v)	16.42	3.32	6.50	26.24

Therefore, confining stress (minor stress) is

$s_c = 12.2 kPa$ and deviator stress is

$$s_d = 26.2 - 12.2 = 14.0 kPa$$

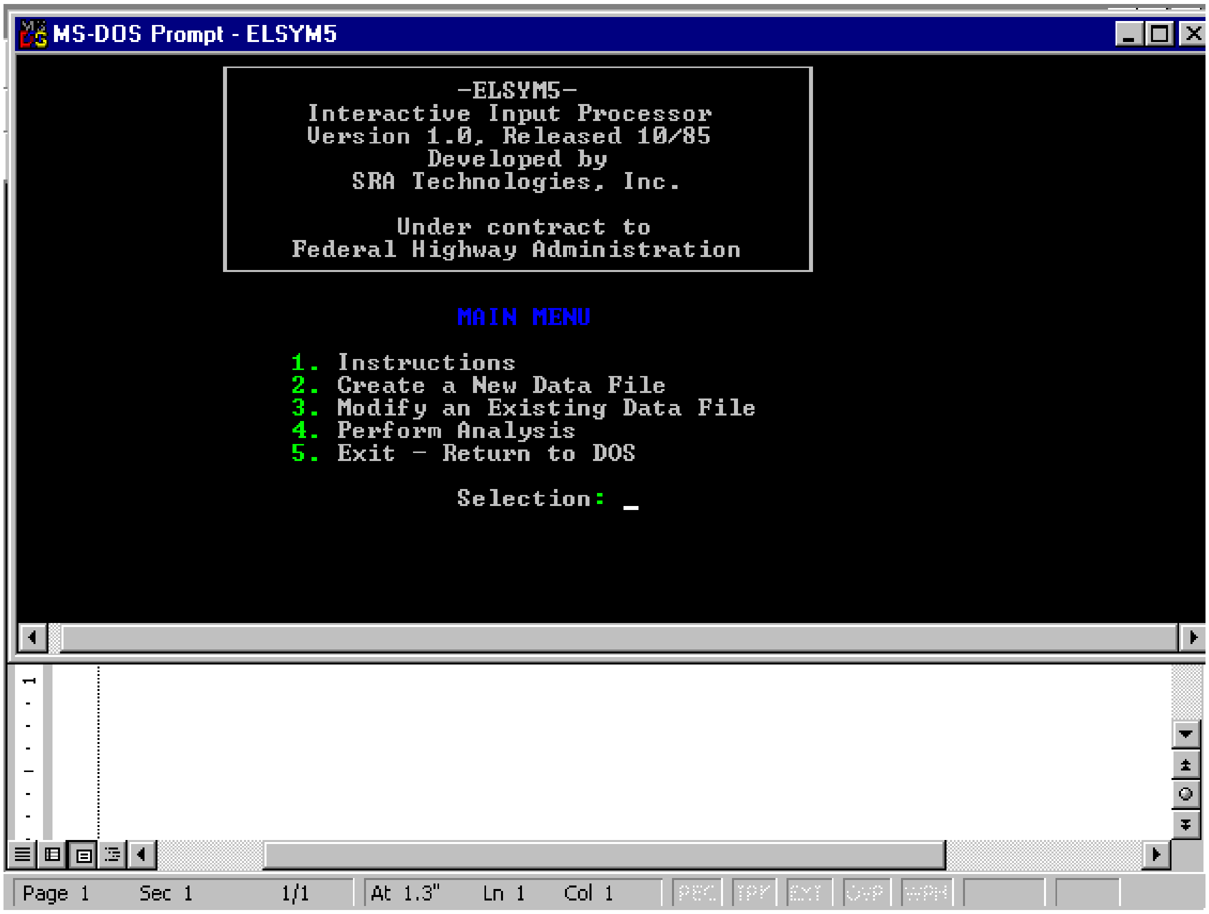

Table A3
Main menu

Note: Select Create a New Data File

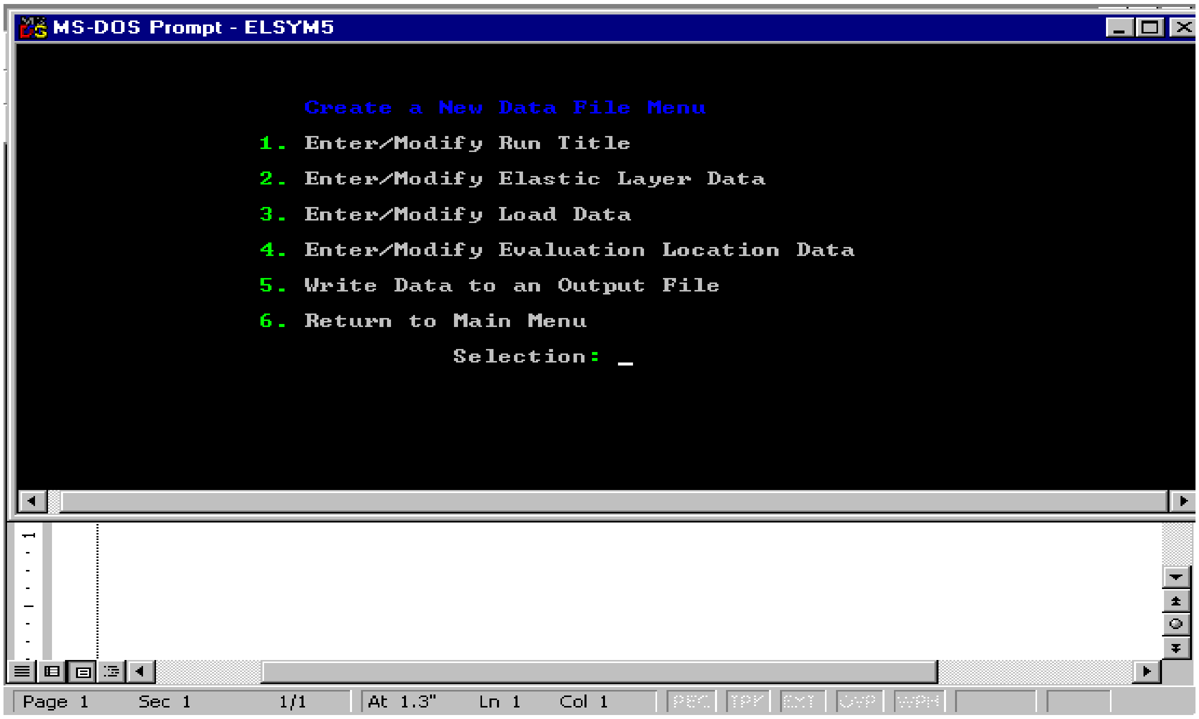

Table A4
Selection menu

Note: Select 1 to 6

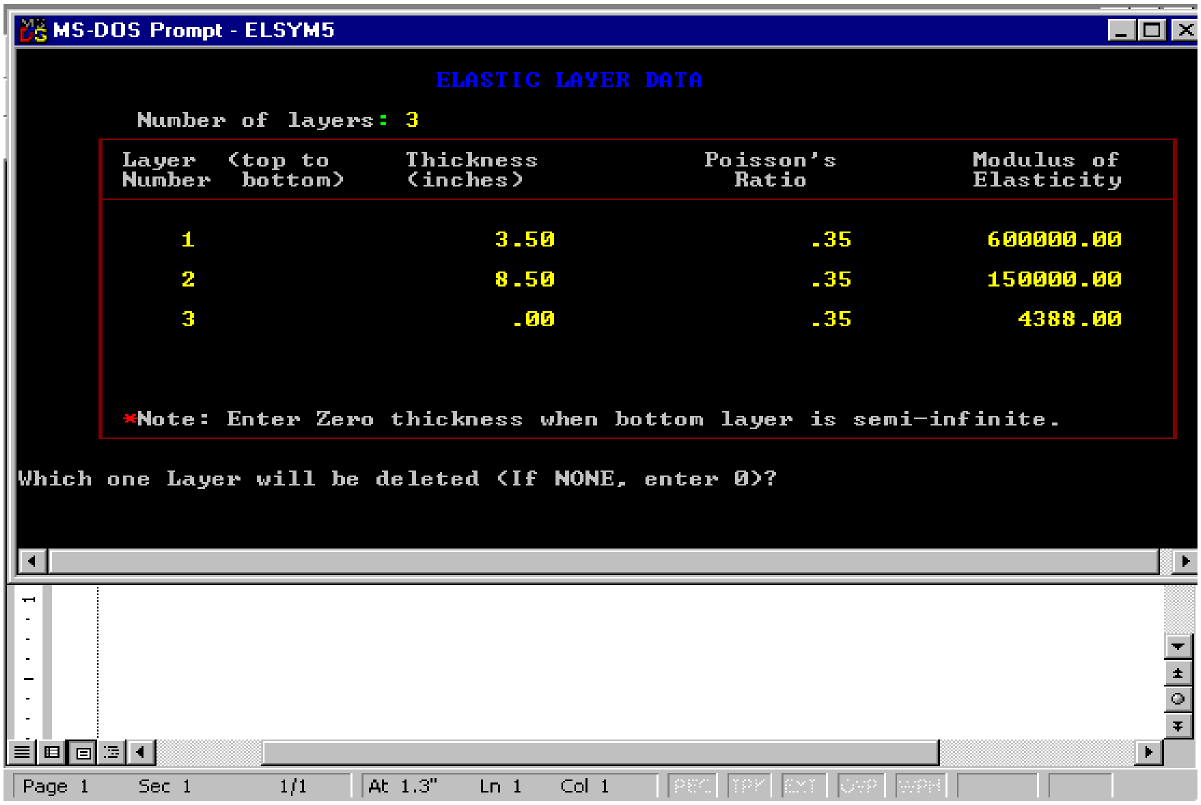

Table A5

Material characteristics

Note: Layer 1: Asphalt concrete, Layer 2: Soil cement, and Layer 3: Subgrade soil

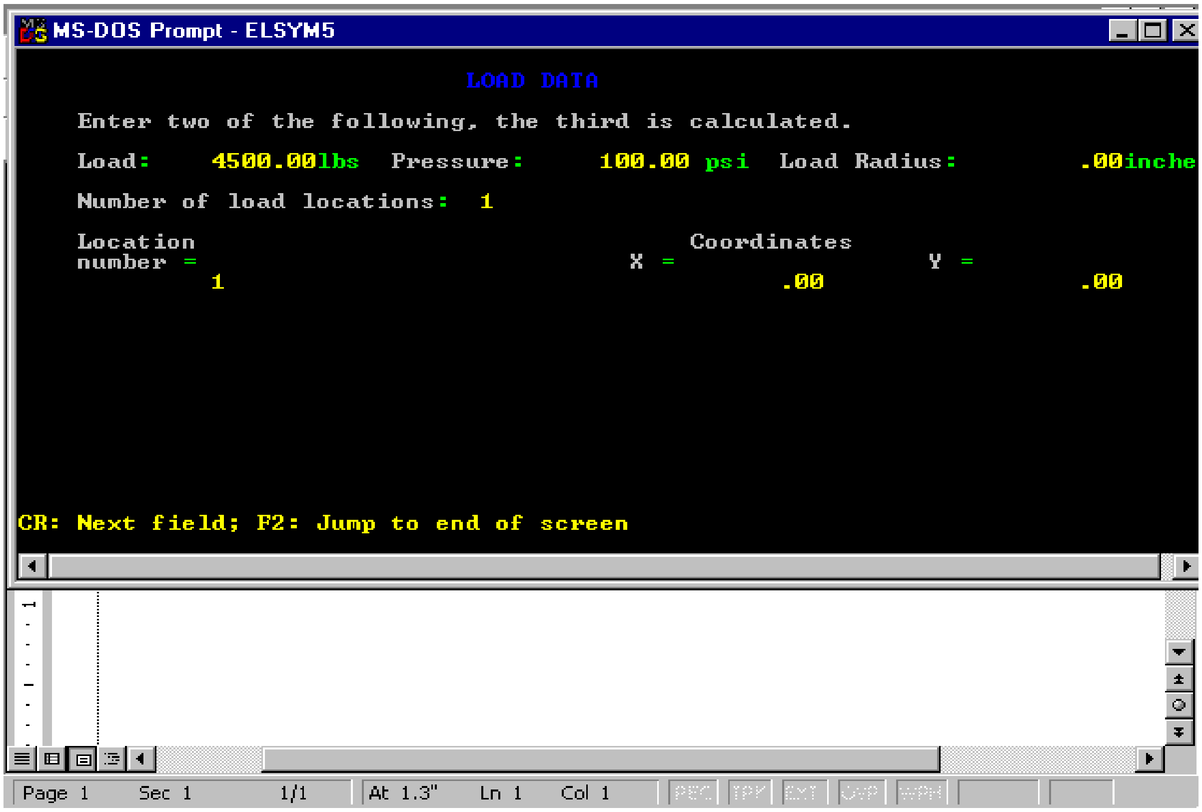

Table A6
Loading input

Note: A wheel load of 4500lbs or stress of 100psi

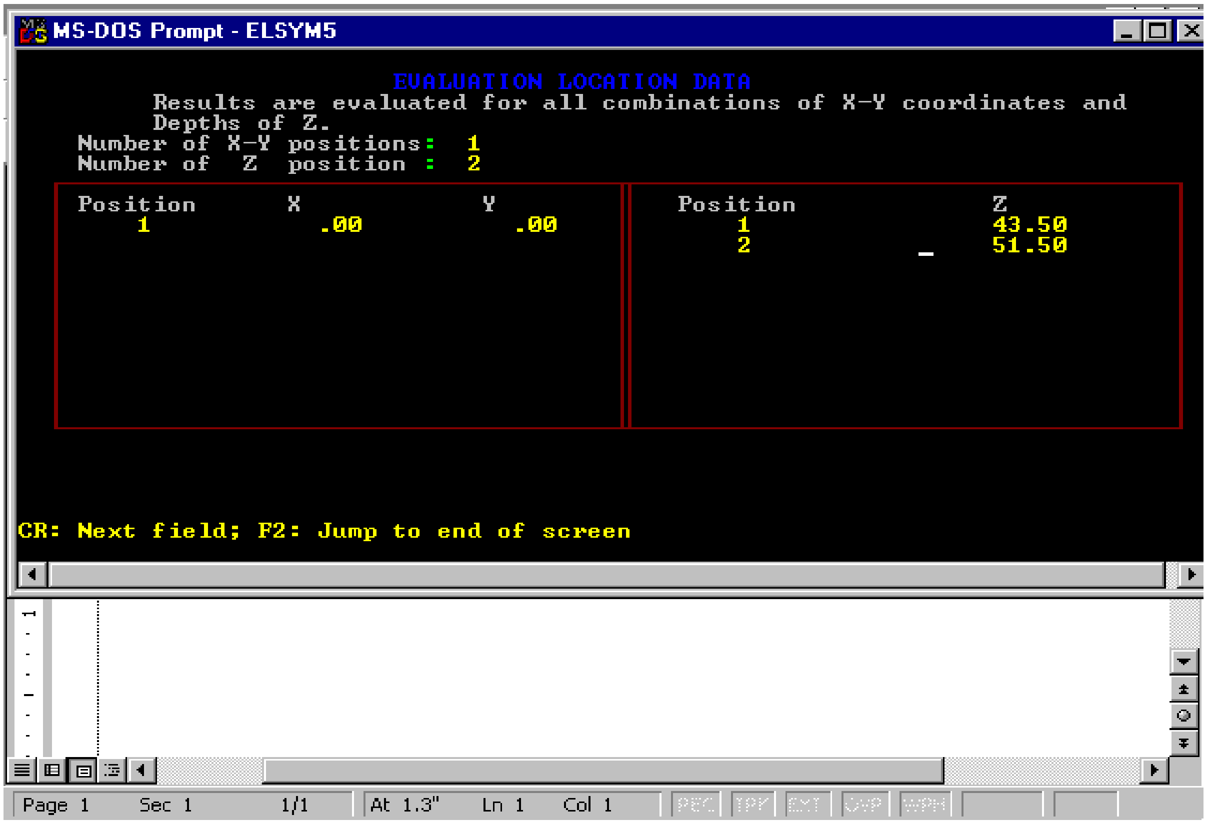

Table A7
Locations for data analysis

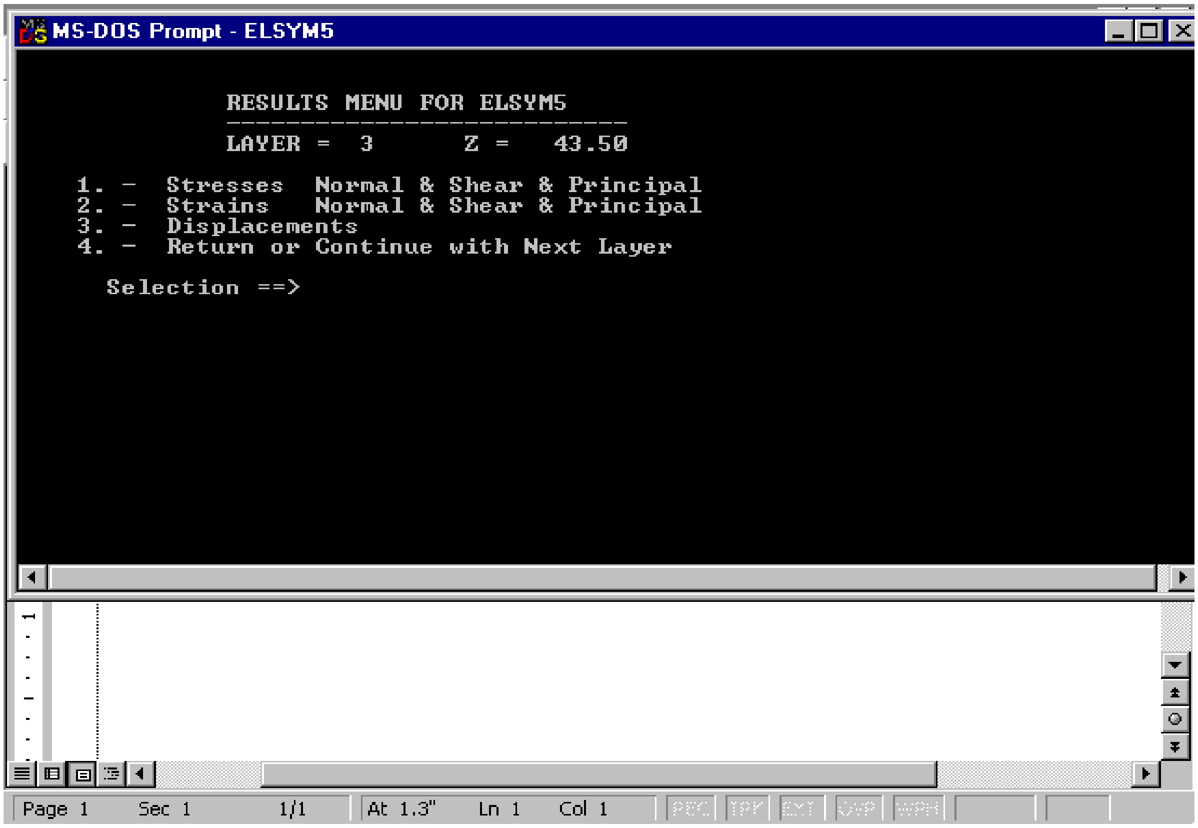

Table A8
Results menu

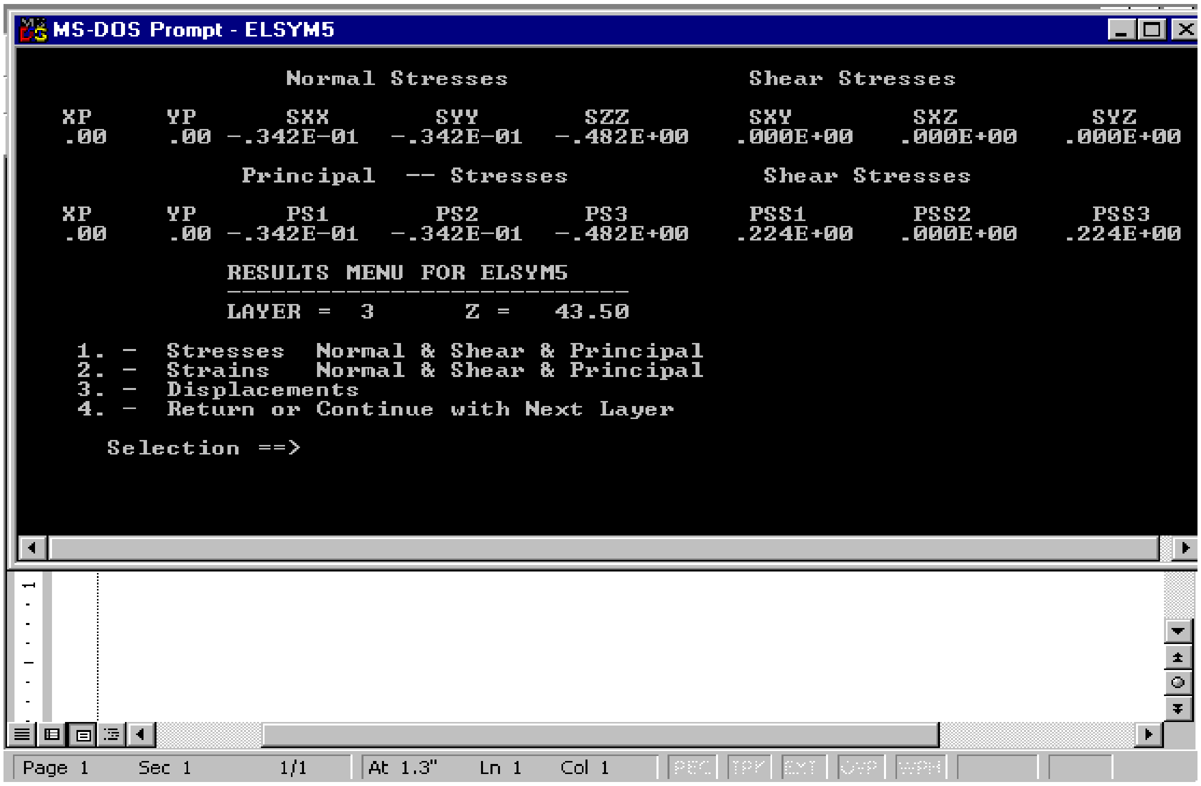

Table A9
Results of stress calculation

Note: Horizontal (minor) stress is 0.0342psi and vertical (major) stress is 0.482psi