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CALIBRATION AND IMPLEMENTATION OF MINIATURE ELECTRONIC CONE PENETROMETERS FOR ROAD AND HIGHWAY DESIGN AND CONSTRUCTION

by

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16. Abstract

A prototype miniature electronic cone penetrometer system is developed for road and highway design and construction control. The equipment is implemented in front of the Research Vehicle for Geotechnical In Situ testing and Support (REVEGITS). The miniature cone penetrometer fabricated by GEOCOGNETICS, Houston, Texas has a cross sectional area of 0.31 in.^{2} (2 cm^2) and a friction sleeve area of 6.2 in.^{2} (40 cm^2). The hydraulic system for miniature cone penetration consists of three jacks, mounted outside, in front of the vehicle. The two outer jacks lower a transverse steel plate to the ground and raise the front of the truck providing the reaction force during penetration. The center jack is the push jack. It has a stroke of 5.9 in. (15 cm) and can deliver a push rate of 2 cm/sec. Scale effects between different size cone penetrometers on in situ test results are investigated using 2.33, 1.55, and 0.20 in.^{2} (15, 10, and 1.27 cm^2) cone penetrometers. Statistical analysis is conducted and regression equations developed to transform Miniature Quasi-Static Cone Penetrometer (MQSC) data and 2.33 in.^{2} (15 cm^2) cone data (cone resistance, q_c; sleeve friction, f_s; and friction ratio, R_f) to the reference penetrometer. From an engineering point of view, a multiplication factor of 0.85 can be used effectively to correct the MQSC cone resistance in order to obtain the reference penetrometer cone resistance. A division factor of 0.85, can also be used to correct the 2.33 in.^{2} (15 cm^2) penetrometer local side friction resistance in order to obtain the reference local side friction resistance. The MQSC's local side friction resistance and friction ratio should be corrected via linear regression equation considering two ranges of cone resistance: (1) soils with q_c equal or smaller than 81.9 ton/ft.^{2} (80 kg/cm^2), and (2) soils with q_c higher than 81.9 ton/ft.^{2} (80 kg/cm^2). No significant correction is necessary for cross-correlating cone resistance of the reference and 2.33 in.^{2} (15 cm^2) cross-section penetrometers. The implementation of the prototype miniature cone penetrometer is tested and verified by comparing penetration profiles with those obtained by the 2.33 in.^{2} (15 cm^2) cone penetrometer. A calibration chamber system for laboratory calibration and development of correlations is also fabricated and calibration tests performed. Initial correlations relating soil compressibility modulus, soil dry density and CBR with cone resistance are developed.

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CALIBRATION AND IMPLEMENTATION OF MINIATURE ELECTRONIC CONE PENETROMETERS FOR ROAD AND HIGHWAY DESIGN AND CONSTRUCTION CONTROL

FINAL REPORT

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ABSTRACT

A prototype miniature electronic cone penetrometer system is developed for road and highway design and construction control. The equipment is implemented in front of the Research Vehicle for Geotechnical In Situ testing and Support (REVEGITS). The miniature cone penetrometer fabricated by GEOCOGNETICS, Houston, Texas has a cross sectional area of 0.31 in.^{2} (2 cm^2) and a friction sleeve area of 6.2 in.^{2} (40 cm^2). The hydraulic system for miniature cone penetration consists of three jacks, mounted outside, in front of the vehicle. The two outer jacks lower a transverse steel plate to the ground and raise the front of the truck providing the reaction force during penetration. The center jack is the push jack. It has a stroke of 5.9 in. (15 cm) and can deliver a push rate of 2 cm/sec. Scale effects between different size cone penetrometers on in situ test results are investigated using 2.33, 1.55, and 0.20 in.^{2} (15, 10, and 1.27 cm^2) cone penetrometers. Statistical analysis is conducted and regression equations developed to transform Miniature Quasi-Static Cone Penetrometer (MQSC) data and 2.33 in.^{2} (15 cm^2) cone data (cone resistance, q_c; sleeve friction, f_s; and friction ratio, R_f) to the reference penetrometer. From an engineering point of view, a multiplication factor of 0.85 can be used effectively to correct the MQSC cone resistance in order to obtain the reference penetrometer cone resistance. A division factor of 0.85, can also be used to correct the 2.33 in.^{2} (15 cm^2) penetrometer local side friction resistance in order to obtain the reference local side friction resistance. The MQSC's local side friction resistance and friction ratio should be corrected via linear regression equation considering two ranges of cone resistance: (1) soils with q_c equal or smaller than 81.9 ton/ft.^{2} (80 kg/cm^2), and (2) soils with q_c higher than 81.9 ton/ft.^{2} (80 kg/cm^2). No significant correction is necessary for cross-correlating cone resistance of the reference and 2.33 in.^{2} (15 cm^2) cross-section penetrometers. The implementation of the prototype miniature cone penetrometer is tested and verified by comparing penetration profiles with those obtained by the 2.33 in.^{2} (15 cm^2) cone penetrometer. A calibration chamber system for laboratory calibration and development of correlations is also fabricated and calibration tests performed. Initial correlations relating soil compressibility modulus, soil dry density and CBR with cone resistance are developed.

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The laboratory and field testing, data acquisition and computational code development phase of this project could not have been completed without the dedicated meticulous efforts of many graduate and undergraduate students. The doctoral research of Dr. Dario Cardoso de Lima, Dr. Pradeep Kurup and Masters research of Mr. Jonxiong Wang were the primary contributors to the project.

The expertise and professionalism of Mr. Raymond Gostowsky and the late Mr. John Vincent in the electronic instrumentation, and design/fabrication of the complex calibration chamber set-up were key factors in the successful completion of the project.

Mr. William T. Tierney (Research Associate), LTRC, contributed boundless physical energy and moral support to the success field implementation phase of the project.

IMPLEMENTATION STATEMENT

The implementation of the penetrometer system in this study will furnish the Department of Transportation with a portable, fast and reliable in-situ soil testing equipment which can be used in subsurface investigations/evaluation as well as embankment construction control especially in locations were accessibility presents a problem. It will provide the engineers and the technical personnel of the Department of Transportation with a wide range of testing capability to be utilized in soil identification and behavior prediction. It is expected that the reduction of disturbed/undisturbed sampling and strength/deformation tests, and reduction of shelf/testing time will result in great savings for the Department of Transportation.

Regression equations developed in this study may be used to convert miniature cone penetration test data to the reference penetrometer data. Classification charts and interpretation methods developed for the reference penetrometer may be then be used to classify the soil and to evaluate various engineering soil parameters. Miniature cone penetration tests were performed in a calibration chamber system on three different but representative soils prevalent in Louisiana. Preliminary correlations between soil compressibility modulus, dry density, CBR and cone resistance were developed.

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INTRODUCTION

The electronic cone penetrometer that works on the principle of measuring tip resistance and frictional resistance around a penetrating probe has become an important in-situ investigation tool of choice for site characterization and determining engineering soil properties. This is mainly attributed to the error free operation of the Cone Penetration Test (CPT) with an automatic data acquisition system and the abundance of information and experience readily available on the CPT method and its interpretation worldwide. A literature review reveals that since its first introduction into ground investigation in the Netherlands in the 1930's, the cone penetration testing technique has been widely accepted and spread through Europe, and in the past two decades the United States and many other countries. CPT originally found its main application in the testing of uniform and homogeneous deposits to determine the bearing capacity of piles. Currently, the use of CPT is being expanded to perform such functions as subsurface characterization in profiling; soil classification; determination of engineering soil properties such as the friction angle and relative density of sand; undrained shear strength and compressibility of clays; determination of settlement; determination of pile capacity; and site construction control for trafficability.

With advances in the functionality and reliability of transducer technology, there have been new additions to the capabilities of the cone penetrometer, such as measuring pore pressure generated during penetration, evaluating soil/groundwater conductivity, and assessing seismic properties of soil strata. Among these developments, the addition of pore pressure sensing elements into the electronic cone penetrometer design has led to the popular acceptance of the Piezocone Penetrometer Test, (PCPT). The pore pressure measurement in PCPT gives more insight into the phenomena associated with the cone penetration mechanism than CPT and helps better identify the strength and deformability characteristics of the soil media being penetrated. Likewise, the probes that measure the soil/groundwater conductivity and dynamic properties of the soil are called "conductivity cone" and "seismic cone" penetrometers, respectively, and have other site specific capabilities. Hereafter in this report "CPT" will be used as a collective term implying various kinds of electronic cone penetrometers. A scaled down version of the electronic CPT probe is commonly referred to as the Miniature Cone Penetrometer (MQCPT or MCPT). The study described in this report is of a LA DOTD research project on MCPT ("Calibration and Implementation of Miniature Electric Cone Penetrometers for Road and Highway Design and Construction Control," State Project No. 736-13-36).

Standard electronic quasi-static cone penetrometers (see Figure 1.1) with a 1.55 in.^{2} (10 cm^2) cross-sectional area, which hereafter will be called the Reference Quasi-Static Electronic Cone Penetrometer (R-QSEC), have been increasingly used in road engineering practice and in-situ soil property evaluations. One major advantage of the R-QSEC is that it can provide continuous and repeatable information in a short period of time throughout the soil profile. It is also possible for this kind of cone penetrometer to adequately perform at moderately deep penetration depths (about 25 to 150 feet (7.62 to 45.72 meters)) because of its characteristic ability to measure eclectric cone resistance on the cone itself as opposed to hydraulic gauges used on the surface by mechanical probes. The error introduced by the frictional resistance caused by soil drag around the push rods and the weight of the rods during deep CPT soundings are minimized with R-QSEC. The expediency, economics reliability, and repeatability of the R-QSEC is strongly recommended, and it has been used extensively in road and highway engineering to provide information on compaction characteristics for construction control and performance evaluation.

However, the need for a high thrust load (about 20 tons (18,144 kilograms)) to push the rods down and a heavy reaction system still limits to some extent the practical field application of R-QSEC. Because of the limitation of R-QSEC, a miniature quasi-static electric cone penetrometer (MQSC) has been developed to expand the field application of QSEC, especially in road and highway engineering. The basic design of a MQSC is based on a similar concept to the electric cone, and its significant advantage over R-QSEC is that it would not necessitate a high thrust load and an elaborate reaction system. It is also economical compared to the R-QSEC.

For road and highway design purposes, the soil compaction characteristics (such as relative density and relative compaction), bearing capacity, compressibility, and shear strength are the most important engineering parameters. The applications of in-situ CPT methods for road engineering in the State of Louisiana are increasing rapidly, and the MQSC penetrometer was investigated for the purpose of classifying of natural soils and determining of engineering properties of compacted embankments for construction control.

The CPT has proved valuable in soil profiling, because the soil type can be identified from the combined measurement of end (tip) resistance and side (sleeve) friction. The test is also used for the derivation of other soil properties, such as relative density, friction angle and cohesion, and bearing capacity, etc. A number of empirical and theoretical interpretations of the CPT results have been developed in recent years, and continued research is adding to the usefulness of the CPT for civil engineering design and performance evaluation purposes. So far, considerable research related to this general interest has been carried out and a number of recommendations for the correlative relationships between cone penetration resistance and soil engineering parameters have been proposed *[1], [2], [3], [4], [5], [6], [7], [8], [9],[10], [11], [12], [13], [14], [15], [16], [17]*. However, further studies of a wider variety of soils are still needed to provide a clearer and comprehensive understanding of this matter. This research investigates the influence of scale effects and develops empirical correlations between cone penetration resistance and important soil engineering parameters for road and highway design and construction control. This report describes the development of state-of-the-art equipment, hardware, and software for a prototype miniature cone penetrometer system for road and highway design and construction control. Development of a calibration chamber system for laboratory calibration and development of correlations for evaluating engineering soil properties from MCPT data are also described. Calibration chamber tests are performed on three soil types at different compactive efforts and at various water contents.

OBJECTIVES

The following were the objectives of this research:

1. Evaluate commercially available miniature cone penetrometers and procure one according to specifications set as a result of the special testing needs peculiar to the soil conditions in Louisiana.

2. Procure a jacking system for advancing and retracting the penetrometer from the ground and also for providing the necessary reaction needed for penetration.

3. Develop a data acquisition system for real time monitoring, acquiring, and displaying of data on a computer screen in graphic form.

4. Conduct In-situ and Laboratory Tests
   (a) Conduct In-situ Tests to evaluate the applicability of MQSC for road and highway design and construction control in Louisiana. Compare the MCPT penetration profiles with CPT profiles obtained with a standard 1.55 in.^{2} (10 cm^2) Fugro-cone penetrometer and with a 2.33 in.^{2} (15 cm^2) Fugro-cone penetrometer to evaluate scale effects and to assess the accuracy of MCPT results.
   (b) Perform Calibration Chamber Tests using the MQSC penetrometer to develop correlations for evaluating engineering soil properties from MCPT data.

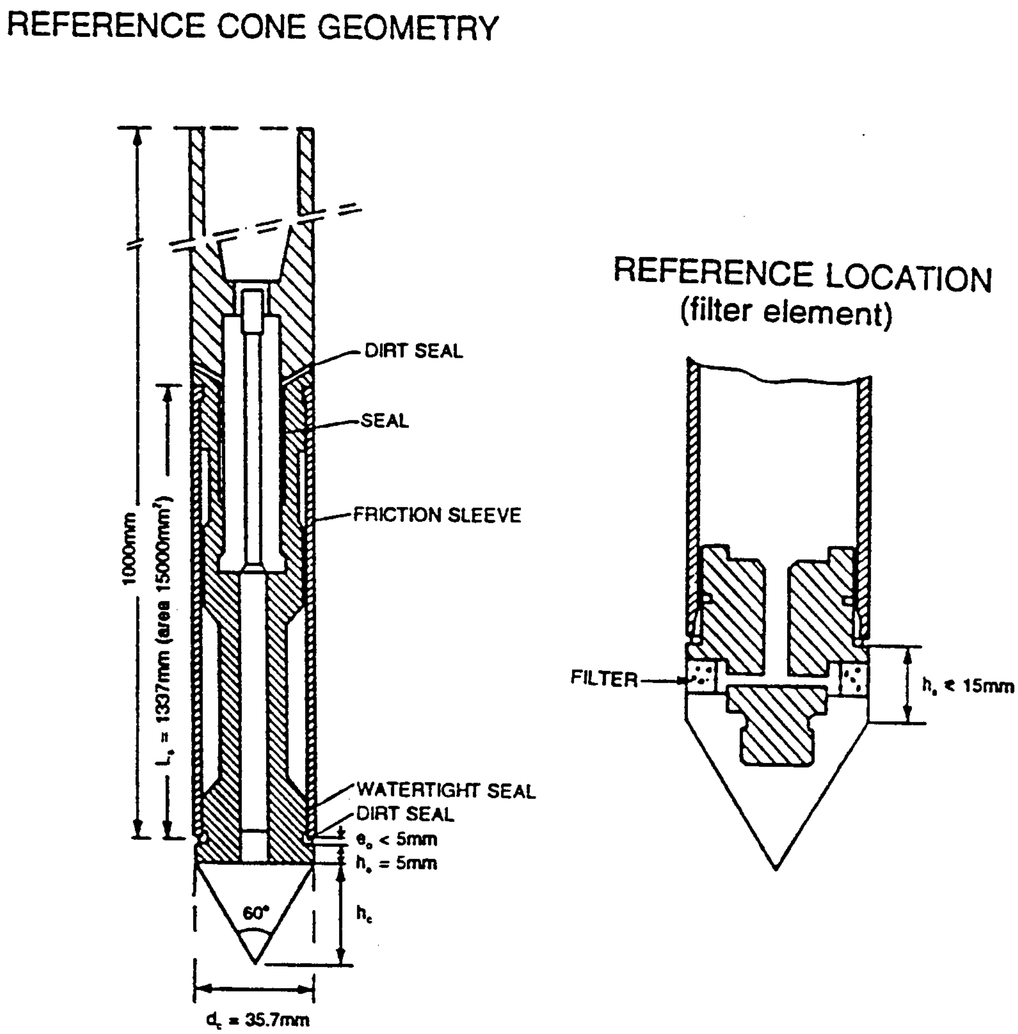


Figure 1.1
Reference cone (ISSMFE 1977, 1989

SCOPE

Two miniature cone penetrometers will be procured (after a comprehensive evaluation of commercially available miniature cone penetrometers) according to specifications set as a result of the special testing needs peculiar to the soil conditions in Louisiana. A jacking system will be procured for advancing and retracting the penetrometer from the ground and also for providing the necessary reaction needed for penetration. A data acquisition system will be developed using DT-2801, data acquisition board for real time monitoring of cone resistance and sleeve friction and also to display the data on a computer screen in graphic form. In-situ Tests will be conducted to evaluate the applicability of MQSC for road and highway design and construction control in Louisiana. Five representative sites encompassing a wide range of sandy, silty, and clayey soils will be selected. Tests will be performed at two compacted embankments and three natural grade soils. The MCPT penetration profiles will be compared with CPT profiles obtained with a standard 1.55 in.^{2} (10 cm^2) Fugro-cone penetrometer and with a 2.33 in.^{2} (15 cm^2) Fugro-cone penetrometer to evaluate scale effects and to assess the accuracy of MCPT results. Calibration Chamber Tests will be performed using the MQSC penetrometer to develop correlations for evaluating engineering soil properties from MCPT data. Three different but representative soils prevalent in Louisiana will be used.

FIELD TESTING PROGRAM

3.1. Introduction

The purpose of the field testing program was to verify and analyze the adequacy of a lightweight prototype, MQSC system for field conditions related to road engineering, i.e., natural grade soil classification and embankment construction control. The possible existence of scale effect between the MQSC and the reference cone penetrometer data had to be investigated to study the applicability of MQSC data to soil classification and basic parameter charts developed from the reference cone penetrometer data. Another goal of this research was to study the field performance of a 2.33 in.^{2} (15 cm^2) cross-section cone penetrometer in use by the Louisiana Transportation Research Center (LTRC) compared to the reference cone penetrometer.

The field testing program implemented to examine the scale effect involved a comparative study of field performance between the MQSC, the 2.33 in.^{2} (15 cm^2) cone, and the reference cone penetrometer. The reference cone penetrometer was used as the basis for comparison of performance. Statistical correlations between the MQSC, the 2.33 in.^{2} (15 cm^2) cone and the reference cone penetrometer data were developed, enabling the immediate use of the field data obtained using the MQSC for soil profiling and strength parameter evaluation

3.2. Equipment and Procedures

A dedicated heavy vehicle is generally required to perform efficient CPT soundings. The four major components of such a vehicle are the specialized chassis and hydraulic system, penetrometer probes, depth encoder, and data acquisition hardware and software. The prototype miniature cone penetrometer system was implemented *[18]* in the Research Vehicle for Geotechnical In-situ Testing and Support (REVEGITS, Figure 3.1) which is a sister vehicle to the Louisiana Electric Cone Penetrometer System (LECOPS). REVEGITS is a 20-ton (18,144-kg) all wheel drive vehicle that incorporates state-of-the-art technology for in-situ subsurface soil exploration for civil and geo-environmental engineering purposes. It is powered by a caterpillar HP diesel engine on a model G-744 6x6 chassis, modified by Zeligson Company of Tulsa, Oklahoma. The CPT system is housed in a specially fabricated van-body mounted vehicle with sufficient reaction weight and off-road maneuverability to carry out in-situ geotechnical investigations. The van-body (subframe and cabin) and the hydraulic pump (maximum pressure of 300 bar (30 MPa) at 1000 rpm), which is driven by the power take-off (PTO) of the truck, were fabricated to the specifications set by the authors at A.P. van den Berg, B.V. of the

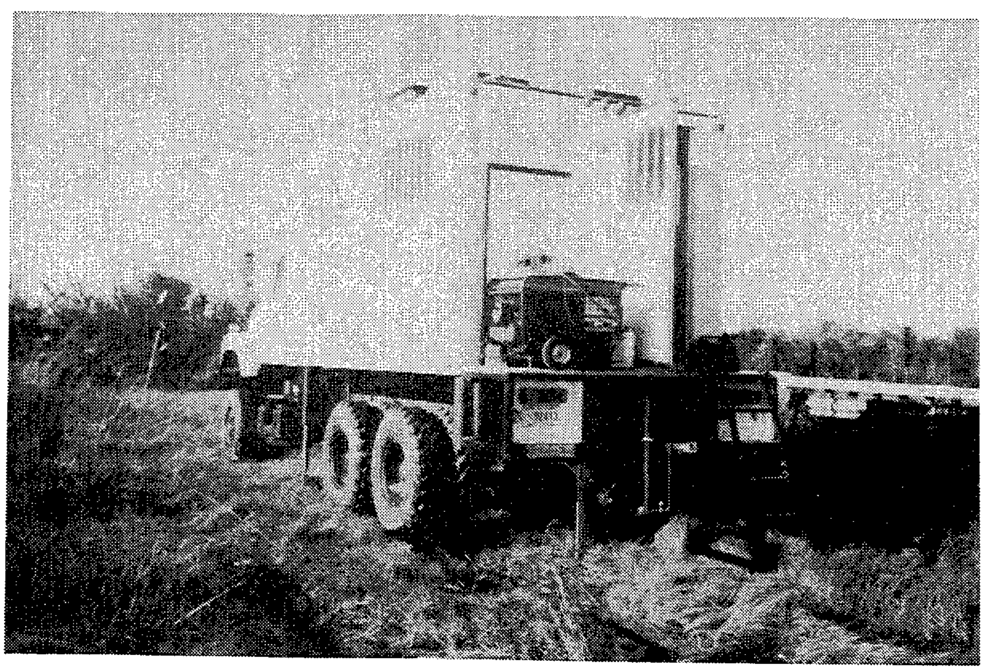


Figure 3.1
General view of the LSU REVEGITS

Netherlands and attached to the chassis in the U.S.

3.2.1. Miniature Cone Penetration and Depth Measurement System

The hydraulic system for miniature cone penetration consists of three jacks mounted outside in front of the vehicle. The two outer jacks lower a transverse steel plate to the ground and raise the front of the truck, providing the reaction force during penetration. The center jack is the push jack. It has a stroke of 5.91 in. (15 cm) and can deliver a push rate of 0.79 in./sec (2 cm/sec). The chucking system applies a grabbing force to the rod and advances or extracts the cone from the soil depending on the selected direction of movement. A friction based force transfer system between the clamping device and the sounding rods allows for the safe manipulation of the rods from any location on the rod not requiring a predetermined clamping point (see Figure 3.2). For Louisiana soils, general sounding depths in the range of 32.8 ft. (10 m) can be reached using a coiled high tensile strength seamless steel rod.

The MQSC depth measurement system consists of a displacement transducer manufactured by Fugro-McClelland. The displacement transducer works via a bi-directional optical incremental shaft.encoder driven by a pulley. For every meter of displacement, 1000 output pulses are generated (for the standard rate of 0.79 in/sec (2 cm/sec)., only 50 pulses are required). These pulses are 90 degree phase shifted and have a square waveform on Transistor Transistor Logic (TTL) levels which are polled by the computer through the digital I/O channel. A data conversion of all incoming signals is performed every time the TTL level is high.

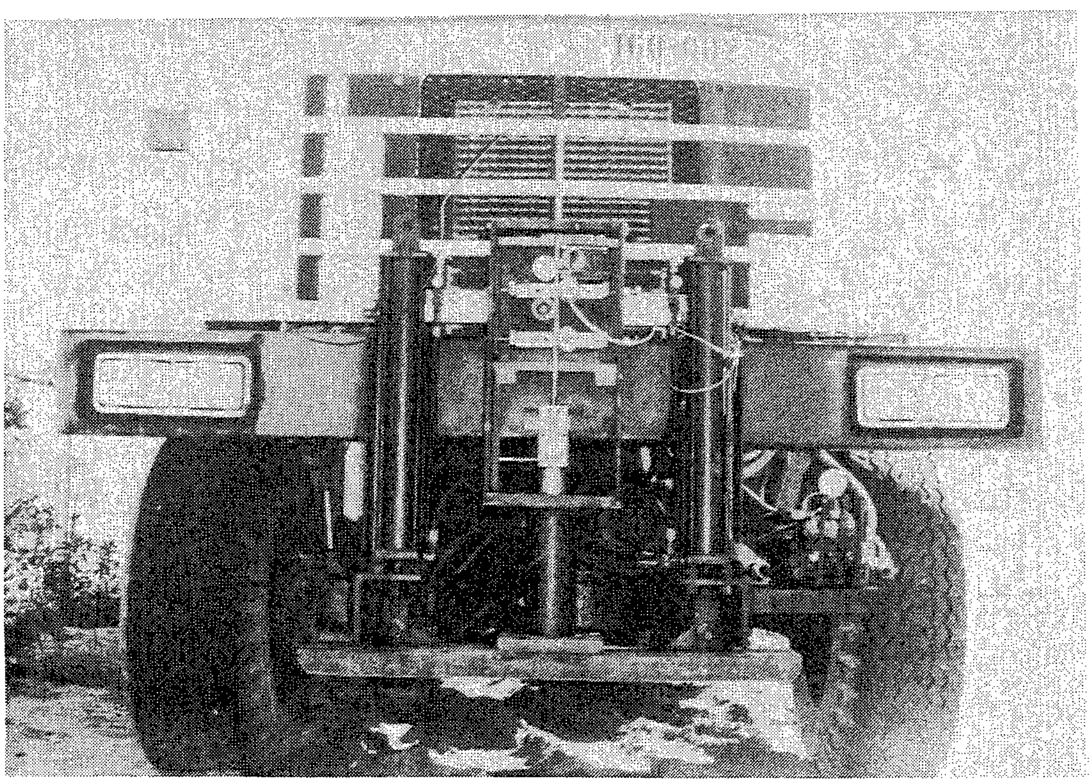


Figure 3.2
Prototype miniature cone penetrometer system

3.2.2. The MQSC Data Acquisition and Reduction System

The electronic data acquisition hardware consists of a Penetrometer Control Unit - Modular (PCU-M) for signal conditioning manufactured by Fugro-McClelland of the Netherlands, a 486 micro computer with a high resolution screen (for real time graphics processing), a 540 Megabyte internal hard disk drive for storage, an analog to digital conversion and digital I/O board, and the Data Translation DT-2801A, an analog to digital conversion and digital I/O board. Signals coming from the cone penetrometer are amplified (by an amplifier and conditional box) before they are transmitted to the DT-2801A for conversion. A Data reduction hardware, i.e. a plotter or a printer, is connected to the system to produce off-line high quality output.

The data acquisition software system for MCPT is programmed in the Turbo Pascal version 4.0 language environment by Borland International and the HALO '88 graphics library by Media Cybernetics. The HALO '88 graphics library allows the graphic portion of the program to be developed in the device-independent environment. That is, any changes in the output device configurations usually require only the installation of the appropriate device driver and minimal re-programming. The software acquires and appends data into a file at 0.79 in. (2 cm) intervals and also displays the data on a computer screen in graphic form plotted in real time. The software is also capable of printing and plotting off-line high quality copy output directly on the job site. Figure 3.3 depicts the MQSC data acquisition and reduction hardware. The operational principle of the software is to continuously poll for a rod-down signal supplied by the PCU-M through one of the digital I/O channels of the DT-2801A. When the condition is met, the TTL depth pulse will be polled from another digital I/O channel. When the TTL logic is true, the program will trigger an analog to digital conversion on all the corresponding channels of the penetrometer. After each set of analog to digital conversions, the measured voltages are scaled to their physical representations and plotted in real time on the gas plasma display of the computer. This allows the operator to obtain the reading at the actual unit of measurement of choice. The subtraction and scaling of the voltages from the tip and combined tip and sleeve load cells are performed by the software. This is in contrast to the larger 1.55 and 2.33 in.^{2} (10 and 15 cm^2) penetrometers that perform the subtraction in the PCU-M.

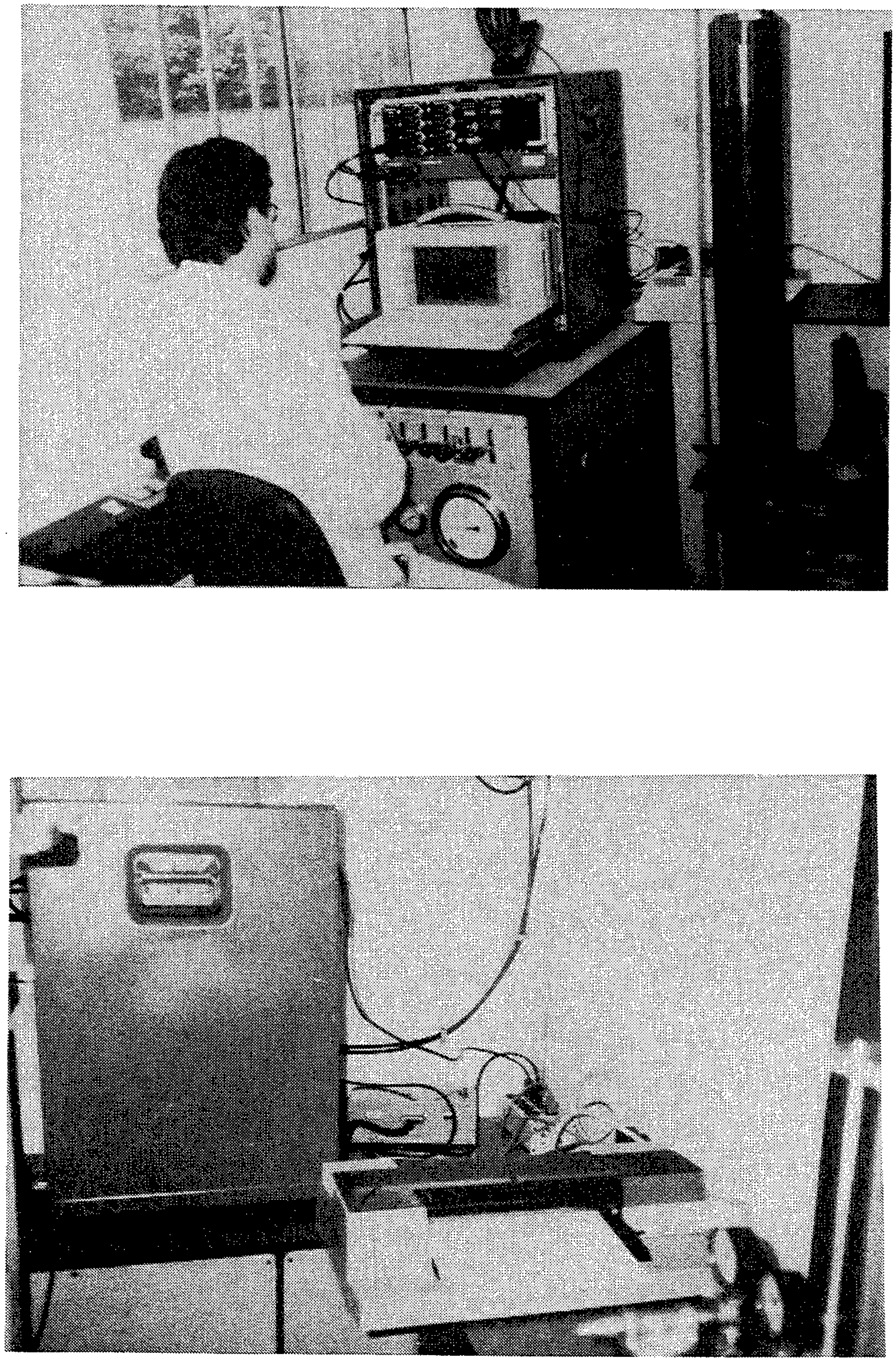


Figure 3.3
REVEGITS data acquisition and reduction system

3.2.3. Cone Penetrometers

Penetrometers (Figure 3.4) used by REVEGITS include the standard friction cone, single and dual piezocones, conductivity cone, and the seismic cone fabricated by Fugro-McClelland Engineers B.V. of the Netherlands. All penetrometers are equipped with an inclinometer to ensure vertical insertion of the probe into the ground during sounding operations. A miniature cone penetrometer was also acquired (from GEOCOGNETICS, Houston, Texas) for the prototype MQSC system. A brief description of all cones is given below:

(1) The Reference Friction Cone Penetrometer is a 1.41 in. (35.7 mm) nominal diameter Fugro-cone penetrometer cross-sectional area of 1.55 in.^{2} (10 cm^2), friction sleeve area of 23.25 in.^{2} (150 cm^2), and a cone apex angle of 60 degrees. It measures cone and local side friction resistance. The Fugro cone, as usually referred to in the geotechnical community, is a subtraction type probe with unequal end area ratio of 0.45, built-in amplifiers, and an incorporated slope sensor.

(2a) The MQSC Penetrometer developed and used in a proprietary manner by Fugro-McClelland is a 0.20 in.^{2} (1.27 cm^2) cross-sectional area, subtraction type penetrometer that can be viewed as a scaled down version of the full size 10 cm^2 reference Fugro penetrometer. It has a friction sleeve area of 3.9 in.^{2} (25.14 cm^2), cone apex angle of 60 degrees, and an unequal end area ratio of 0.75. It measures cone resistance and combined cone and local side friction resistance. This miniature cone penetrometer was never available commercially, and Fugro-McClelland has since discontinued its production. A 0.31 in.^{2} (2 cm^2) miniature cone penetrometer described below was developed as the alternative.

(2b) The miniature cone penetrometer fabricated by "GEOCOGNETICS," Houston, Texas, on contract to LTRC has a projected cone area of 0.31 in.^{2} (2 cm^2), friction sleeve area of 6.2 in.^{2} (40 cm^2),and a cone apex angle of 60 degrees. It measures cone resistance and combined cone and local side friction resistance.

(3) The 2.33 in.^{2} (15 cm^2) Cone Penetrometer is a 0.17 in. (43.7 mm) nominal diameter Fugro cone penetrometer (cross-sectional area of 2.33 in.^{2} (15 cm^2)), with a friction sleeve area of 31 in.^{2} (200 cm^2), and a cone apex angle of 60 degrees. It measures cone and local side friction resistance. The Fugro-cone is a subtraction type with unequal end area ratio of 0.59, built-in amplifiers, and an incorporated slope sensor.

(4) The 2.33 in.^{2} (15 cm^2) Dual-Piezocone Penetrometer is similar to the 2.33 in.^{2} (15

cm^2) Fugro friction penetrometer with the additional capability of measuring pore pressure behind the friction sleeve and the pore pressure measurement at the cone tip.

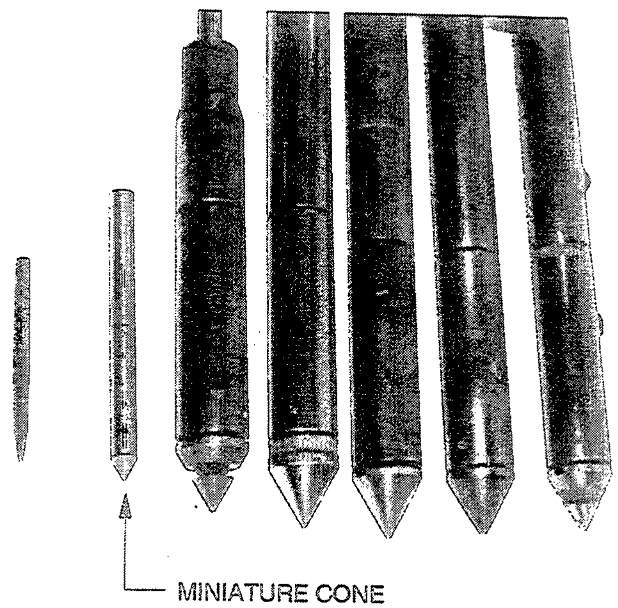


Figure 3.4
Cone penetrometers

3.3. Scale Effects

In order to investigate scale effects between different size cone penetrometers, in-situ tests were performed using 2.33 in.^{2}, 1.55 in.^{2}, and 0.2 in.^{2} (15 cm^2, 10 cm^2, and 1.27 cm^2) Fugro cone penetrometers *[19]*. The MCPT's were performed using the MQSC truck developed by Fugro-McClelland Engineers. Soundings, generally 32.81 ft. (10 m) deep, were performed in three natural grade and two highway embankment soils. At each chosen location, five soundings were executed with the reference and 2.33 in.^{2} (15 cm^2) cone penetrometers, and three soundings were performed with the MQSC. Limitations in the MQSC thrust system capacity prevented some of the soundings from reaching the 32.81 ft. (10 m) depth at Highland Road (natural ground). Continuous penetration tests spaced two meters from each other and at a penetration rate of 0.79 in. (2 cm/sec). were a standard in the field test program, in accordance with the International Reference Test Procedure *[20], [21]*.

The sites evaluated in this field test program are classified as natural grade soils and embankments. Five representative sites encompassing a wide range of sandy, silty, and clayey soils were selected for the field test program. Two compacted embankments and three natural grade soils were investigated.

(1) Big River Industries is a recent alluvium soil with a predominance of inorganic clays of high plasticity. Some pockets of organic silt clays of low plasticity and organic clays of medium to high plasticity are also present in the soil profile. It is located on U.S. 190, approximately 20 miles (32.2 km) west of Baton Rouge, Louisiana. The ground water was observed at 3.28 ft. (1 m).

(2) Highland Road (Natural Ground) is also a recent alluvium soil with a predominance of silty clays to clays, clayey silts to silty clays, and inorganic clays of high plasticity. It is located near the intersection of I-10 and Highland Road South, in Baton Rouge, Louisiana.

(3) Iowa (Natural Ground) is a terrace soil with a predominance of sandy silts to clayey silts, clayey silts to silty clays, sands to silty sands, sands, and of silty clays and clay pockets. It is located in the vicinity of the intersection of I-10 and the U.S. 165 North, close to Iowa, Louisiana.

(4) Highland Road (Embankment) is a silty clay/clayey silt embankment. It is located on the median section of the embankment at the intersection of I-10 and Highland Road, in Baton Rouge, Louisiana.

(5) McElroy Swamp (Embankment) test section is predominantly sands to silty sands, silty clays, and clays pumped from the Mississippi River into the highway grade line in the mid-60's. It is located at mile 191 on I-10, median section, approximately 40 miles (64.4 km)

Southeast of Baton Rouge, Louisiana.

Figures 3.5 to 3.7 display plots of the MQSC, reference, and 2.33 in.^{2} (15 cm^2)cone penetrometers data plotted sequentially as an adjoining continuous sounding. In this fictitious profile, the averaged CPT data from each site investigated are displayed sequentially, creating a profile (fictitious) of the pooled data with an imaginary sounding depth and a substantial size data file to be used in computational analysis. Figures 3.8 to 3.9 present plots of cone resistance ratios (0.20 in.^{2}/ 1.55 in.^{2} and 2.33 in.^{2}/1.55 in.^{2} (1.27 cm^2/10 cm^2 and 15 cm^2/10 cm^2)), and local side friction resistance ratios (0.20 in.^{2}/ 1.55 in.^{2} and 2.33 in.^{2}/1.55 in.^{2} (1.27 cm^2/10 cm^2 and 15 cm^2/10 cm^2)) of the pooled data. There is no significant difference between the cone resistance and friction ratio obtained by the 2.33 in.^{2} (15 cm^2) and the 1.55 in.^{2} (10 cm^2) reference penetrometer. Scale effects can be observed when comparing the miniature cone penetration profiles with the profiles obtained using the c and the 1.55 in.^{2} (10 cm^2) penetrometers. Statistical evaluation of these results are described in the next section.

3.4 Statistical Evaluation of the Field Testing Program

Results of a simple linear regression *[19], [22], [23]* applied to the MQSC and 2.33 in.^{2} (15 cm^2) penetrometer data versus the reference penetrometer data are summarized below. The units of q_c, f_s are in kg/cm^2, and the unit of R_f is in percentage. (Since cone penetrometer technology originated in Europe, kg/cm^2 is the standard unit for sounding profiles; therefore, the figures presented here will use these units. However, 1 kg/cm^2 is approximately equal to 1.0241 ton/ft.^{2}.) The regression analysis directed to the evaluation of local side friction resistance and friction ratio performance of the MQSC considered the data divided into two soil ranges: (1) f_{s1}, R_{f1} - soils with q_c smaller or equal to 81.9 ton/ft.^{2} (80 kg/cm^2), and (2) f_{s2}, R_{f2} - soils with q_c higher than 81.9 ton/ft.^{2} (80 kg/cm^2).

MQSC versus Reference Penetrometer:

$$q_{c(1.55\ in.2\ (10\ cm2))} = 0.359 + 0.861 * q_{c(MQSC)} \quad (1)$$

$$f_{s1(1.55\ in.2\ (10\ cm2))} = 0.234 + 0.836 * f_{s1(MQSC)} \quad (2)$$

$$f_{s2(1.55\ in.2\ (10\ cm2))} = 0.497 + 1.115 * f_{s2(MQSC)} \quad (3)$$

$$R_{f1(1.55\ in.2\ (10\ cm2))} = 3.196 + 0.511 * R_{f1(MQSC)} \quad (4)$$

$$R_{f2(1.55\ in.2\ (10\ cm2))} = 1.270 + 1.330 * R_{f2(MQSC)} \quad (5)$$

15 cm^2 versus Reference Penetrometer:

$$q_{c(1.55\ in.2\ (10\ cm2))} = -0.729 + 1.055 * q_{c(2.33\ in.2\ (15\ cm2))} \quad (6)$$

$$f_{s(1.55\ in.2\ (10\ cm2))} = 0.0197 + 1.150 * f_{s(2.33\ in.2\ (15\ cm2))} \quad (7)$$

$$R_{f(1.55\ in.2\ (10\ cm2))} = 0.812 + 0.931 * R_{f(2.33\ in.2\ (15\ cm2))} \quad (8)$$

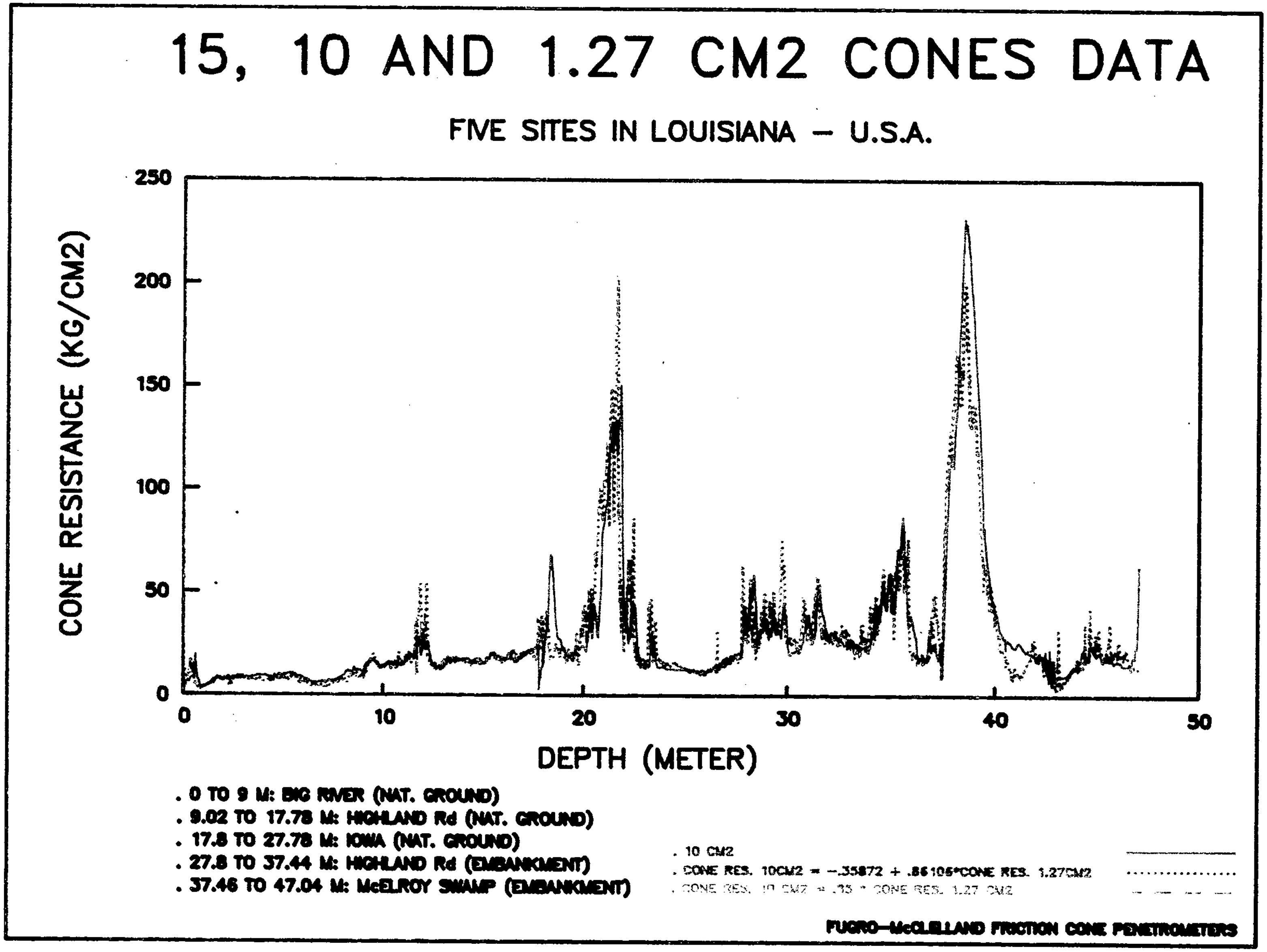


Figure 3.5
Averaged cone resistance (pooled profiles)

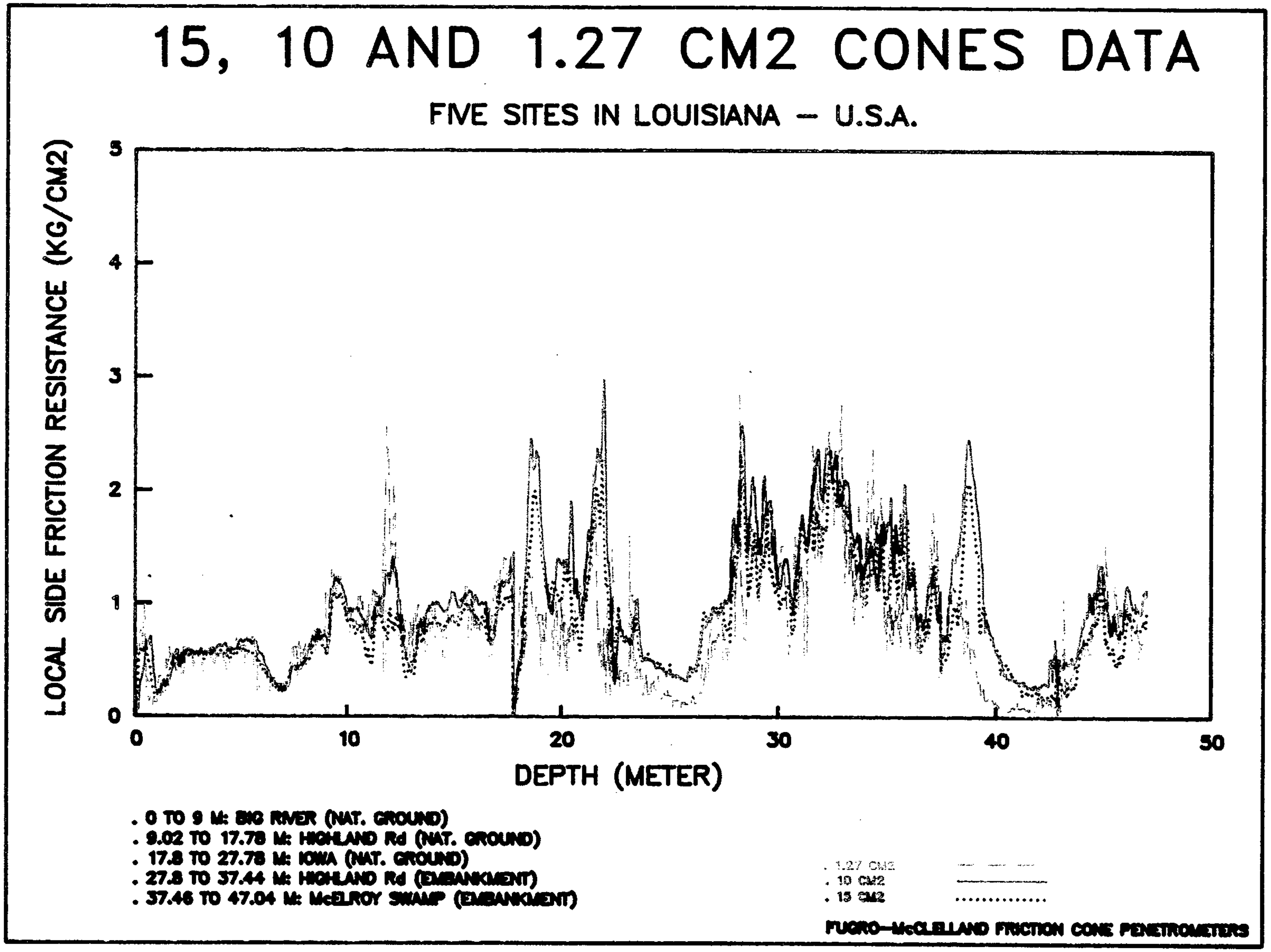


Figure 3.6
Averaged local side friction resistance (pooled profiles)

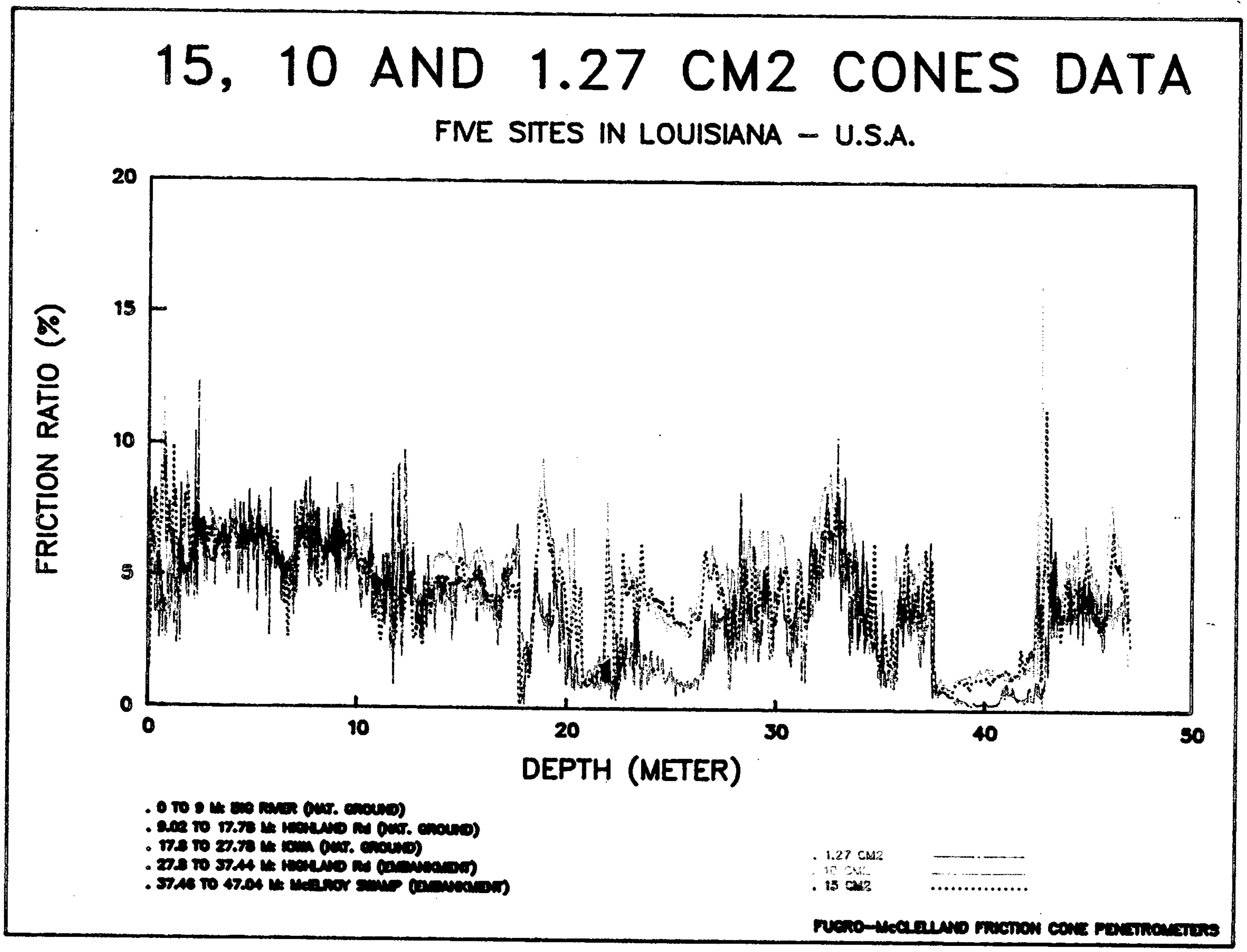


Figure 3.7
Averaged friction ratio (pooled profiles)

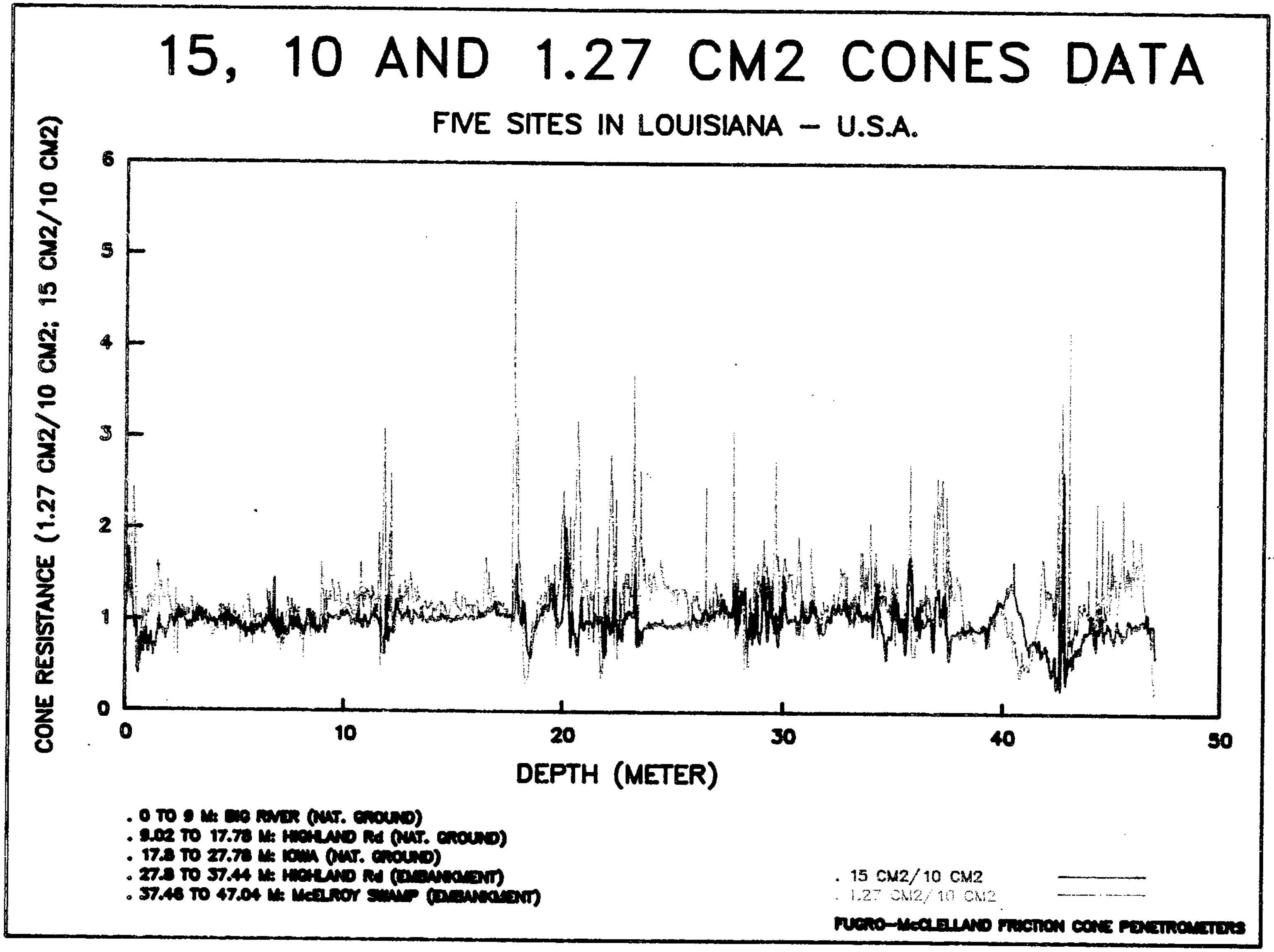


Figure 3.8
Cone resistance ratios (pooled profiles)

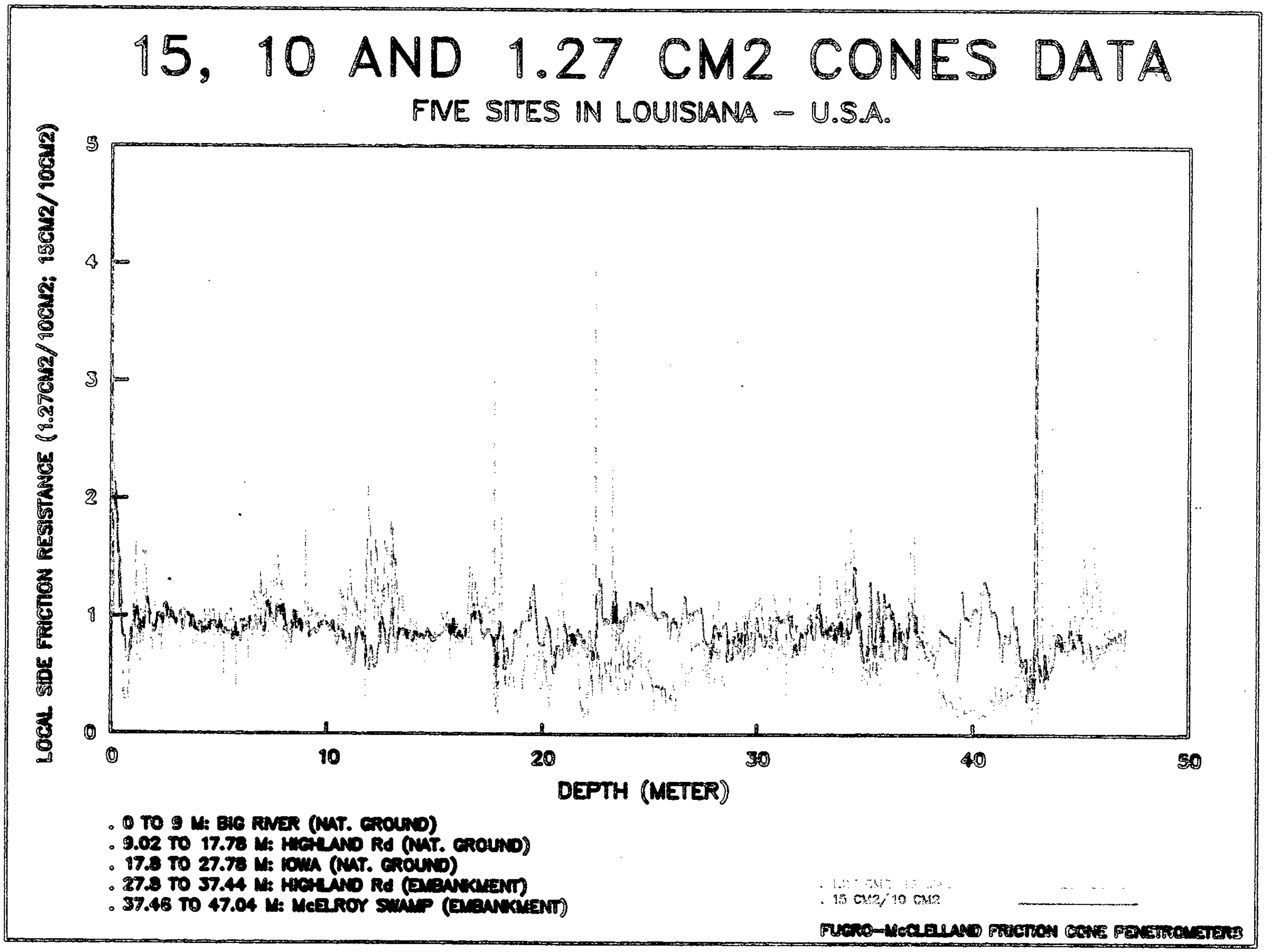


Figure 3.9
Local side friction resistance ratios (pooled profiles)

Statistical analysis of the data after field testing *[19], [22], [23]* indicated that cone resistance's mean and standard deviation decrease with an increase in cone dimensions. The decrease in standard deviation with increase in cone diameter indicates higher capability of a miniature cone to capture more of the soil variability than a large dimension cone penetrometer. Figures 3.10 to 3.14 depict the MQSC and 15 cm^2 penetrometer's cone resistance, local side friction resistance, and friction ratio corrected via linear regression equations. As depicted in Figure 3.11, a multiplication factor of 0.85 can be used effectively to correct the MQSC cone resistance to obtain the reference penetrometer cone resistance (i.e., $q_{c(1.55\ in.2\ (10\ cm2))} = 0.85 * q_{c(MQSC)}$). A division factor of 0.85, as shown in Figure 3.13, also can be used to correct the 2.33 $in.^2$ (15 cm^2) penetrometer local side friction resistance to obtain the reference local side friction resistance (i.e., $f_{s(1.55\ in.2\ (10\ cm2))} = (1/0.85) * f_{s(2.33\ in.2\ (15\ cm2))}$). The MQSC's local side friction resistance and friction ratio should be corrected via linear regression equations by considering two ranges of cone resistance: (1) soils with q_c equal or smaller than 81.9 $ton/ft.^2$ (80 kg/cm^2), and (2) soil with q_c higher than 81.9 $ton/ft.^2$ (80 kg/cm^2). No significant correction is necessary for cross-correlating cone resistance of the reference and 2.33 $in.^2$ (15 cm^2) cross-section penetrometers.

3.5. Field Testing of the Prototype Miniature Cone Penetrometer System

The implementation of the prototype miniature cone penetrometer was tested and verified by comparing its penetration profiles with those obtained with the 2.33 $in.^2$ (15 cm^2) cross-sectional area cone penetrometer developed by Fugro-McClelland Engineers B.V., the Netherlands. The 2.33 $in.^2$ (15 cm^2) cone penetrometer has a friction sleeve area of 31.0 $in.^2$ (200 cm^2) and a 60 degrees cone apex angle. Penetration tests were performed using the 2.33 $in.^2$ (15 cm^2) Fugro cone penetrometer and a 0.31 $in.^2$ (2 cm^2) miniature cone penetrometer (by Geocognetics, Houston, Texas) at a site near the intersection of Highland Road and Interstate-10 (LA SR-42) in Baton Rouge, Louisiana. The Highland Road embankment is a silty clay/clayey silt embankment. Four MCPT's were performed 3.28 ft. (1 m) apart (square layout) and a 2.33 $in.^2$ (15 cm^2) CPT was performed at the center. The MCPT's were performed to a depth of 22.97 feet (7 meters) and the 2.33 $in.^2$ (15 cm^2) CPT was performed to a depth of 32.8 feet (10 meters). The penetration profiles of the four MCPT's are compared with the 2.33 $in.^2$ (15 cm^2) cone penetration profiles in Figures 3.15 through 3.18. In the first test (MCPT1), the tip load cell indicated a large zero offset after the penetrometer was extracted from the ground. It can be observed from the cone resistance profile of MCPT1 that at depths greater than 13.1 ft. (4 m) the cone resistance begins to drop(approximately linear) and reaches zero at a depth of 24.6 ft. (7.5 m) (where the test was stopped). The CPT and three other MCPT profiles did not indicate such a decrease in cone resistance below

13.1 ft. (4 m) nor were any zero offsets observed. The reason for this erroneous behavior is unexplained, and hence the MCPT1 profile is not used for interpretation. The MCPT2 profile indicated an almost uniform cone resistance, sleeve friction, and friction ratio below 3.28 ft. (1 meter) up to a depth of 24.6 feet (7.5 meters). The MCPT, however, does indicate larger fluctuation in the cone resistance, sleeve friction, and friction ratio values and a higher standard deviation because of its capability to capture local soil characteristics and thin layer properties in comparison to large size penetrometers, which globalize the soil properties around it. In MCPT3 the cone resistance profile, between 11.5 ft. (3.5 m) and 12.9 ft. (4.25 m) showed a stiff layer distinctively different from the other tests. Hence, between a depth of 11.5 and 12.9 ft. (3.5 and 4.25 m) MCPT3 data is ignored (only the average between MCPT2 and MCPT4 are taken). Figure 3.19 compares the 2.33 in.^{2} (15 cm^2) CPT profiles with the average penetration profile (average of MCPT2, MCPT3, and MCPT4) transformed using the regression equations developed by de Lima, 1990; de Lima and Tumay, 1991for the 0.20 in.^{2} (1.27 cm^2) miniature cone penetrometer *[19], [22]*. It can be seen that the average MCPT (transformed) profile compares well with the 2.33 in.^{2} (15 cm^2) CPT profile. This indicates that there is no significant scale effect between the 0.31 in.^{2} (2 cm^2) and the 0.20 in.^{2} (1.27 cm^2) miniature cone penetrometers. This is similar to the previous observation made by comparing the 2.33 in.^{2} (15 cm^2) CPT profile with the 1.55 in.^{2} (10 cm^2) CPT profile. MCPT data must be converted to the reference cone penetrometer data using regression equations 1 through 5, before using any evaluation method developed for the reference cone penetrometer. Any classification chart or interpretation method developed for the reference cone penetrometer may then be used to evaluate the 0.31 in.^{2} (2 cm^2) miniature cone penetrometer test data. Some of the available soil classification charts developed for the reference cone penetrometer are given in the appendix.

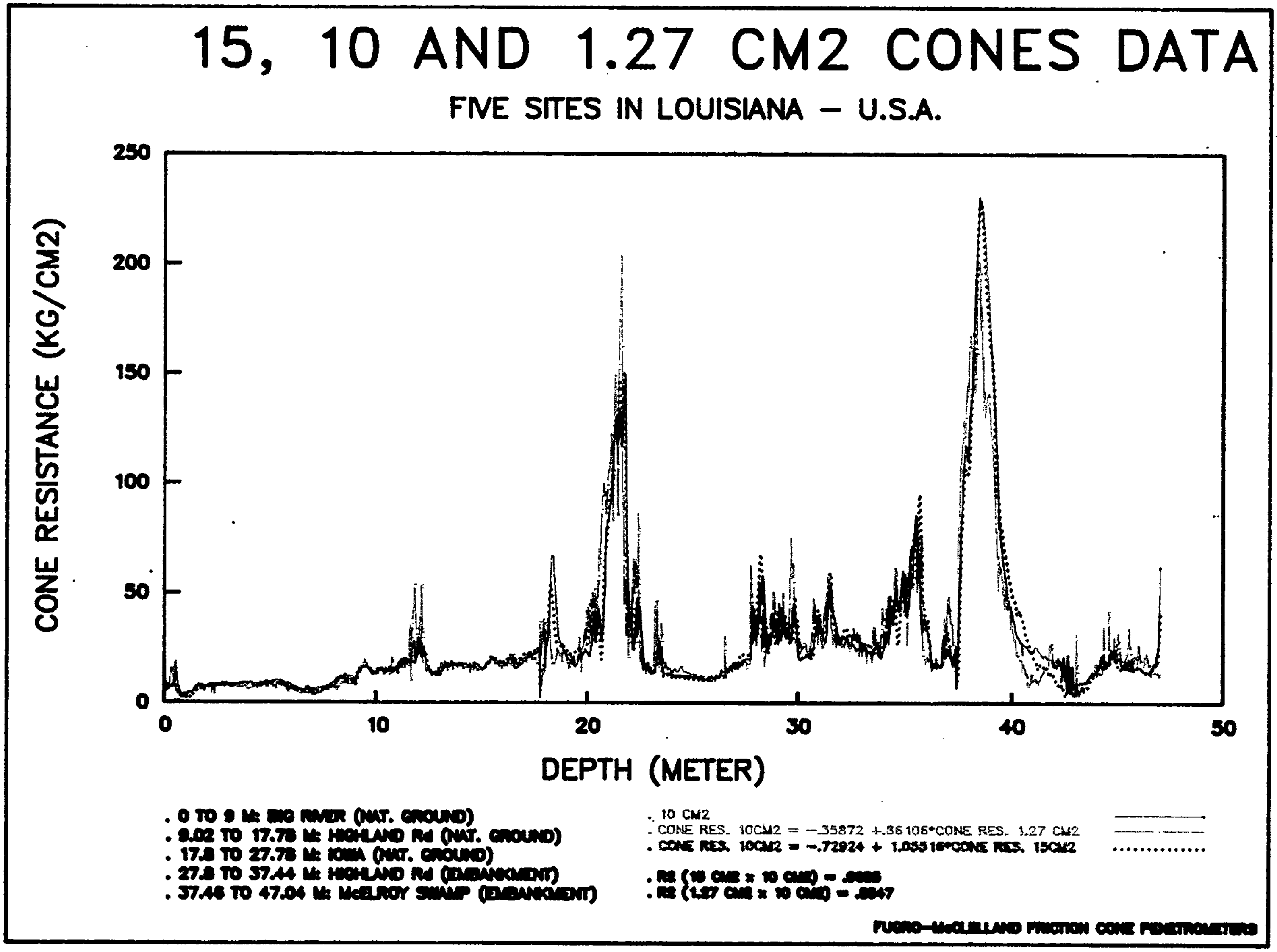


Figure 3.10
Cone resistance (pooled profiles): MQSC and 15 cm^2 penetrometer data corrected via simple linear regression equations

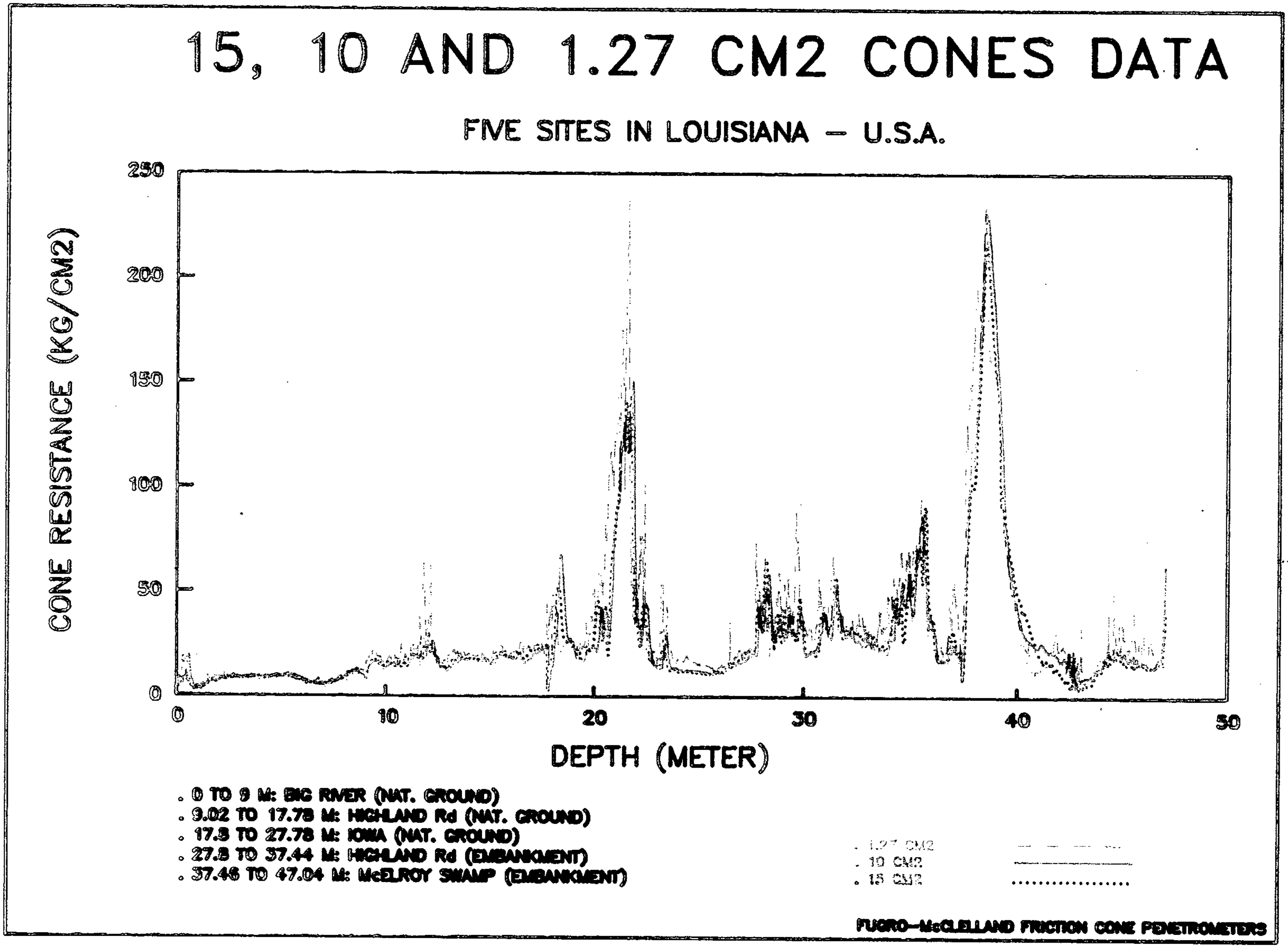


Figure 3.11
Cone resistance (pooled profiles): MQSC data corrected via simple linear regression equation and correction factor

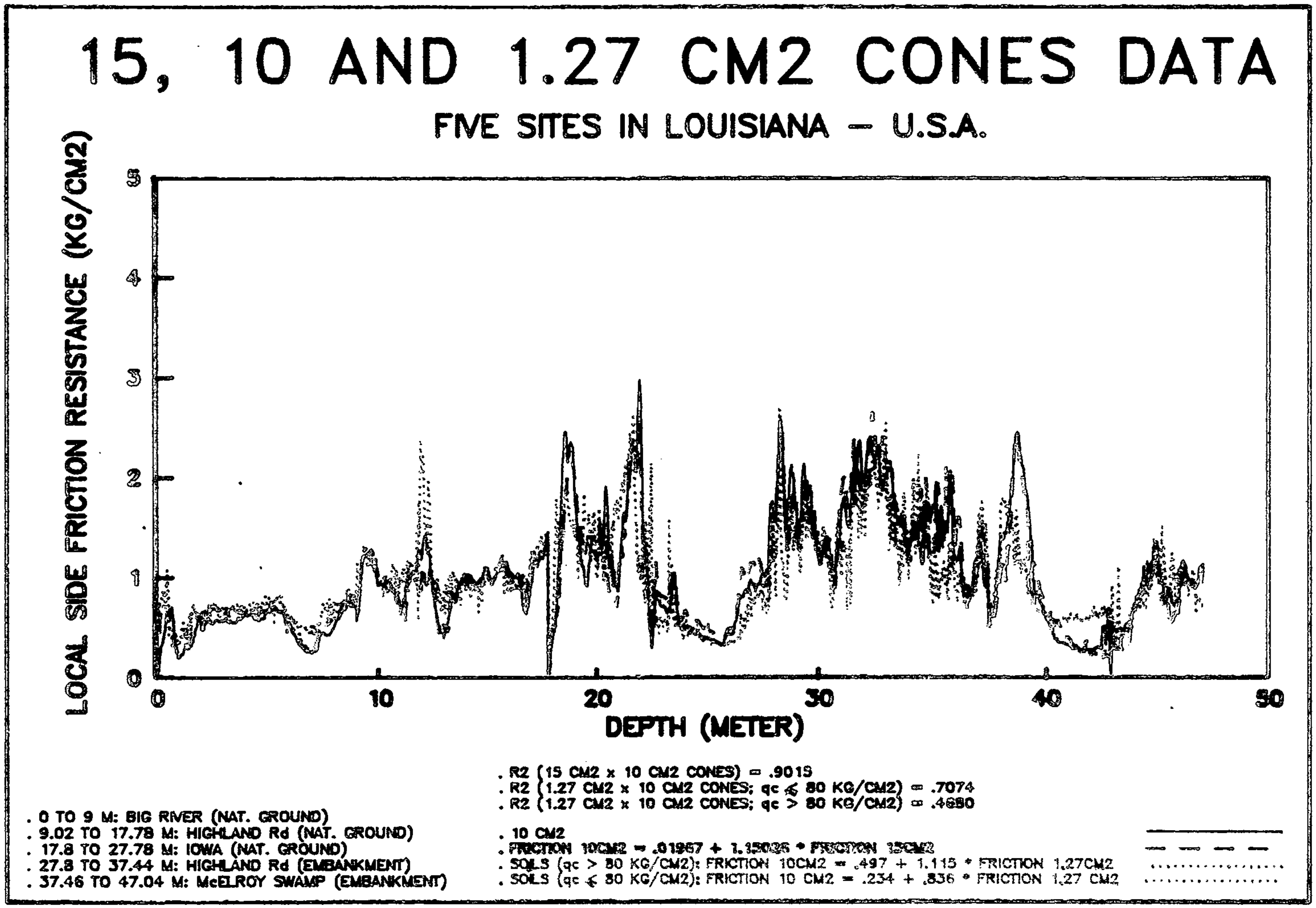


Figure 3.12
Local side friction resistance (pooled profiles): MQSC and 15 cm^2 data corrected via simple linear regression equations

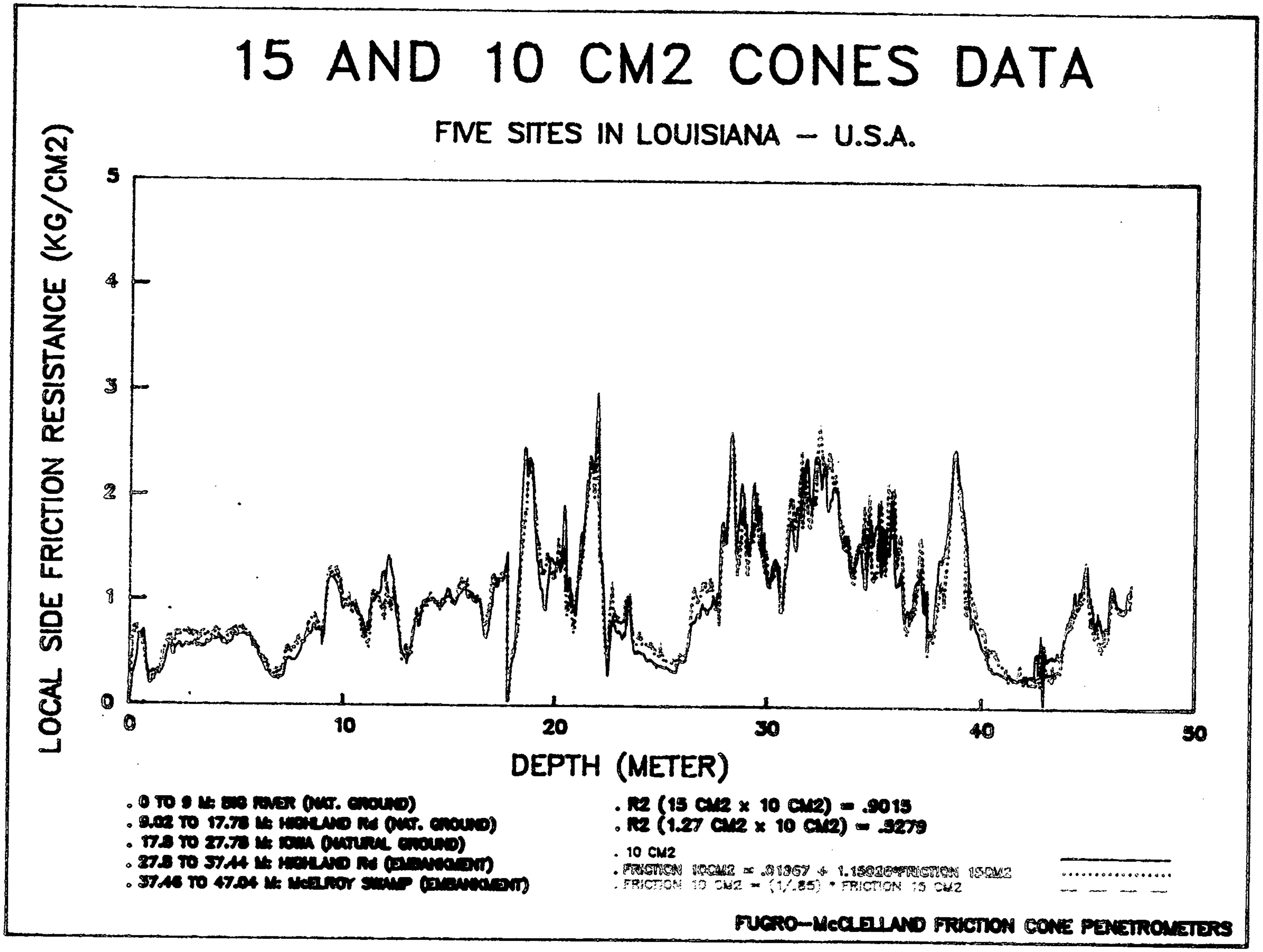


Figure 3.13
Local side friction resistance (pooled profiles): 15 cm² penetrometer data corrected via simple linear regression equation and correction factor

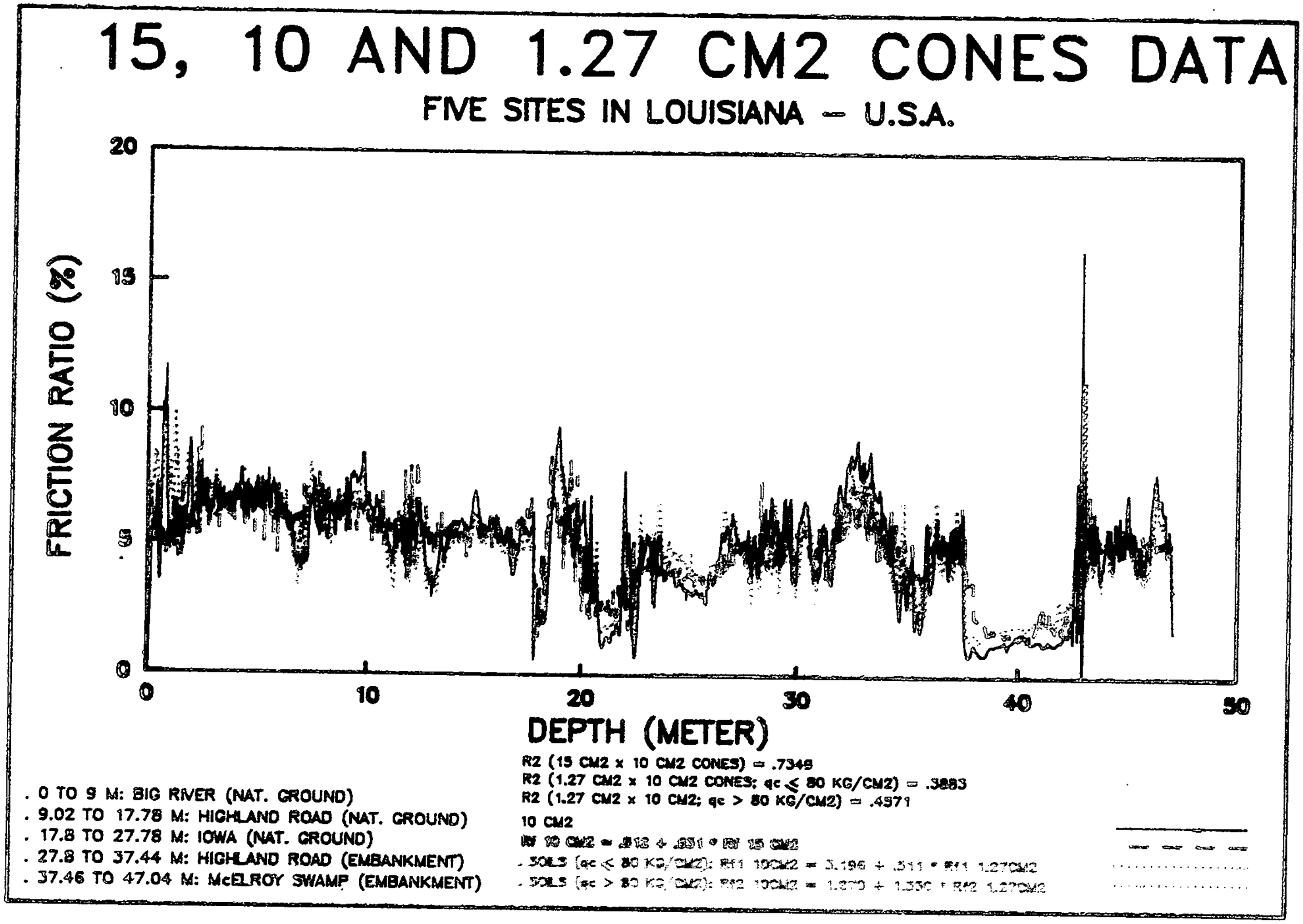


Figure 3.14

Friction ratio (pooled profiles): MQSC 15 cm² penetrometer data corrected via simple linear regression equations

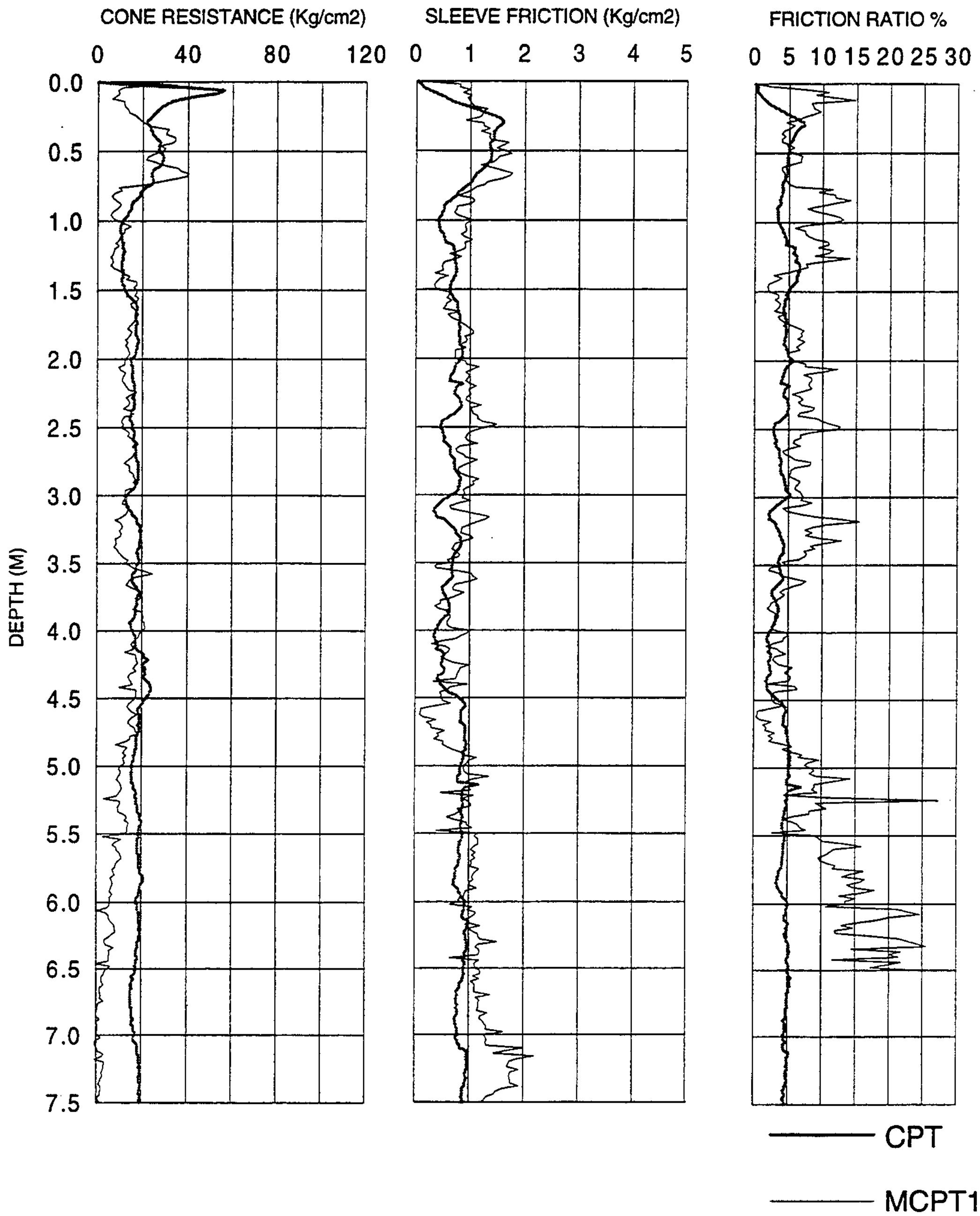


Figure 3.15
Comparison of profiles between 15 cm² CPT

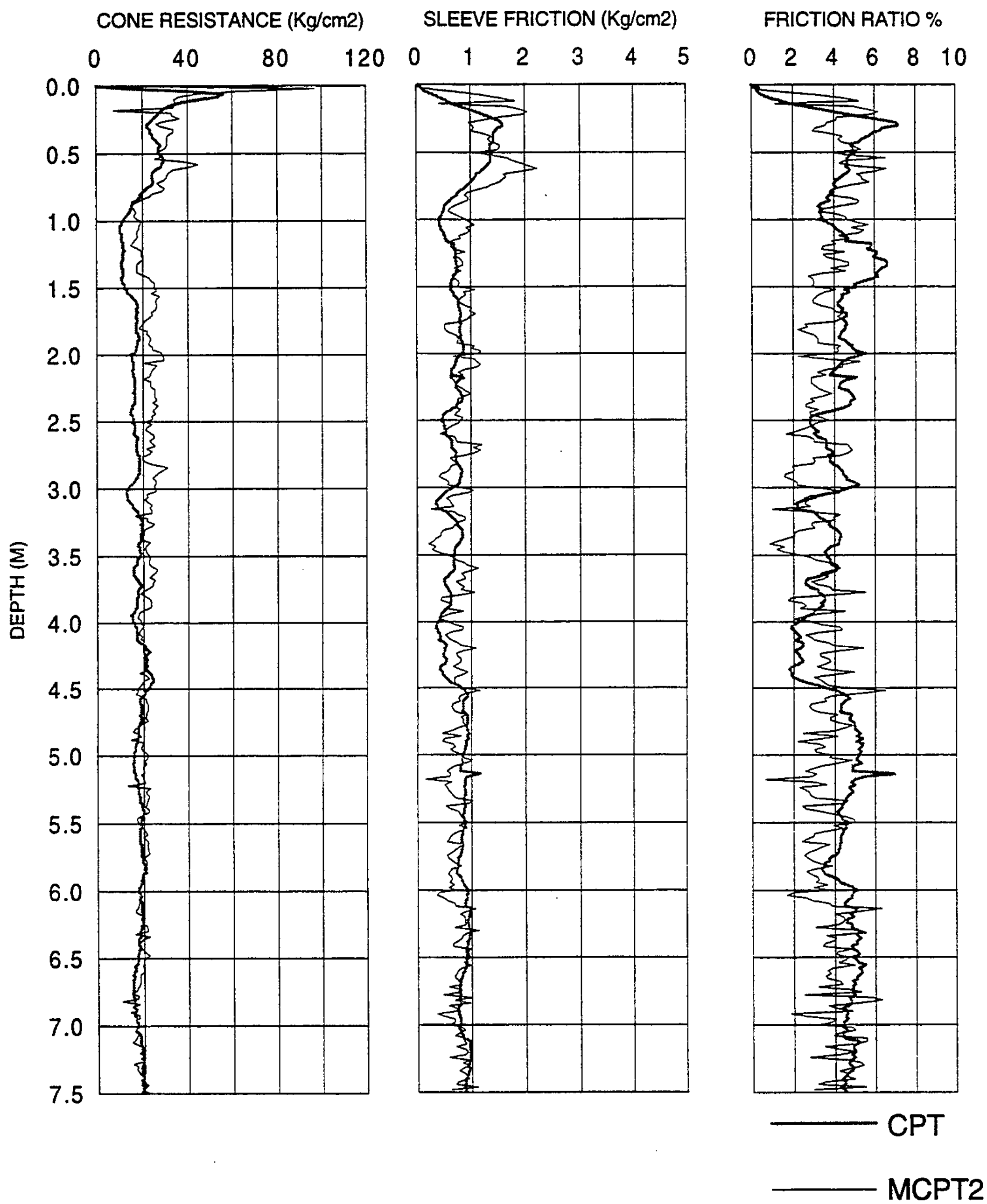


Figure 3.16
Comparison of profiles between 15 cm² CPT data MCPT2

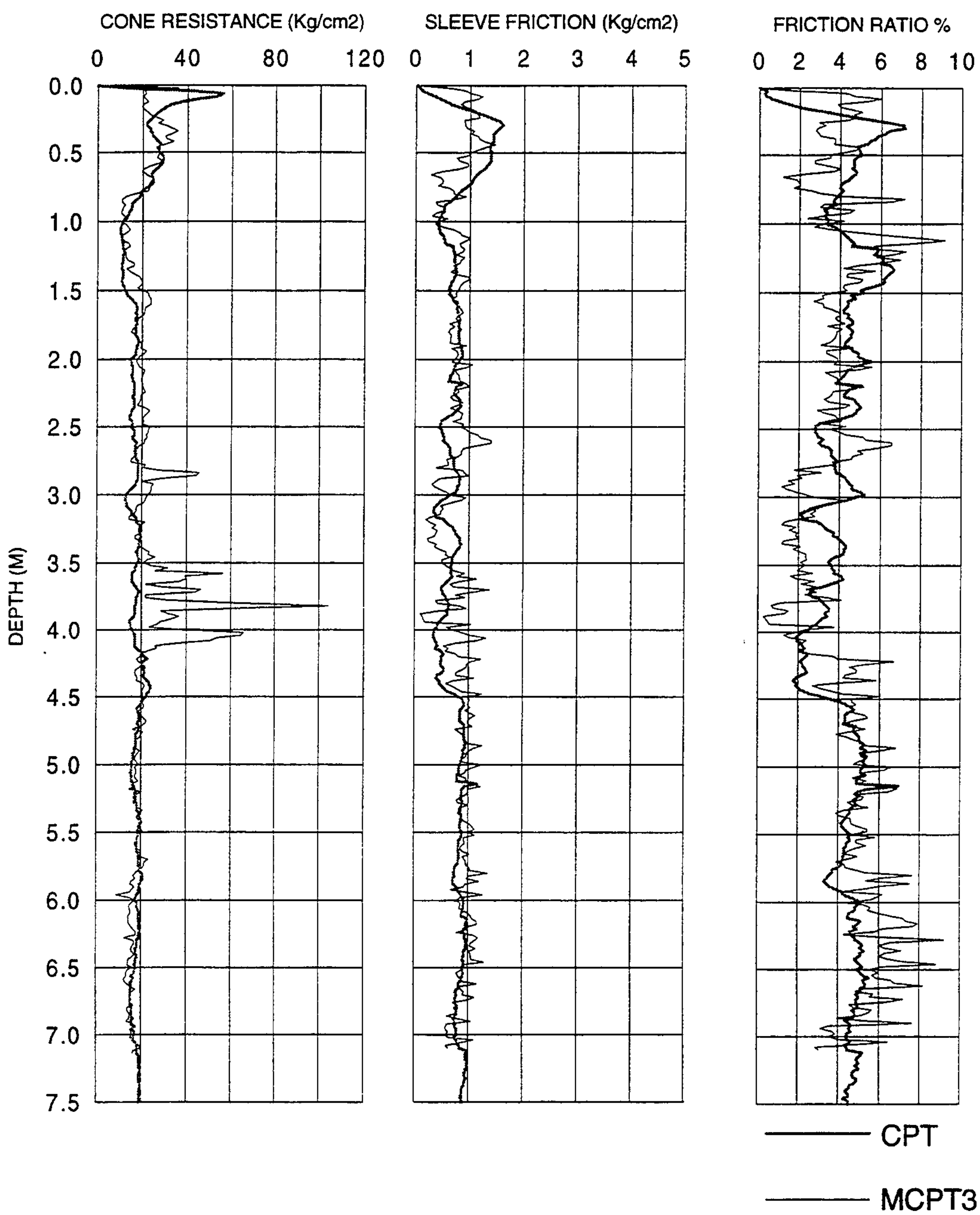


Figure 3.17
Comparison of profiles between 15 cm² CPT data MCPT3

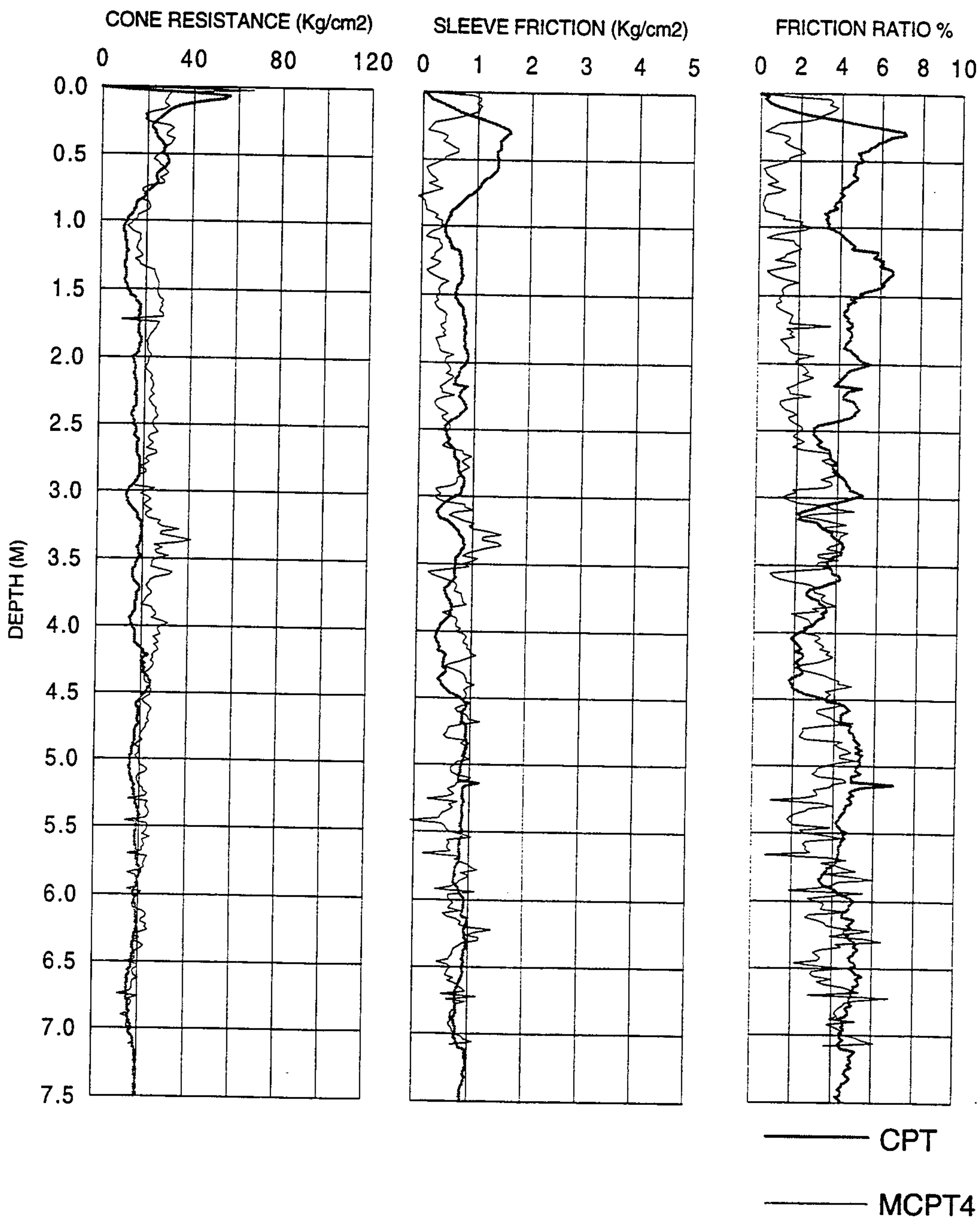


Figure 3.18
Comparison of profiles between 15 cm² CPT data MCPT3

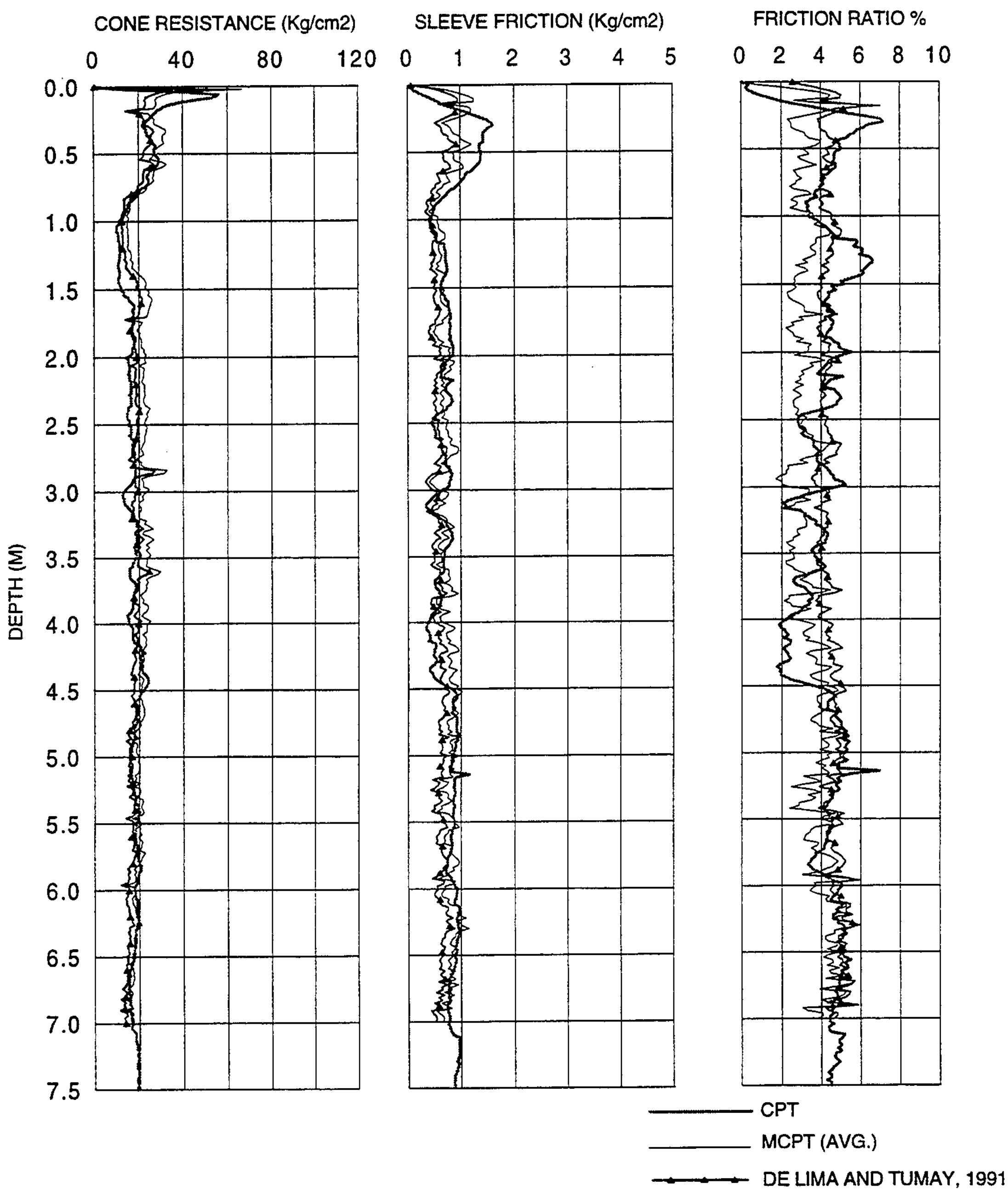


Figure 3.19
Comparison of profiles between 15 cm² CPT, average MCPT, and the equations of de Lima and Tumay, 1991

LABORATORY TESTING PROGRAM

4.1. Introduction

Conventional laboratory tests routinely performed in road engineering for characterizing and defining engineering properties of natural grade soils and embankments can be grouped as follows: Sieve Analysis, Atterberg Limits, Compaction (Standard Proctor and Modified), California Bearing Ratio (CBR), and strength and compressibility tests (unconfined, triaxial, and odometer). Soil parameters obtained from the Proctor and CBR tests executed at the Standard and Modified AASHTO compactive efforts are among those most frequently used in road engineering for design and construction control. Each level of compactive effort is typically related to the required compaction characteristics of natural grade soils and embankments of roads. Consequently, parameters such as CBR, optimum water content (w_{opt}), and maximum dry density (γ_{dmax}) become design and construction control characteristic elements in road engineering.

Initially, the laboratory test phase of the present research envisioned the development of a preliminary test program on selected types of soils to establish the basis for calibrating a miniature cone penetrometer, the MQSC, to be employed in construction control of road embankments in Louisiana. Calibration is to be understood in the context of generating correlations between cone penetration resistance measurements, (cone tip resistance, q_c and friction ratio, R_f), and soil parameters for road engineering design and construction control of embankments, such as soil compressibility modulus, soil dry density (γ_{dry}), and CBR.

4.2. Calibration Chamber Testing

Calibration chamber tests are performed to calibrate in-situ testing devices such as cone penetrometers (including the piezocone), dilatometers, pressuremeters, piezovane shearing devices, etc. and also to conduct tests on model foundations and ground anchors. Field calibration tests have numerous disadvantages because of soil inhomogeneity and uncertainties regarding the magnitude of in-situ stresses and stress history of the deposit. It is extremely difficult and nearly impossible to obtain undisturbed samples from the field to determine reference soil parameters. Moreover, the influence of any particular parameter (stress history, stiffness, void ratio, compressibility, soil fabric, and anisotropy) cannot be studied by varying it independently in the field. Laboratory calibration tests have definite advantages since homogeneous, reproducible, and instrumented soil specimens subjected to a known stress history can be prepared and tested under controlled boundary conditions. Various parametric studies can also be conducted.

Rigid-wall test pits that impose a boundary condition of zero lateral strain have been used in the past by Melzer (1968) and Tcheng (1966)*[24], [25]*. Rigid-wall calibration chambers have serious disadvantages; it is not possible to control lateral stresses. It was also pointed out by Holden in 1971 that very large specimens with a diameter ratio (calibration chamber diameter/cone diameter) of 200 would be required to minimize the influence of rigid boundary effects on the test results. It was concluded that a smaller flexible-wall calibration chamber could simulate stresses (and strains) similar to those approaching field conditions *[1]* .

4.3. Louisiana State University Calibration Chamber System (LSU/CALCHAS)

The Louisiana State University Calibration Chamber System (LSU/CALCHAS) (Figure 4.1) designed by de Lima (1990), de Lima and Tumay (1991), and Tumay and de Lima (1992) consists of a calibration chamber, a panel of controls (data acquisition/control system), a hydraulics and chucking system, a penetration depth measurement system, and the cone penetrometers *[19], [22],[23]*.

4.3.1. Double Wall Flexible Chamber

The LSU/CALCHAS is a double walled flexible chamber (Figure 4.2) that can house specimens 20.67 in. (525 mm) in diameter and 32.09 in. (815 mm) high. The two cylindrical shells made of stainless steel 304 plates are 0.25 in. (6.35 mm) thick. The internal diameter of the inner and outer shells is 22.05 in. and 22.83 in. (560 mm and 580 mm), respectively, and the shells are 35.83 in. (910 mm) high. The shells are designed to withstand a maximum pressure of 208.85 psi (1440 kN/m^2). The sample top plate (20.67 in. (525 mm) in diameter and 1.5 in. (38.1 mm) high) is made of 6061 T-6 aluminum. The bottom plate is similar to the top plate and rests on a 20.67 in. (525 mm) diameter piston. The rubber membrane around the specimen is sealed (water tight) around the top and bottom plates using four "O" rings. The top plate transfers the vertical thrust of the piston on the specimen into the chamber top lid. The top lid made of 6061 T-6 aluminum is 25 in. (635 mm) in diameter and 1.5 in. (38.1 mm) high. The top lid and top plate provide for tests to be conducted at three different locations in the specimen, as shown in Figure 4.3. These holes are sealed by adapters during specimen reconsolidation against back pressure. The adapters are designed to permit PCPT under back pressure. The top lid is connected to the piston cell ring using twelve stainless steel 304 rods (0.5 in. (12.7 mm) in diameter). This acts as a self-reacting frame when the specimen is stressed and also provides reaction for the push jack during cone penetration. The inner cell (annular space between the specimen and the inner shell) and outer cell (space between the inner and outer shells) are filled with deaired water by water lines connected to the top lid.

4.3.2. Piston Cell

The piston cell is a double walled cylinder (16.9 in. (430 mm) high) made of stainless steel with the same diameter as the cylindrical shells. The inside cell space of the piston is kept free for instrumentation (pore pressure transducers and cables). A hollow piston shaft, 2.5 in. (63.5 mm) diameter and 16 in. (406 mm) long, is attached to the bottom piston plate. The chamber bottom plate, 25 in. (635 mm) in diameter and 1.5 in. (38.1 mm) thick, carries a piston guide to allow smooth vertical movement of the piston. The annular space between the inner and outer cell walls and some grooves at the bottom of the piston plate are filled with deaired water through a port in the walls of the piston cell. This piston is raised by pressurizing the water in the piston cell with an air-water system. The piston cell ring, the piston cell, and the chamber bottom plate are kept together by twelve stainless steel 304 rods (0.5 in. (12.7 mm) in diameter). A linear varying displacement transducer (LVDT) connected to the piston shaft measures the vertical deformation of the specimen. The lateral volume change of the specimen is measured by an air-water system.

4.3.3. Panel of Controls

The operation of the LSU/CALCHAS is servo controlled. The panel of controls (Figure 4.4) is equipped with two Fairchild model T-5700 electro-pneumatic transducers for independent control of the vertical and horizontal stresses by digital to analog (D/A) signals sent from an IBM personal computer through a data acquisition board (Data Translation DT-2801A). The panel also has pressure regulators for manual control of the stresses. There are five Sen Sym ST2000 pressure transducers and four Marsh process gauges to measure the stresses (inner and outer sell pressures, vertical stress, and back pressure) in the specimen. The panel is equipped with air-water systems to pressurize the piston and sample cells as well as to saturate specimens under back pressure.

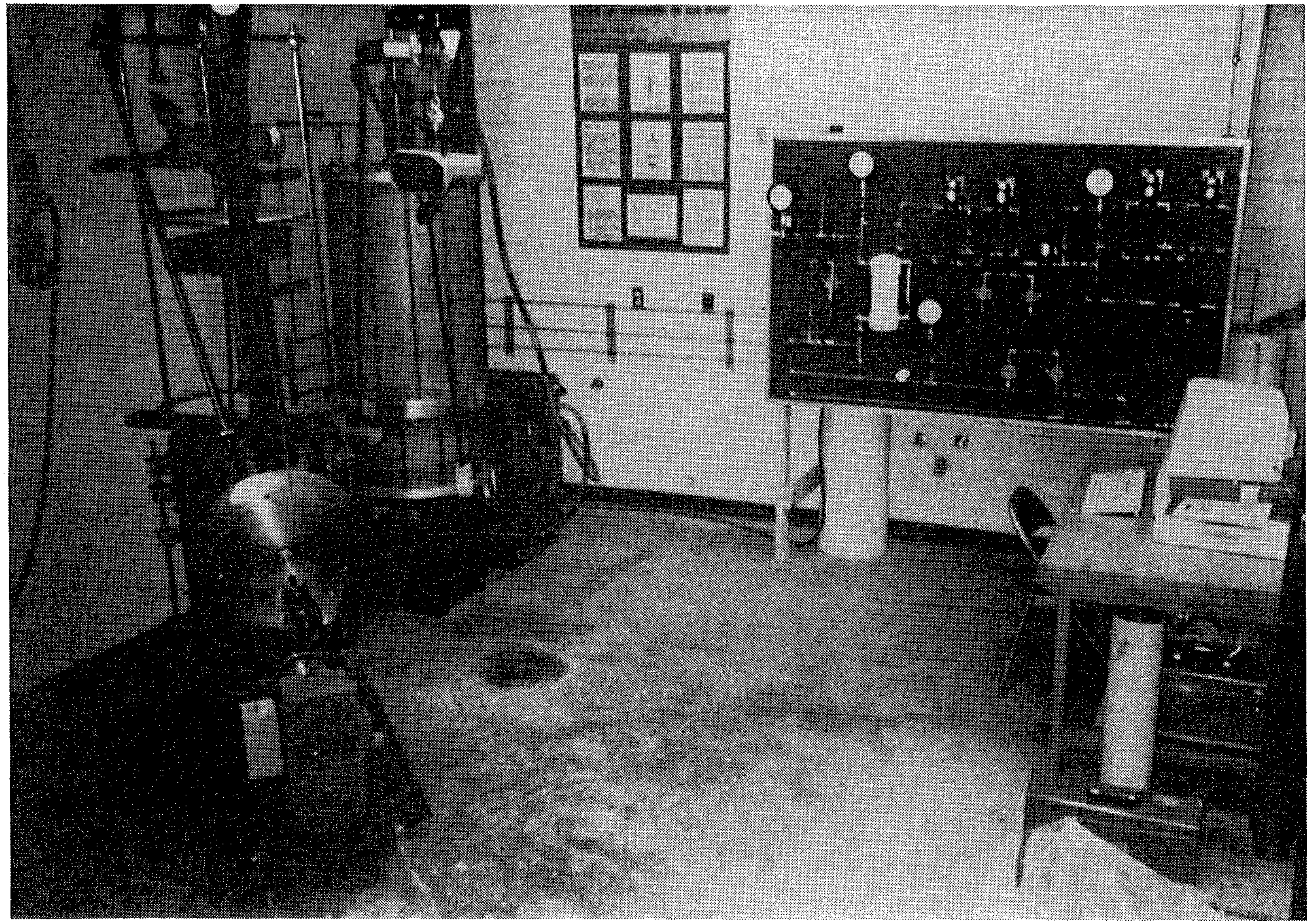


Figure 4.1
Louisiana State University Calibration Chamber System (LSU CALCHAS) (Tumay and de Lima, 1992)

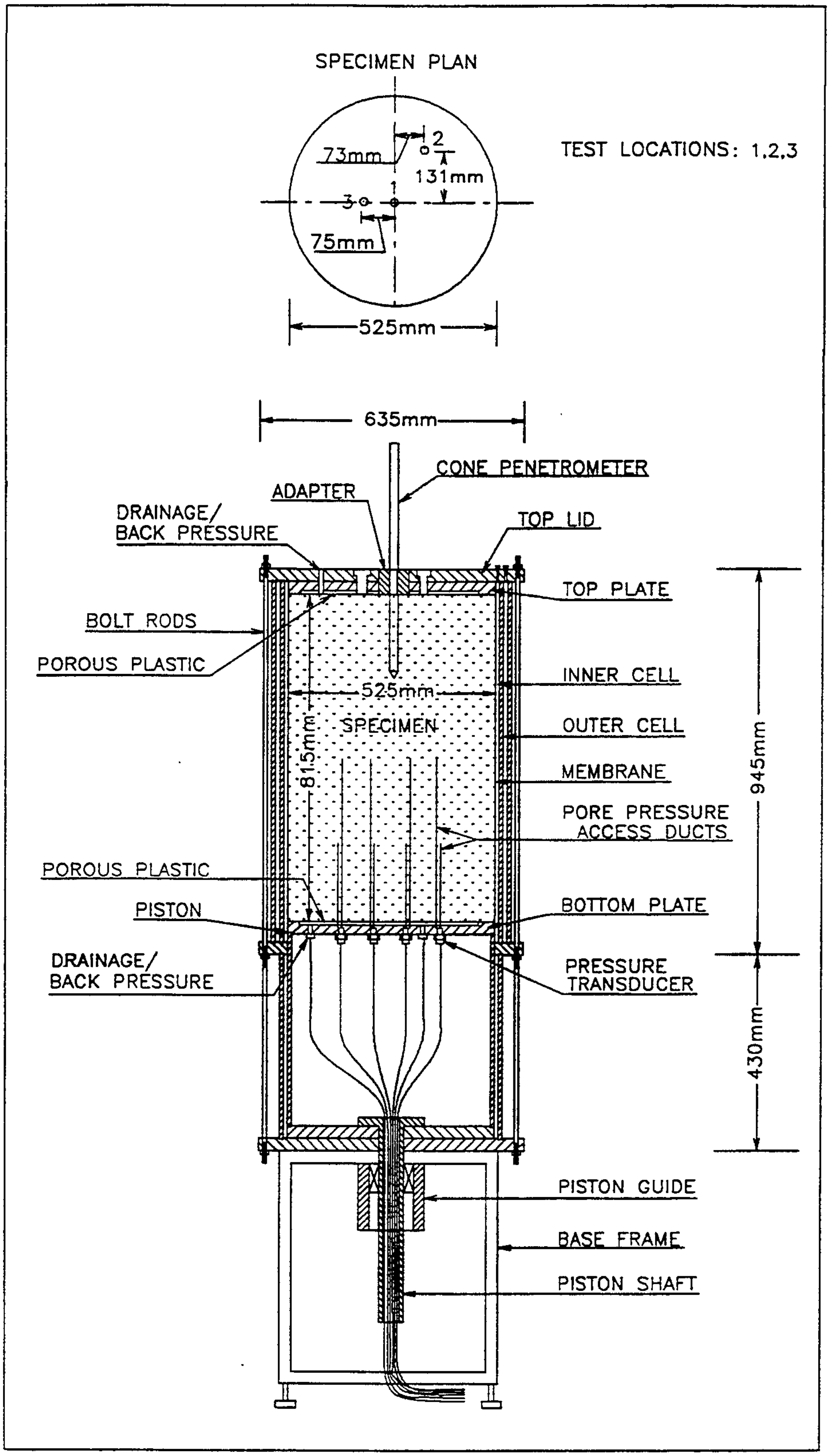


Figure 4.2
Schematic of the flexible double wall calibration chamber

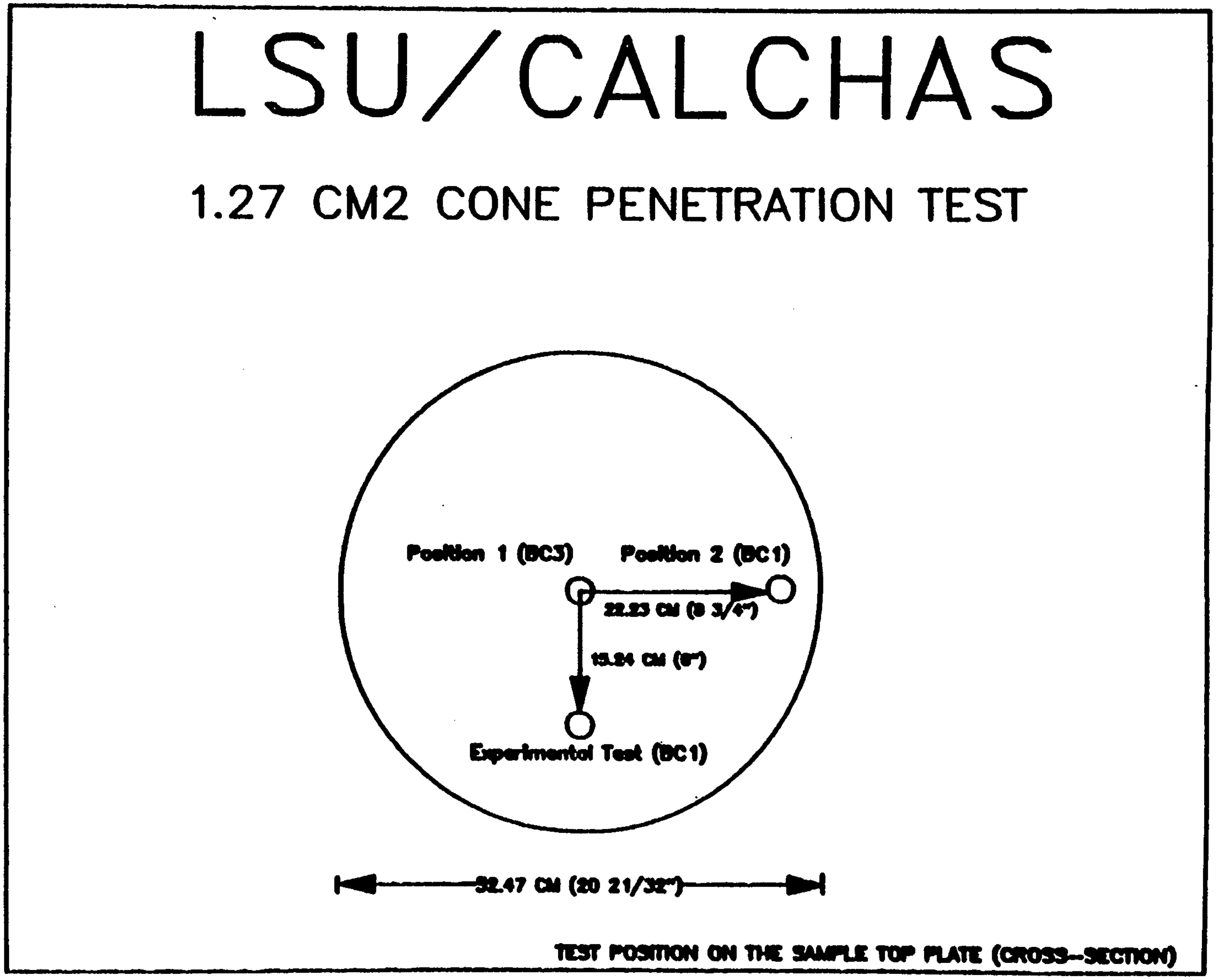


Figure 4.3
Position of penetration tests performed in the soil sample

4.3.4. Hydraulic and Chucking (Push/Pull) System

The hydraulic system used for cone penetration consists of a dual piston, double acting hydraulic jack mounted on a collapsible frame (Figure 4.5). The frame is 84.25 in. (2140 mm) high in its extended state. It is mounted on top of the top lid of the chamber and allows for penetrating the specimen in a single stroke of 25.20 in. (640 mm) or less (for stage testing). Such a single stroke continuous penetration is desirable especially in saturated cohesive specimens where stress relaxation and pore pressure dissipation can occur during a pause between strokes. The push jack is equipped with a chucking system to grab the push rods during penetration and extraction of the cone penetrometer. The hydraulic push jack system is designed to test the 0.16 in.^{2} (1 cm^2) miniature quasi-static cone penetrometer with friction sleeve and the 1.55 in.^{2} (10 cm^2) Fugro-cone penetrometer (reference cone).

4.3.5. Penetration Depth Measurement System

The penetration depth is measured using an electronic analog to digital converter depth decoding system. The depth decoder consists of a metal disk, a light emitting diode, and an optical sensor. Holes are drilled at equal distances on the circumference of the disk. As the cone advances (penetrates the specimen), a cable connected to a push rod and wound around the pulley (connected
to the disk), mechanically turns the disk. The distance between two consecutive holes on the disk represents a penetration of 2 cm. The light emitting diode and the sensor are installed on either side of the disk. When the light emitted by the diode passes through a hole, the sensor senses the light and generates a pulse that triggers the multiplexer to switch the channels for analog to digital (A/D) conversion. This process continues until the end of cone penetration.

4.3.6. Auxiliary System

The LSU/CALCHAS auxiliary system consists of a trolley crane system, a hanging scale, and a mixer. The material handling system for the LSU/CALCHAS consists of two overhead cranes (two ton capacity) moving on a horizontal beam that rolls transverse rail beams. This overhead crane system covers the total area of the LSU/CALCHAS. It is designed for lifting the specimen (and the former) and accurately centering and placing the specimen on top of the chamber piston. It is also used for lifting and placing the hydraulic push jack on the top lid of the chamber. In addition, a one-ton electric crane rolling on a cantilever swivel beam is available. A digital hanging scale, MSI-3260 Challenger, manufactured by Measurement Systems International, with

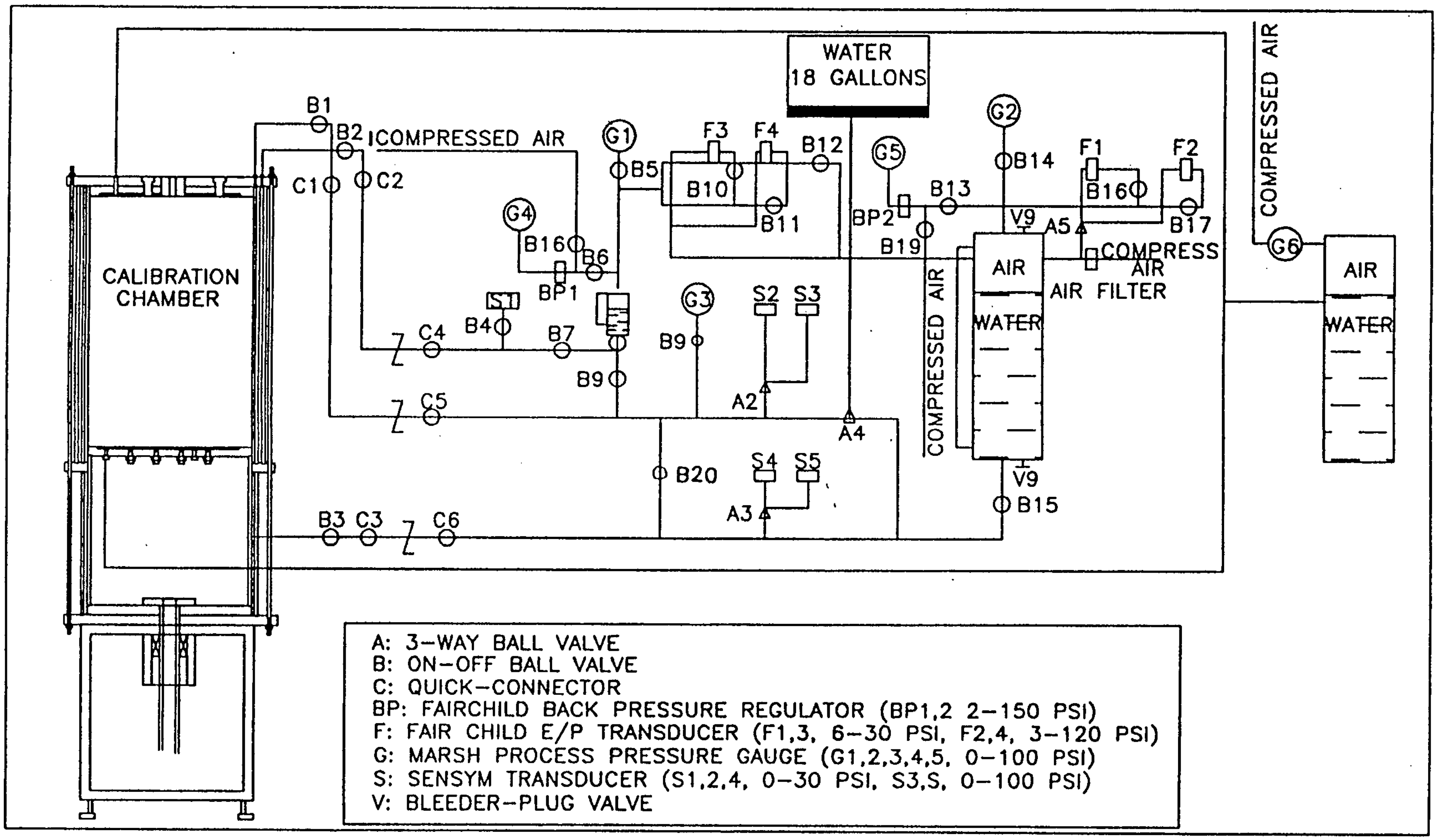


Figure 4.4
Panel of controls

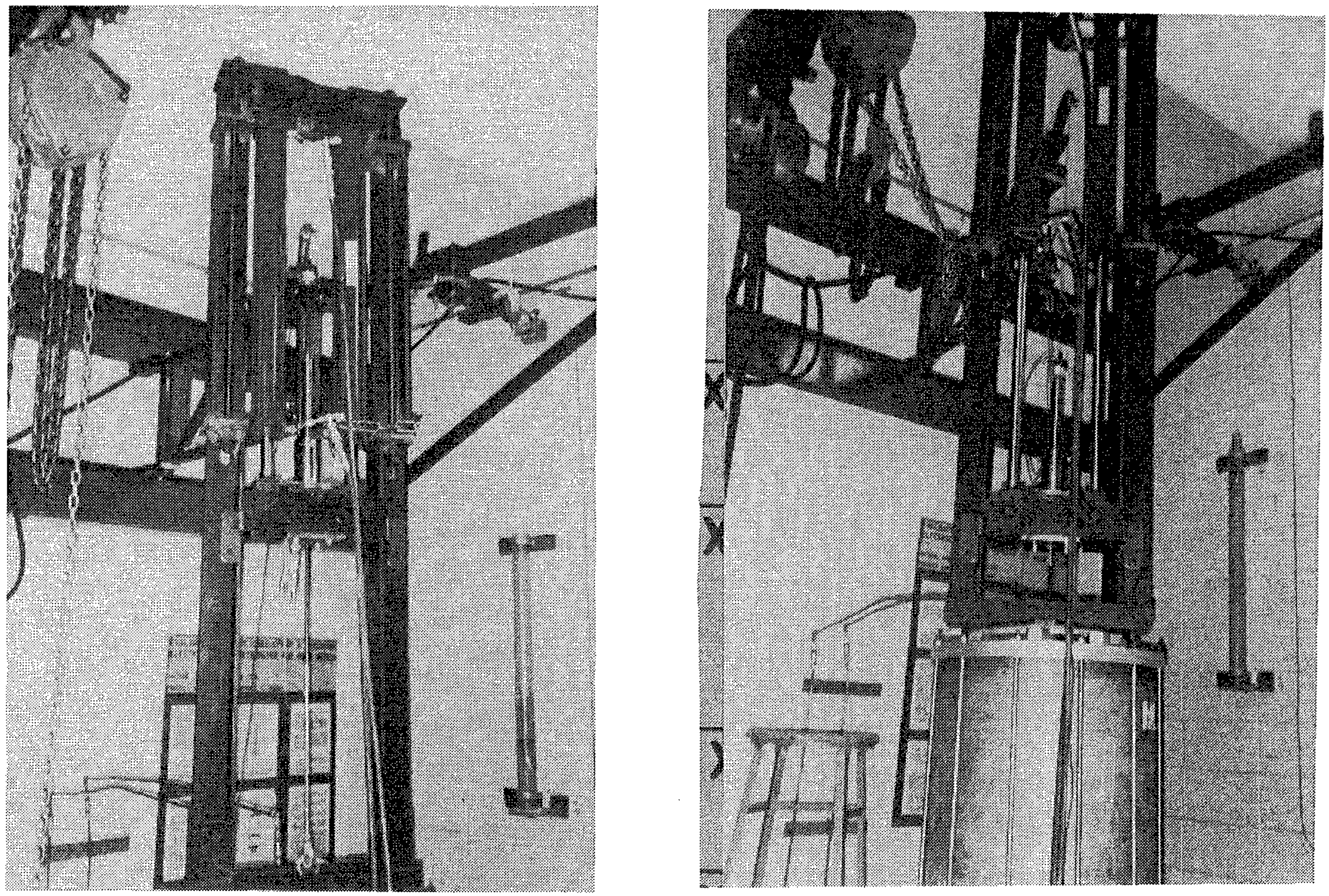


Figure 4.5
Piezocone penetration test: (a) before pushing; and (b) after pushing

a maximum load capacity of 2000 pounds (907.2 kilogram) and a reading precision of 1 pound (0.45 kilogram), is integrated to the two-ton crane and used for determining the soil sample weight after compaction. The mixer is a utility concrete mixer with 1/3 hp (249 W) motor, drum rpm of 30 to 32, and drum capacity of 3.5 ft.^{3} (0.1 m^3) manufactured by Olympia Industrial Inc.

4.3.7 Cone Penetrometers

Miniature Piezocone Penetrometer

The miniature piezocone penetrometer, Figure 4.6, fabricated by Fugro-McClelland Engineers B.V., the Netherlands, has a projected cone area of 0.16 in.^{2} (100 mm^2) and a cone apex angle of 60 degrees. The maximum normal load capacity is 1124 lb (5 kN). The penetrometer has two alternatives for the filter location. The filter can be located in the lowest 1/4 of the cone at the tip (U1 configuration, Figure 4.6a), or starting 0.02 in. (0.5 mm) above the base of the cone and 0.08 in. (2 mm) vertical height (U2 configuration, Figure 4.6a). The filter is made of sintered stainless steel and has a pore size of 1.18 mil (30 μm). The pressure transducer has a stainless steel sensing diaphragm and a measuring range of 507.6 psi (3.5 MPa). There is no friction sleeve in the penetrometer. In order to measure sleeve friction, a miniature quasi-static cone penetrometer is used.

Miniature Quasi-Static Cone (MQSC) Penetrometer

The MQSC penetrometer used to conduct the QCPT is a 0.20 in.^{2} (127 mm^2) cross-sectional area subtraction type Fugro-McClelland cone penetrometer, with a friction sleeve 2.48 in. (63 mm) long and an apex angle of 60 degrees (Figure 4.7). It measures cone resistance and the combined cone and local sleeve friction resistances. The MQSC push rod has a reduced diameter of 0.38 in. (9.53 mm) compared to the cone, which is 0.48 in. (12.72 mm) in diameter. This is in contrast to the piezocone penetrometer which has a push rod of the same diameter as the cone.

4.3.8. Equipment for Specimen Preparation

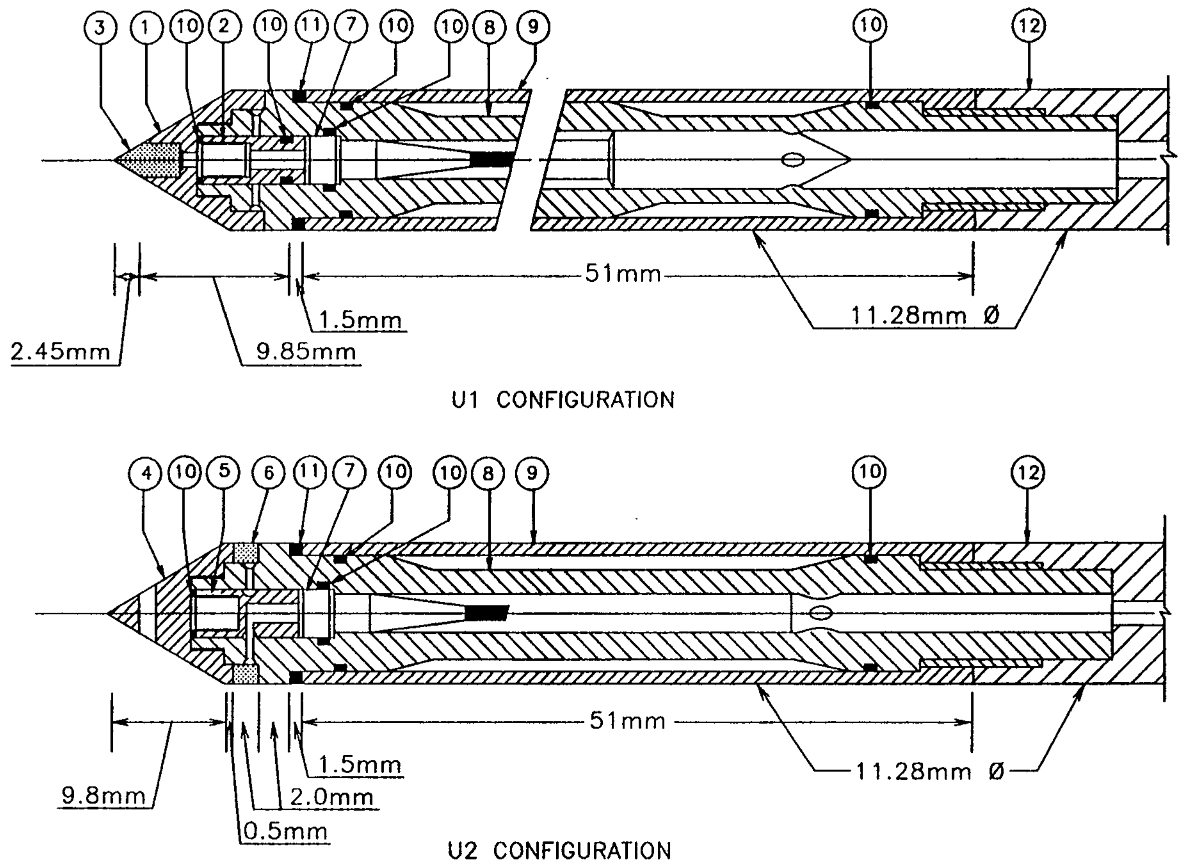
Automatic Tamper and Compaction Mold

An automatic tamper was designed and fabricated for the investigation of the effect of compaction parameters on MQSC results conducted on compacted soil samples 530 mm (20.87 in.) in diameter and 790 mm (31.10 in.) in height. The automatic tamper is composed of a 2 HP/1710 rpm Dayton adjustable speed motor drive resting on a 635 mm x 635 mm x 12.7 mm (25 in. x 25 in. x ½ in.) steel plate, a fly-wheel 635 mm (25 in.) in diameter, a 2.44 m (8 ft.) high steel frame, steel cable and pulley system, and a grabber. At each revolution of the motor driven fly-wheel, the grabber picks up the dual pie-shaped rammer while it rests on the specimen, raises it to a specified drop height, and releases the rammer for a free fall drop. Therefore, a drop is always relative to the specimen elevation. The distribution of blows is uniform over the surface of the soil sample and it is controlled by a set angle of the spacer rod, which rotates the grabber as it lifts and circulates the rammer. A dual pie-shape face tamping head in steel with two symmetrical sector faces each 20500 mm^2 (31.8 $in.^2$) is attached to a tubular steel shaft 38.1 mm (1 1/2 in.) in diameter and 2.34 m (7.67 ft.) long. The tamping head and shaft form a 70.64 kg (155.7 lb.) rammer. A split compaction mold and extension collar 819 mm x 528.6 mm x 6.4 mm (32 1/4 in. x 20 13/16 in. x 1/4 in.) and a 203.2 mm x 819 mm x 6.4 mm (8 in. x 32 1/4 in. x 1/4 in.), respectively, in steel are used in conjunction with a 523.9 mm x 76.2 mm (20 5/8 in. x 3 in.) spacer disk of T-6 aluminum. The two sides of the compaction mold and extension collar are fastened with nine 6.4 mm (1/4 in.) bolts. The split compaction mold is attached to a 635 mm x 635 mm x 12.7 mm (25 in. x 25 in. x 1/4 in.) baseplate, which provides a firm base for compacting the soil sample. A 1.59 mm (1/16 in.) thick rubber membrane is affixed to the interior perimeter of the compaction mold before beginning compaction. During compaction, two vacuum pumps with a capacity of 1 atm apply vacuum at three different sections of the compaction mold. These sections are equally spaced along the mold height in order to keep the membrane touching the internal perimeter of the compaction mold and to prevent any damage. A seal is applied between the compaction mold half cylinders in order to allow for applying vacuum during sample compaction. Figure 4.8 depicts the automatic tamper.

When preparing a compacted sample with this large dimension compaction mold, an equivalent energy equation [4.1] can be used to reproduce a specific compaction effort similar to ASTM.

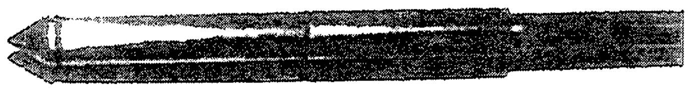
$$E = \frac{(P * L * N * n)}{V} \qquad (4.1)$$

where E: Compaction effort applied to the soil sample per unit of volume;
P: Weight of the sliding hammer;
L: Drop height of weight;
N: Number of blows applied to each layer;
n: Number of layers;
V: Volume of compaction mold



DESCRIPTION			
1	CONE TIP W1	7	PRESSURE TRANSDUCER
2	SPACER W1	8	MEASURING ELEMENT
3	FILTER W1	9	SLEEVE
4	CONE TIP W2	10	O-RING
5	SPACER W2	11	QUAD RING
6	FILTER W2	12	PUSH ROD

Figure 4.6
Schematic of the miniature piezocone penetrometer



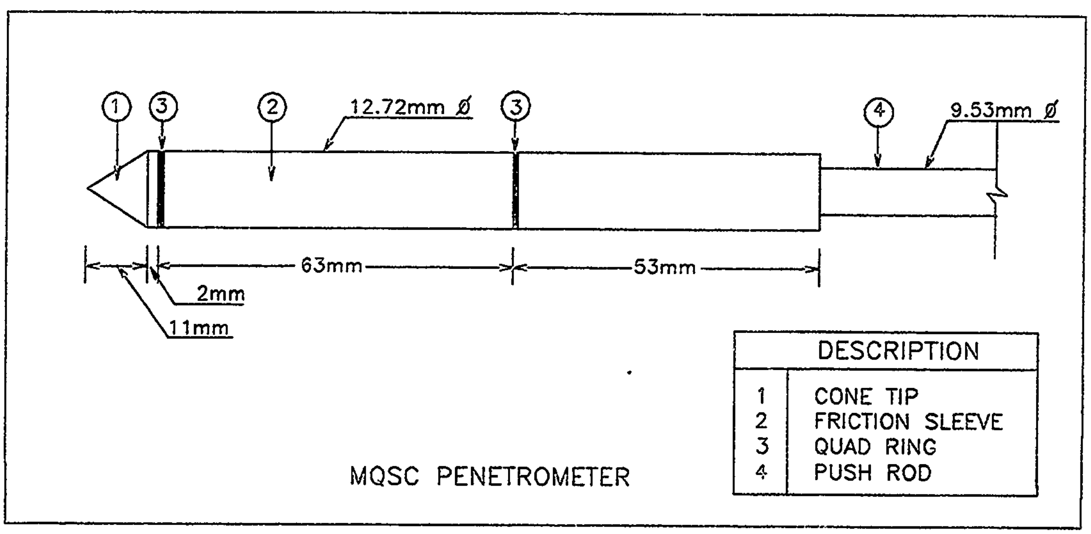


Figure 4.7
Miniature quasi-static cone (MQSC) pentrometer

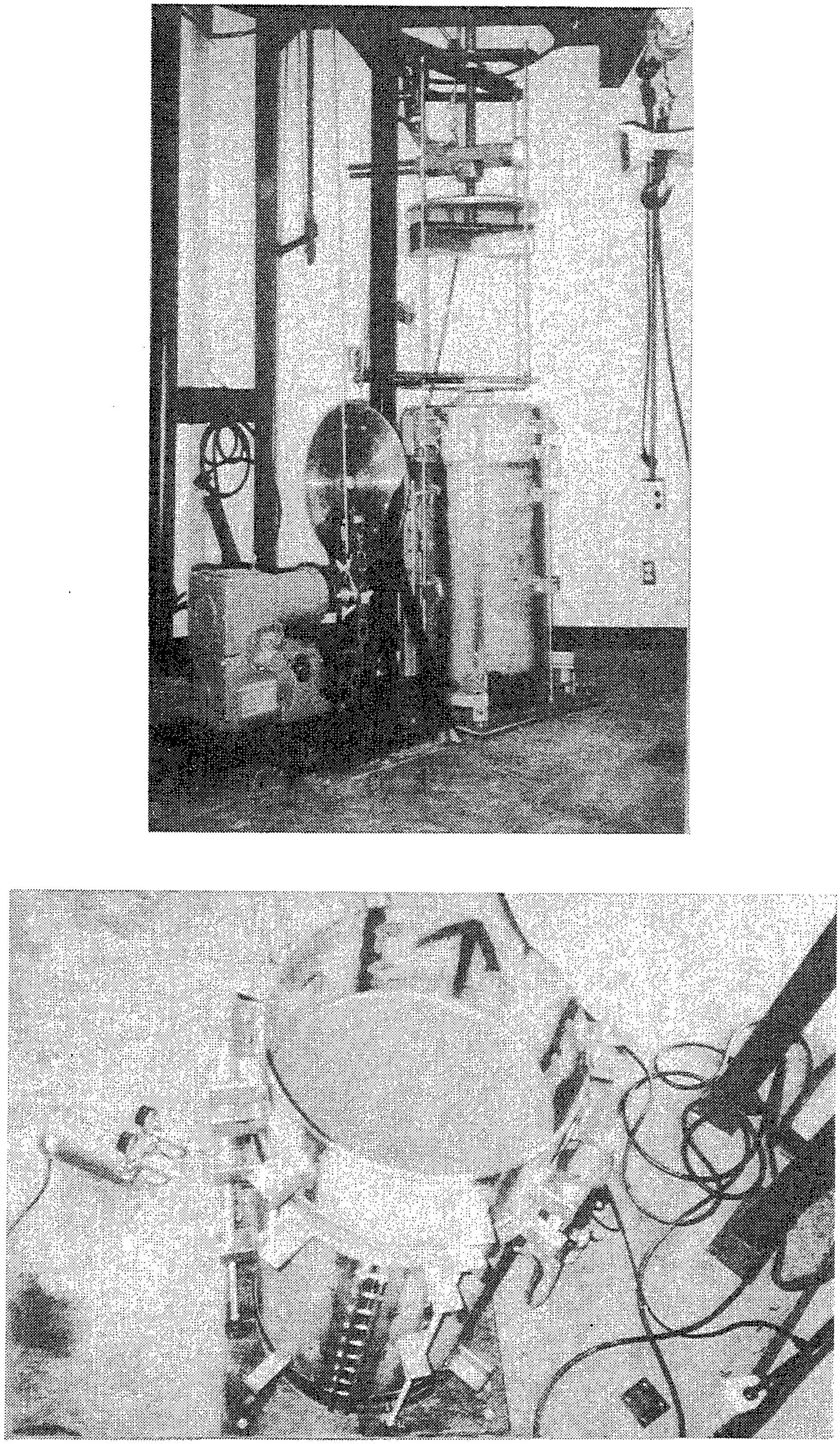


Figure 4.8
Details of the LSU CALCHAS automatic tamper and compaction mold

Slurry Consolidometer

An automated slurry consolidometer is available for preparing preconsolidated cohesive soil specimens *[26]*. The slurry consolidometer (Figure 4.9) consists of two PVC tubes, 525 mm (20 11/16 in.) inside diameter, 15 mm (9/16 in.) thick, and 812 mm (32 in.) high. The lower tube is split longitudinally into two halves and held together by a metal frame. This feature eliminates the need for any extrusion and minimizes disturbance of the soil specimen while transferring it into the calibration chamber. The upper tube serves as an additional storage compartment for the high water content slurry during the initial stage of consolidation. At the end of the slurry consolidation, the specimen is confined in a 1.59 mm (1/16 in.) thick rubber membrane in the lower tube. The inside surface of the lower tube is lined with sandpaper to provide friction and prevent slippage of the membrane which may otherwise be caused by the consolidating slurry. The ends of the two PVC tubes are lined with rubber gaskets for a water tight fit and to prevent damage to the membrane. The upper tube of the consolidometer is bolted to the lower one using six steel rods that connect an aluminum top lid to the bottom base frame. This assembly acts as a reaction frame for the loading system. The base frame is mounted on four rollers so the consolidometer with the specimen can be moved easily. The consolidometer is designed to consolidate specimens up to a maximum vertical stress of 80.1 psi (552 kPa). The base plate at the bottom of the consolidometer has holes for drainage/back pressure and for pore pressure measurements. Eight pore pressure access ducts connected to individual pressure transducers extend through the base plate into the specimen.

The loading system consists of a reaction frame with a single acting hydraulic cylinder (jack) powered by an air-hydraulic pump and bolted to the top lid. Load from the push jack is transferred to the soil through a steel piston rod and an aluminum piston plate. The vertical stress is recorded and controlled by monitoring the oil pressure using a pressure transducer. The piston and the aluminum base plates have porous plastic discs attached to their inner ends with connections to allow two-way drainage. An LVDT connected to the piston rod measures the vertical settlement of the sample during the consolidation phase. The data acquisition software developed in Pascal acquires and appends data (pore pressures, consolidation stress and consolidation settlement) into a file and also displays the data on a computer screen in graphic form plotted in real time. A schematic view of the slurry consolidometer set up is shown in Figure 4.10.

Pluviator

It is also possible to test cohesionless soils and artificially cemented soil specimens using the pluviator available at LSU *[27]*.

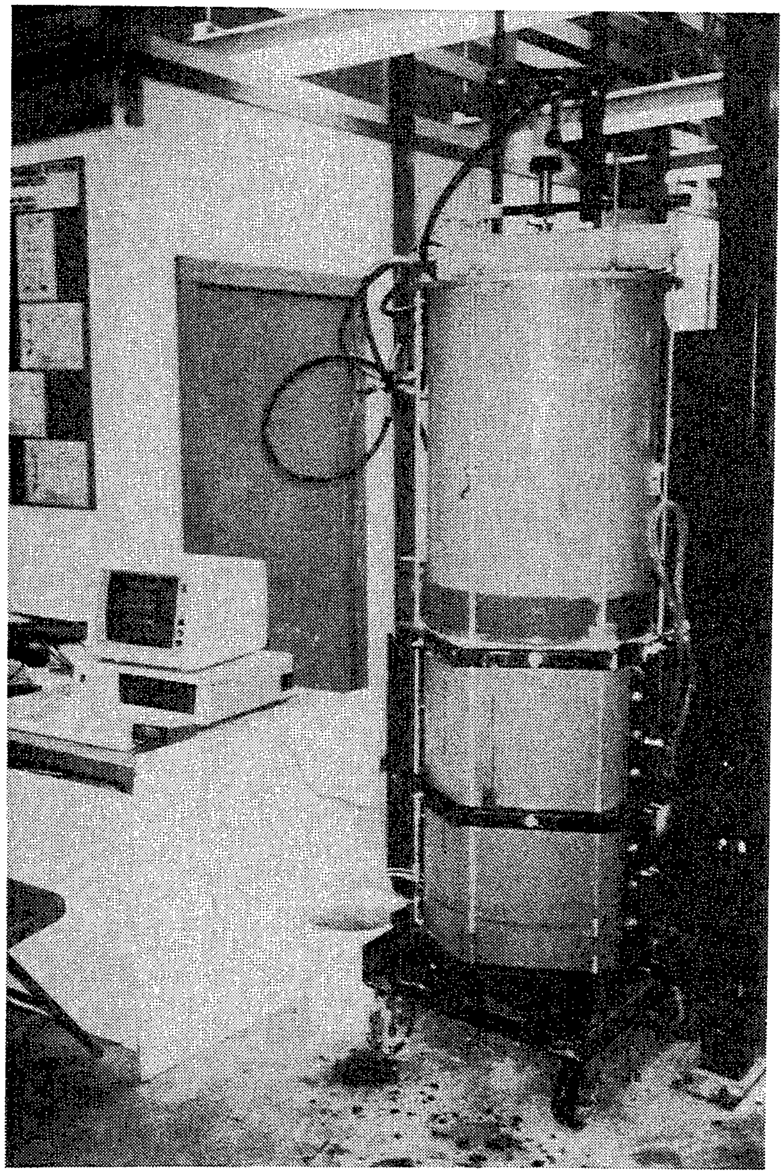


Figure 4.9
Slurry consolidometer

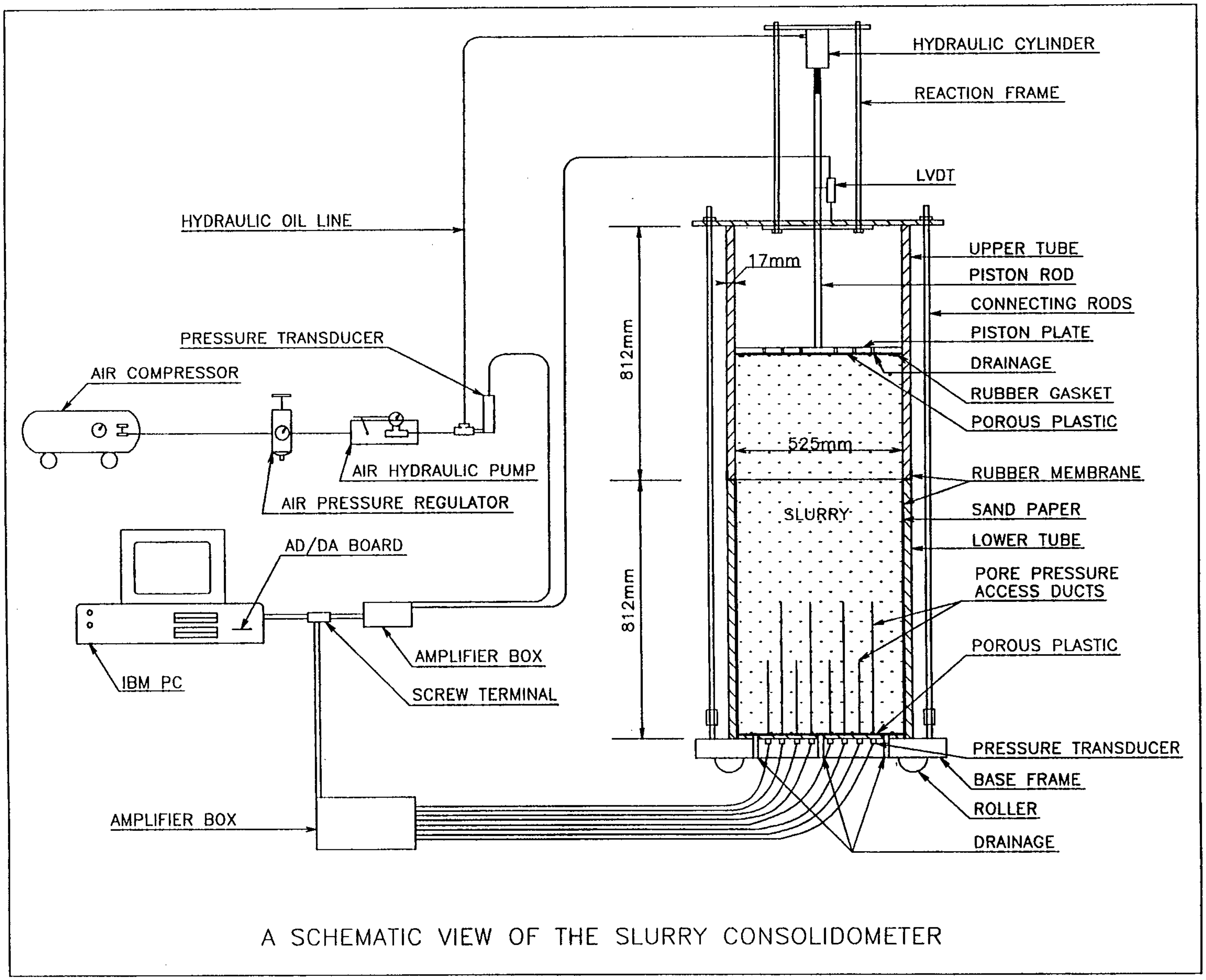


Figure 4.10
Schematic of the slurry consolidometer system

Specimen Boundary Conditions

The LSU/CALCHAS can consolidate and test soil specimens at a variety of stress paths including K_0 consolidation (at zero lateral strain). It can also simulate the four traditional penetration boundary conditions commonly referred in literature as:

BC1: Constant vertical stress and constant lateral stress

BC2: Zero vertical strain and zero lateral strain

BC3: Constant vertical stress and zero lateral strain

BC4: Zero vertical strain and constant lateral stress

4.3.9 Data Acquisition and Reduction Software

The data acquisition and reduction software consists of computer programs written in Turbo Pascal version 4.0 around the HALO'88 graphics library environment. For clarity, the consolidation phase and each of the boundary conditions during the penetration phase are controlled via independent software. Instrumentation of the CALCHAS allows all of the pertinent test data to be automatically recorded. The data recorded during a complete penetrometer test are listed below:

Consolidation Phase:

Vertical stress on the sample;

Vertical deflection of the sample;

Lateral stress developed in the inner sample cell;

Lateral stress developed in the outer sample cell;

Spatial pore pressure (in saturated cohesive soils).

Penetration Phase:

Cone resistance;

Local side friction resistance;

Cone penetration depth;

Vertical stress on the sample;

Vertical deflection of the sample;

Lateral stress developed in the inner sample cell;

Lateral stress developed in the outer sample cell;

Spatial pore pressures (in saturated cohesive soils).

4.4 Laboratory Testing

The laboratory testing program of this investigation envisioned the development of a calibration chamber system (CALCHAS) capable of performing cone penetration tests in compacted, consolidated or pluviated soil samples, under specified boundary restraints.
The test procedure can be divided in five steps, as follows:

Sample Preparation;
Chamber Preparation;
Consolidation Phase;
Penetration Phase;
Panel of Controls Shut-Down and Removal of the Sample.

Experimental work carried out on an artificially prepared mixture of 80% fine sand and 20% kaolinite in dry weight (liquid limit of 25%, plastic limit of 18%, and plastic index of 7) for the verification of the laboratory testing equipment is described by de Lima (1990) *[19]* and de Lima and Tumay (1992) *[19], [23]*.

Wang (1993) conducted cone penetration tests on compacted soil samples in the calibration chamber (CC) using the 0.20 $in.^2$ (1.27 cm^2) miniature quasi-static cone penetrometer *[28]*. The results were compared with those obtained using the 1.55 $in.^2$ (10 cm^2) standard quasi-static electric cone penetrometer and the 2.33 $in.^2$ (15 cm^2) electric friction cone penetrometer. Three kinds of soils, which represent different extremes of the sediments from the spectrum of characteristic soils in the State of Louisiana, were used to prepare compacted samples for the calibration chamber tests. A testing scheme, which simulates a statistical "Factorial Model," was used to design the testing in order to investigate the cross influence of different experimental factors.

Tip resistance and friction ratio were identified as the most prominent character indices in soil classification. Water content, pre-consolidation stress, compactive effort, and diameter ratio have significant influence on CPT results. It also has been demonstrated that MQSC, in general, could be used in field CPT testing instead of standard size larger cones without a large scale deviation from current CPT standards.

In this study, the primary testing was accomplished in the calibration chamber tests using the miniature quasi-static cone penetrometer. Three different but representative soils prevalent in Louisiana were used. The detailed research program can be described as follows:

1) A series of soil laboratory conventional tests, including grain size analysis, Atterberg limits, compaction test (standard and modified compaction effort), CBR, and other strength tests (i.e., triaxial and unconfined compression tests, etc.), on three representative Louisiana soils was conducted in order to make comprehensive soil classification and determine engineering properties.

2) Calibration chamber tests on the compacted soil samples prepared from different soil types, water contents, and compaction efforts with varying test conditions (i.e., cone type, boundary condition, vertical consolidation stress, and testing location, etc.) were conducted to analyze the effects of those testing factors on CPT results.

3) The effect of different test factors on the cone penetration resistance in the calibration chamber tests was evaluated.

4) Based on the results of the previous studies, empirical correlations between miniature cone penetration results (typically, tip resistance) and soil engineering parameters such as CBR, M_o, and γ_{dry} for soils commonly encountered in Louisiana were developed in order to provide an effective and practical approach to road and highway engineering and design.

Three different types of soils furnished by the Louisiana Transportation Research Center, which represent different extremes of the sediments from the spectrum of characteristic soils in Louisiana, were used in this study to prepare compacted samples for the calibration chamber cone penetration test. They were:

Soil	Location
Sandy soil	Port Allen Stock Pile, LA
Silt soil	Baines, LA
Clay soil	Big River Industries, Erwinville, LA

For convenience, these three soils, hereafter, will be called River sand, Baines silt and Erwinville clay, respectively.

4.5. Soil Classification

The grain size distribution of the three types of soils are shown in Figure 4.11.

River Sand

This soil consists of 84% sand, 15% silt, and only 1% clay. Therefore, it can be described as non-plastic fine sand or silty-sand and classified as A-3 (using AASHTO system) or SP-SM (using Unified system).

Baines Silt

This soil consists of 49.86% sand, 36.75% silt, and 13.39% clay. Therefore, it can be described as sandy silt and classified as A-4(1) (using AASHTO) or CL-ML (using Unified system).

Erwinville Clay

It was found that this soil has a high plasticity index (PI = 54) and it consists of 79% clay, 20% silt, and only 1% sand (Appendix B-3). Therefore, this soil can be described as silty clay or clay and classified as A-7-5(65) (using AASHTO) or CH (using Unified system). Compaction characteristics and CBR-dry density relationships of the three types of soils are shown in Figures 4.12 and 4.13.

4.6. Correlation Between Cone Data and Embankment Engineering Properties

Correlations between soil compressibility modulus, dry density, CBR, and cone resistance were developed using the MCPT calibration chamber test results.

4.6.1. Correlation Between Cone Resistance and Soil Compressibility Modulus M_o

Figure 4.14 shows a statistical correlation between miniature cone tip resistance (q_{cn}) and soil compressibility modulus (M_o). It is evident that the cone resistance gradually increases with the increase in compressibility modulus. It is further found that Baines silt has a relatively low compressibility modulus compared to the other two types of soils.

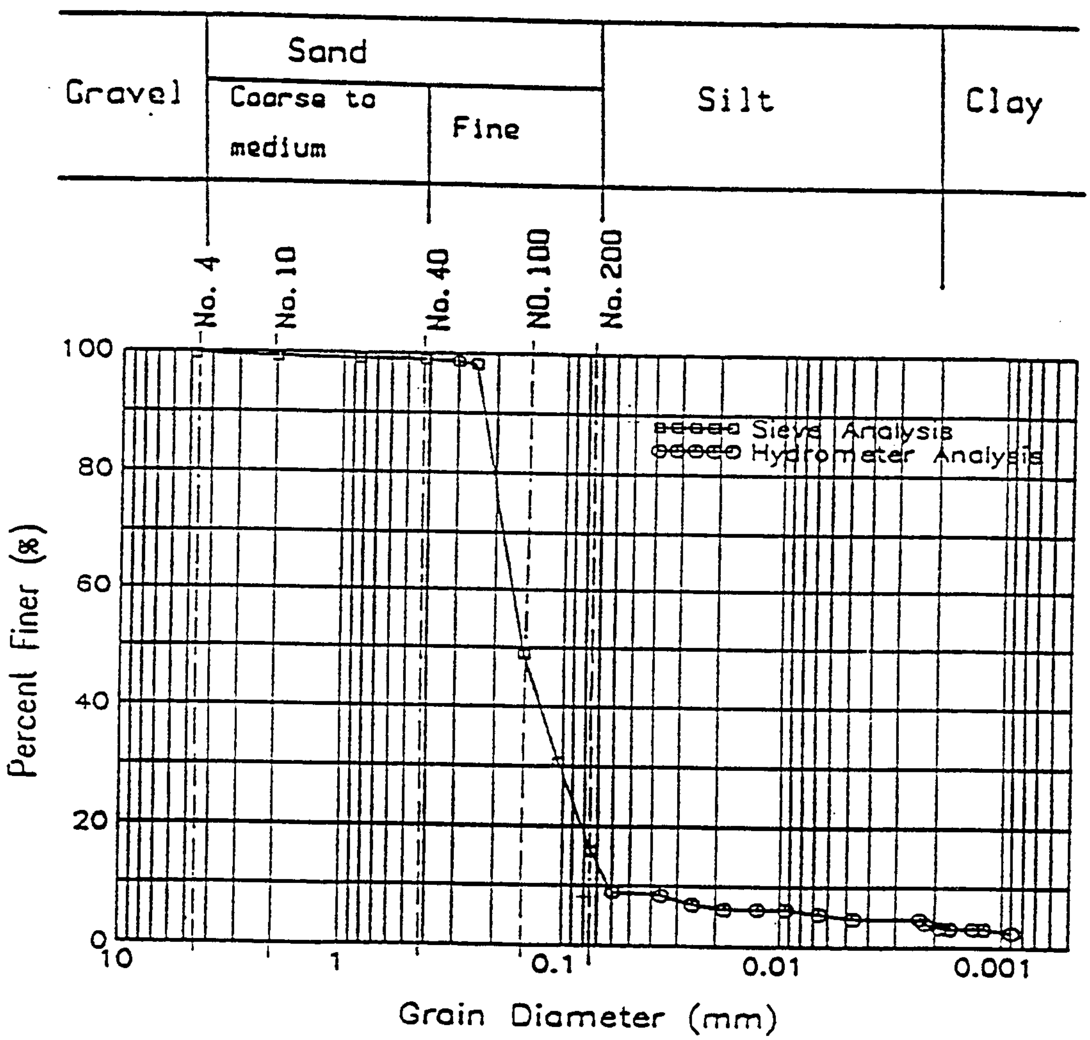


Figure 4.11(a)
Grain size distribution of river sand

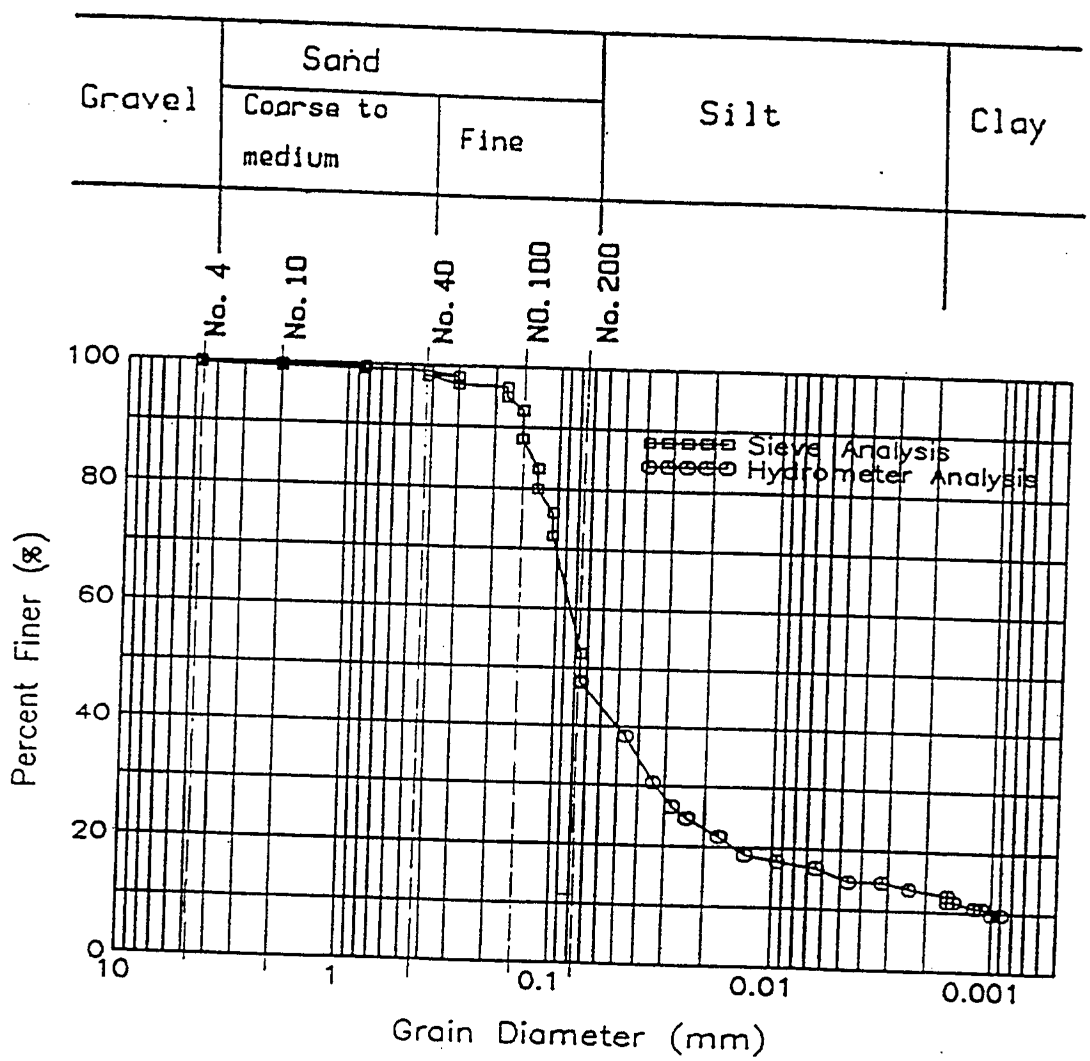


Figure 4.11(b)
Grain size distribution of Baines silt

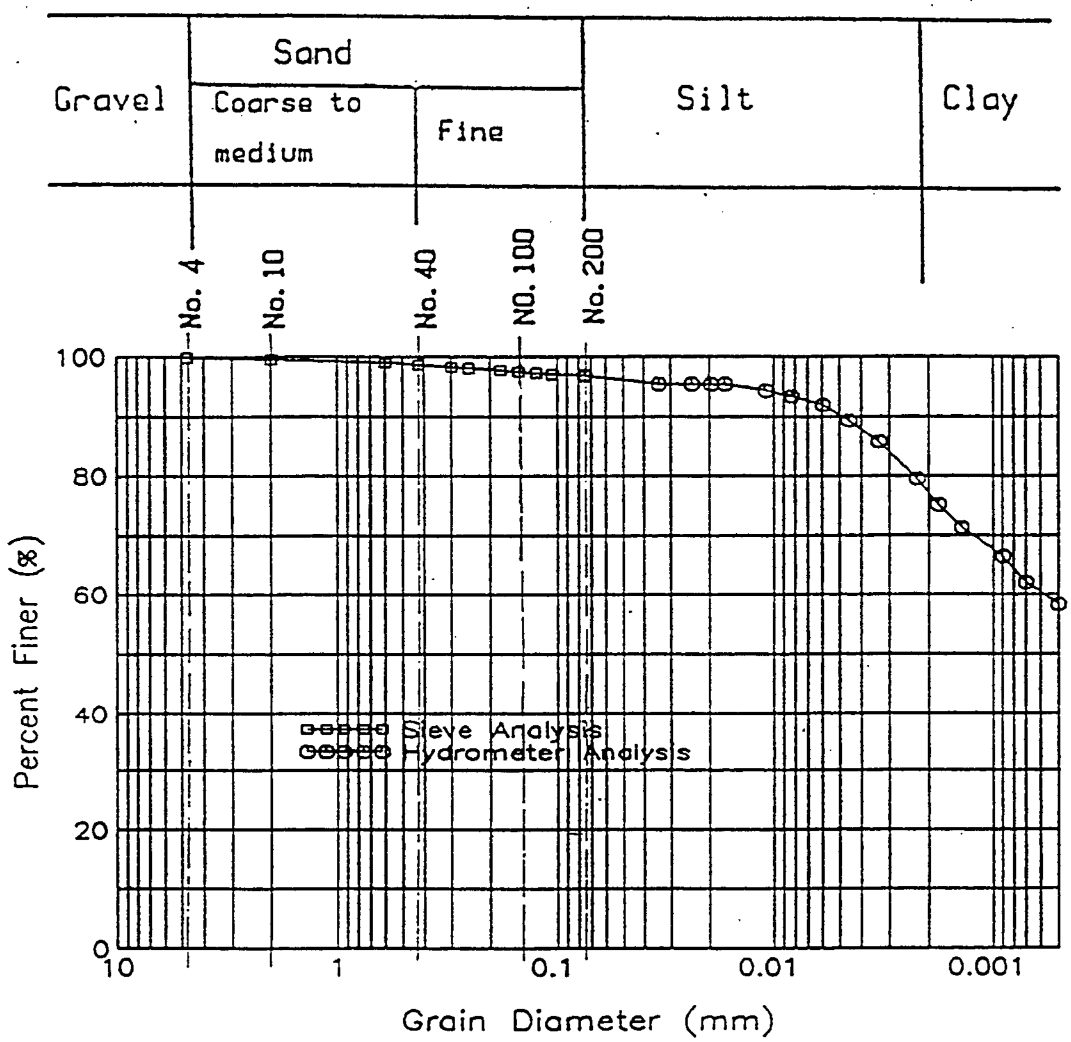


Figure 4.11(c)
Grain size distribution of Erwinville Clay

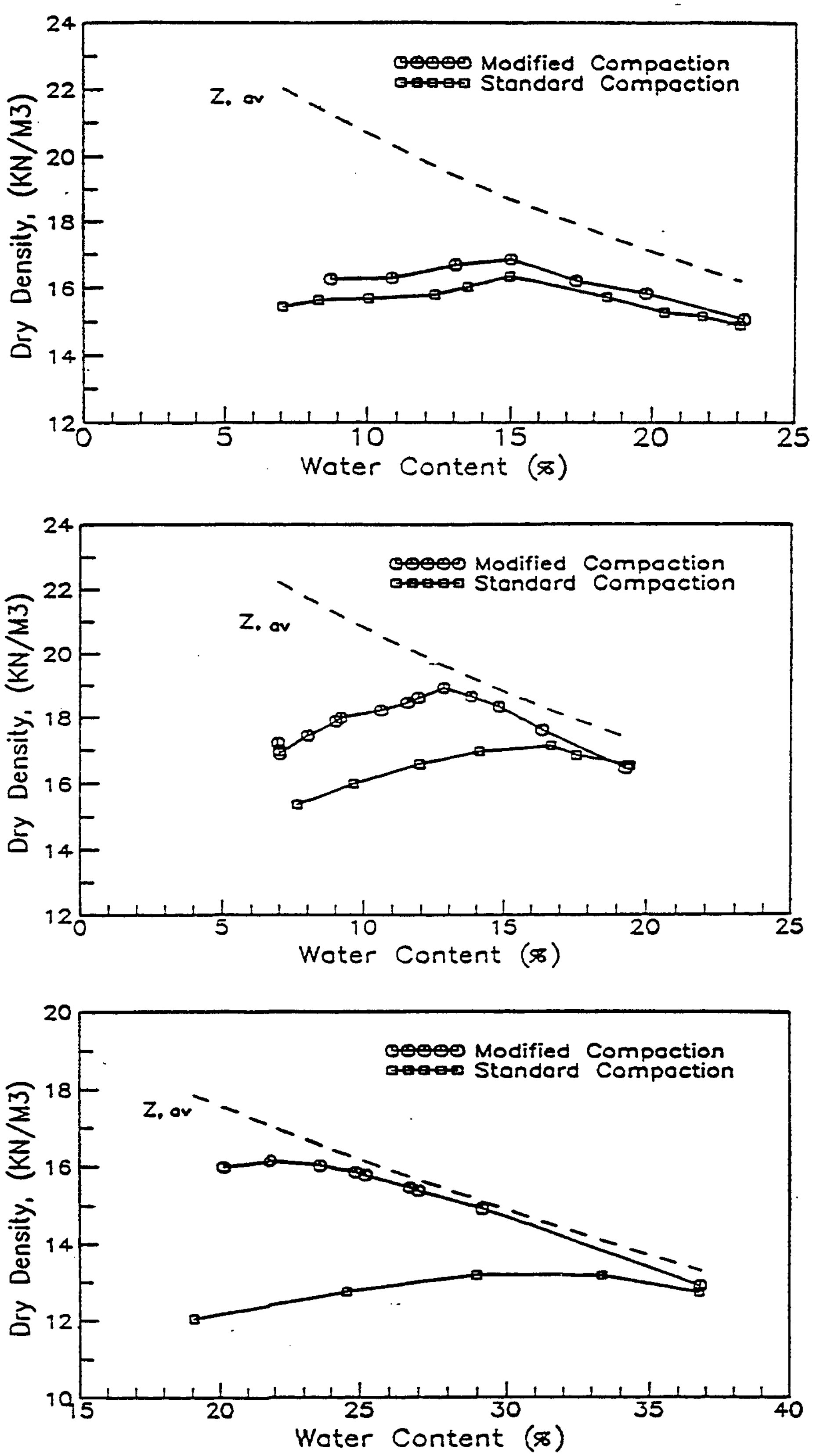


Figure 4.12
Compaction characteristics (a) river sand (b) Baines silt (c) Erwinville Clay

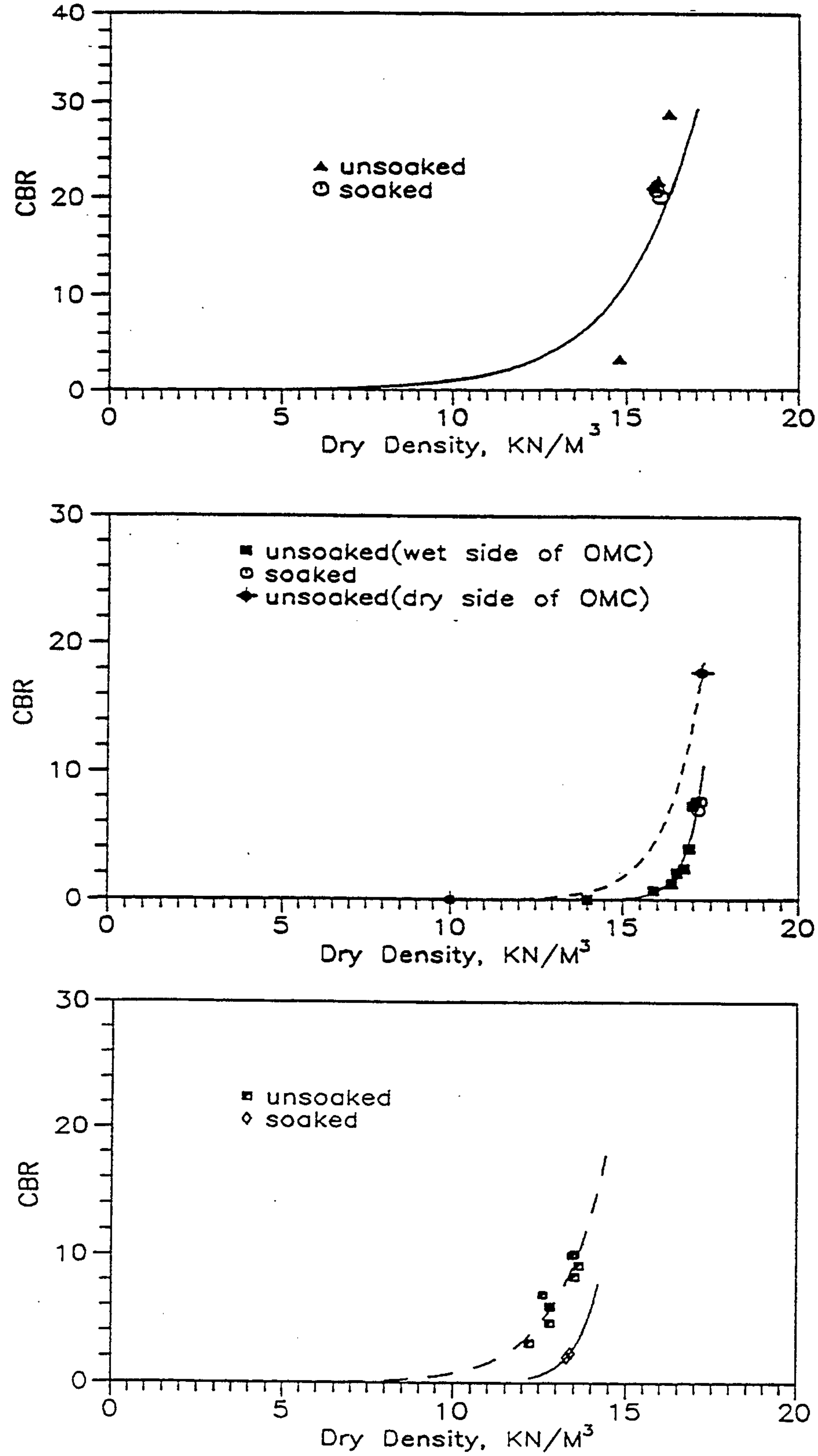


Figure 4.13
CBR-dry density relationship (a) river sand (b) Baines silt (c) Erwinville Clay

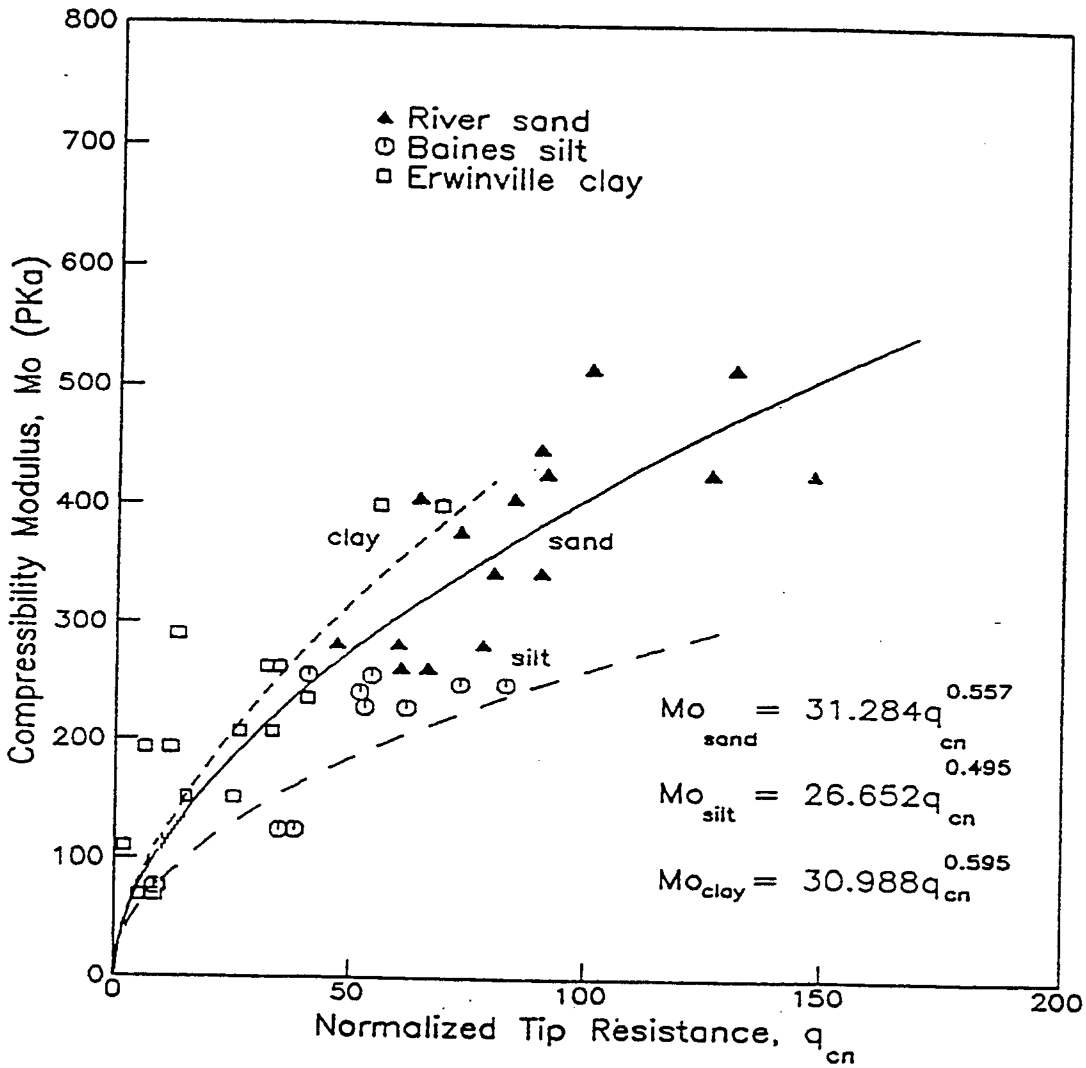


Figure 4.14
Correlation of tip resistance versus compressibilty modulus

A set of empirical correlations between normalized tip resistance and soil compressibility modulus was developed:

$$
\begin{aligned}
M_{o(\text{River sand})} &= 31.284\, q_{cn}^{0.557}\ (r^2 = 0.982)\ (4.2\text{-a}) \\
M_{o(\text{Baines silt})} &= 26.652\, q_{cn}^{0.495}\ (r^2 = 0.990)\ (4.2\text{-b}) \\
M_{o(\text{Erwinville clay})} &= 30.988\, q_{cn}^{0.595}\ (r^2 = 0.992)\ (4.2\text{-c})
\end{aligned}
\qquad (4.2)
$$

4.6.2. Correlation Between Cone Resistance and Soil Dry Density γ_{dry}

An overall trend between cone resistance and dry density may be inferred from Figure 4.15. Correlations relating normalized cone resistance with soil dry density are given in equation 4.3a-c. It is noted that for the same dry density, silt soil has the lowest tip resistance among these three types of soils.

$$
\begin{aligned}
\gamma_{dry\,(\text{River sand})} &= 2.260 \ln(q_{cn}) + 5.902\ (r^2 = 0.835) && (4.3\text{-a}) \\
\gamma_{dry\,(\text{Baines silt})} &= 2.363 \ln(q_{cn}) + 7.253\ (r^2 = 0.799) && (4.3\text{-b})\ (4.3) \\
\gamma_{dry(\text{Erwinville clay})} &= 1.615 \ln(q_{cn}) + 8.212\ (r^2 = 0.871) && (4.3\text{-c})
\end{aligned}
$$

4.6.3. Correlation Between Cone Resistance and CBR

Figure 4.16 shows the correlation of normalized tip resistance versus CBR for three types of soil. The relationship between tip resistance and CBR of Erwinville clay is approximately linear, however, somewhat nonlinear correlative trends were observed for Baines silt and River sand. Relevant correlations are given as follows:

$$
\begin{aligned}
CBR_{(\text{River sand})} &= 0.438\, \text{Exp}(0.043\, q_{cn})\ (r^2 = 0.988) && (4.4\text{-a}) \\
CBR_{(\text{Baines silt})} &= 0.398\, \text{Exp}(0.043\, q_{cn})\ (r^2 = 0.991) && (4.4\text{-b}) \quad (4.4) \\
CBR_{(\text{Erwinville clay})} &= 0.337\, q_{cn} + 0.007\ (r^2 = 0.983) && (4.4\text{-c})
\end{aligned}
$$

The above correlations to estimate soil compressibility modulus, dry density, and CBR from cone resistance should be used with caution in view of the limited data base and the scatter observed. Additional calibration chamber studies and in-situ tests on a wider range of soil type are needed to further validate/refine these correlations.

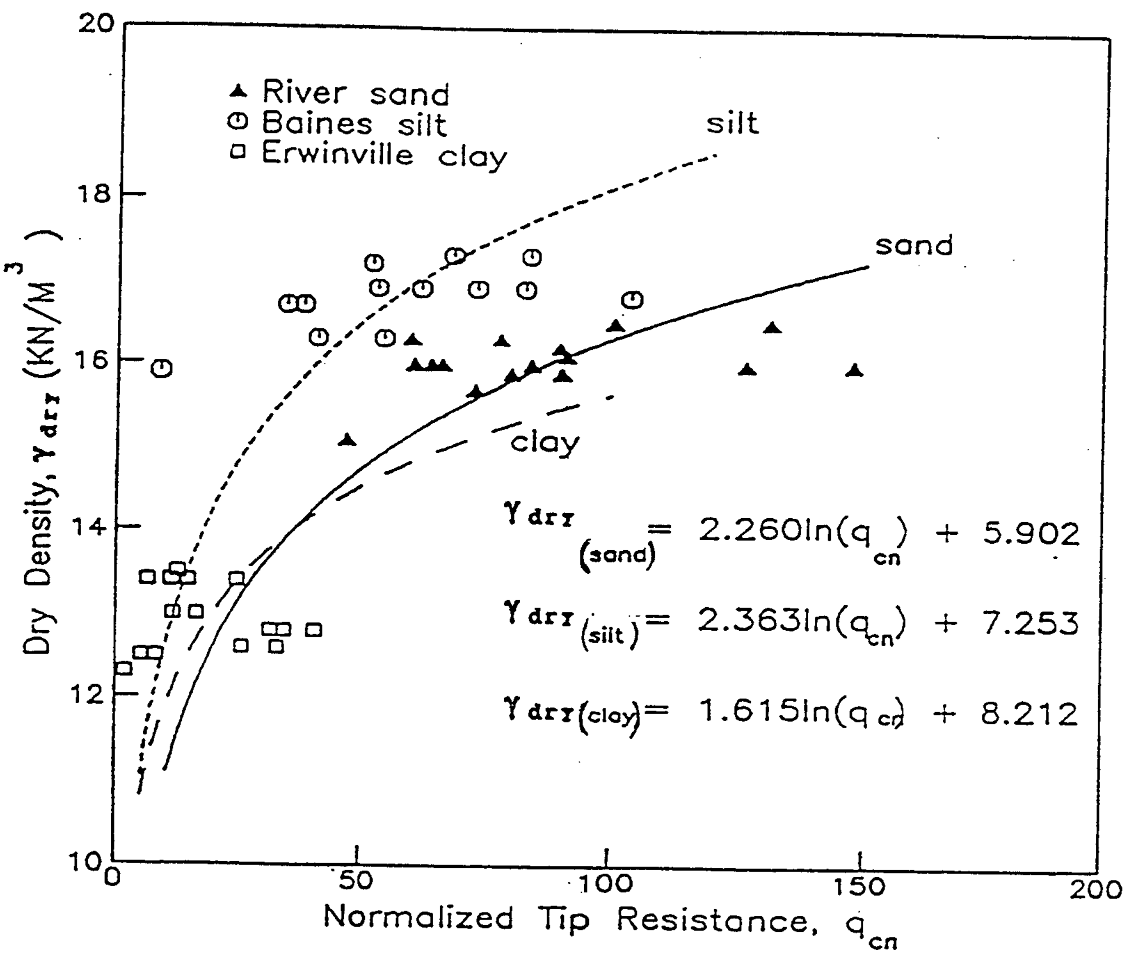


Figure 4.15
Correlation of tip resistance versus dry density

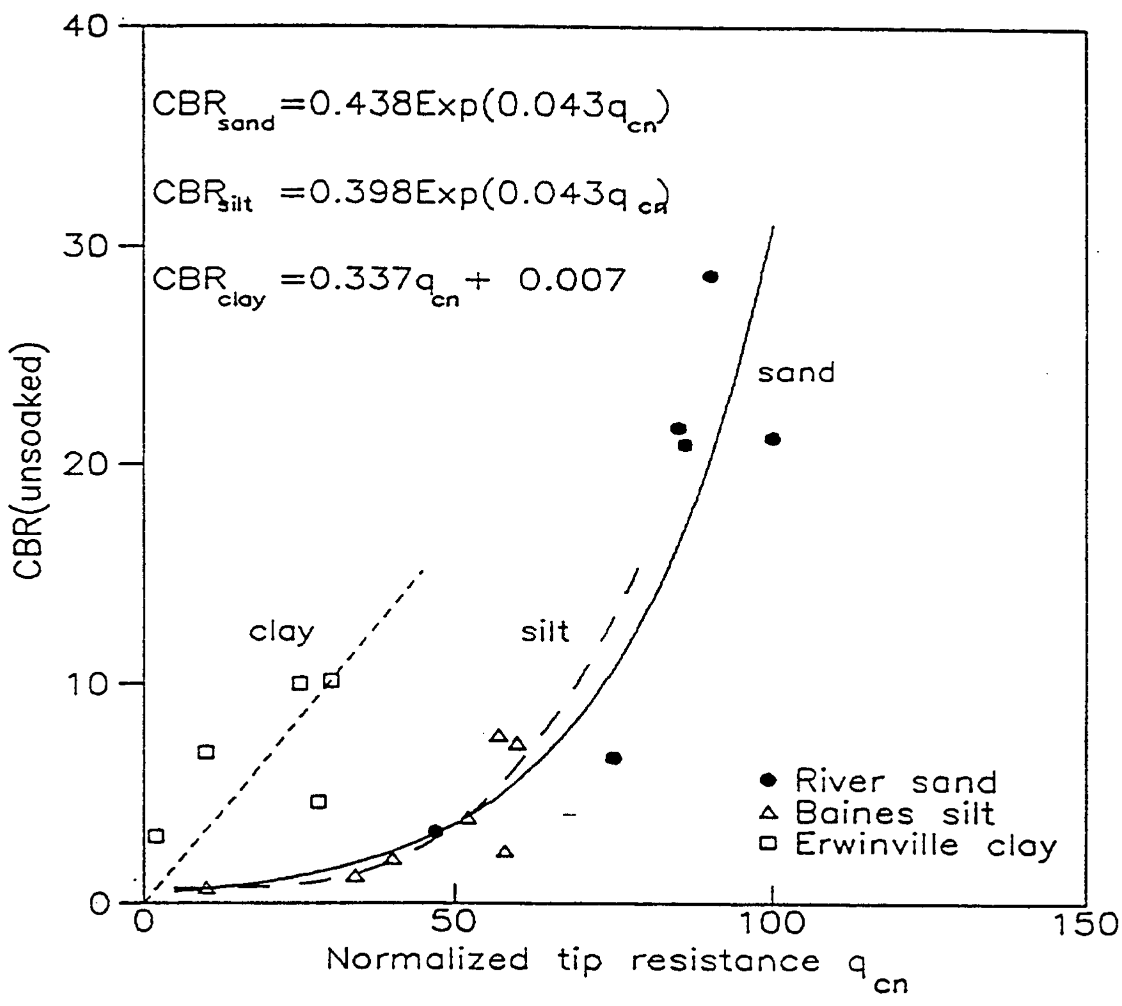


Figure 4.16
Correlation of tip resistance versus CBR

CONCLUSIONS

A prototype miniature electronic cone penetrometer system was developed for road and highway design and construction control. The equipment was implemented in the Research Vehicle for Geotechnical In-Situ Testing and Support (REVEGITS). Since Fugro-McClelland has discontinued production of the 0.20 in.^{2} (.27 cm^2) proprietary cone penetrometer, a 0.31 in.^{2} (2 cm^2) cross sectional area cone penetrometer with a friction sleeve area of 6.2 in. ^{2}(40 cm ^{2}) developed by GEOCOGNETICS, Houston, Texas was chosen as the alternative.

In order to investigate scale effects between different size cone penetrometers, in-situ tests were performed using 2.33 in.^{2}, 1.55 in.^{2}, 0.31 in.^{2}, and 0.20 in.^{2} (15 cm^2, 10 cm^2, 2 cm^2, and 1.27 cm^2) cone penetrometers. Statistical analysis was conducted and regression equations developed to transform 0.20 in.^{2} and 2.33 in.^{2} (1.27 cm^2 and 15 cm^2) cone data (cone resistance and sleeve friction) to the reference penetrometer. In-situ tests performed using the 0.31 in.^{2} (2 cm^2) miniature cone penetrometer showed no significant scale effects between the 0.31 in.^{2} and 0.20 in.^{2} (2 cm^2 and 1.27 cm^2) miniature cone penetrometers. Hence the same regression equations developed to transform 0.20 in.^{2} (1.27 cm^2) cone data to the reference penetrometer data is found applicable for the 0.31 in.^{2} (2 cm^2) miniature cone penetrometers. A multiplication factor of 0.85 can be used effectively to correct the 0.31 in.^{2} (2 cm^2) cone resistance in order to obtain the reference penetrometer cone resistance ($q_{c(1.55\,in.2\,(10\,cm2))} = 0.85 * q_{c(0.31\,in.2\,(2\,cm2))}$). The local side friction resistance and friction ratio should be corrected via linear regression equations considering two ranges of cone resistance: (1) soils with q_c equal or smaller than 81.9 ton/ft.^{2} (80 kg/cm^2), and (2) soils with q_c higher than 81.9 ton/ft.^{2} (80 kg/cm^2) No significant correction is necessary for cross-correlating cone resistance of the reference and 2.33 in.^{2} (15 cm^2) cross-section penetrometers. The implementation of the prototype miniature cone penetrometer was tested and verified by comparing penetration profiles with those obtained by the 2.33 in.^{2} (15 cm^2) cone penetrometer.

A calibration chamber system for laboratory calibration and formulation of correlations was developed and calibration chamber tests performed. Preliminary correlations relating soil compressibility modulus, soil dry density, and CBR with cone resistance were developed. Additional calibration chamber studies and in-situ tests need to be conducted to further validate/refine these correlations.

RECOMMENDATIONS

The 0.31 in.^{2} (2 cm^2) miniature electric cone penetrometer may be used for shallow subsurface investigations for road and highway design and construction control of embankments. It is recommended that the 0.31 in.^{2} (2 cm^2) miniature cone penetrometer test data be converted to reference penetrometer data using the regression equations given in this report before using any classification chart or interpretation method developed for the reference cone penetrometer. Correlations between miniature cone penetrometer data and engineering soil properties developed from calibration chamber tests may be used to obtain a preliminary estimate of the soil compressibility modulus, dry density, and CBR. These correlations should be used with caution until further validation by in-situ tests and calibration chamber studies on a wider range of soil type are completed.

The following recommendations are proposed for future equipment design and testing:

- The existing MQSC thrust system has a stroke of only 5.91 in. (150 mm), and hence the cone advance is not continuous. Normal stress release, excess pore pressure drop, and dissipation due to rate and consolidation effects can occur during pauses between strokes of the intermittent pushing. It is hence desirable to have a continuous push device in future designs. A single, continuous push rod assembly is recommended to minimize the problem of ground water seeping in through joints and damaging the electronics. A better design/procedure needs to be developed for easy service access and to minimize time during initial set-up.

- The prototype miniature cone penetrometer system is mounted in front of the 20-ton (18,144-kg) REVEGITS. For mobility and to provide site accessibility, the system may be mounted on a smaller (1-ton (907-kg)) 4 wheel drive all-terrain vehicle. This mounting arrangement is possible due to the smaller reaction forces needed to push a miniature cone compared to large size cones. The unit may be operated by one person and will be more economical and have a higher production rate.

- Develop and implement a new state-of-the-art data acquisition hardware system in order to eliminate the need for an expensive PCU-M. Upgrade the existing software for data acquisition, processing, and analysis.

- Test and evaluate the continuous feed electronic miniature cone penetrometer system in well-characterized and well-documented sites (i.e. National Geotechnical Experimentation Sites) to further expand the data base on scale effects.

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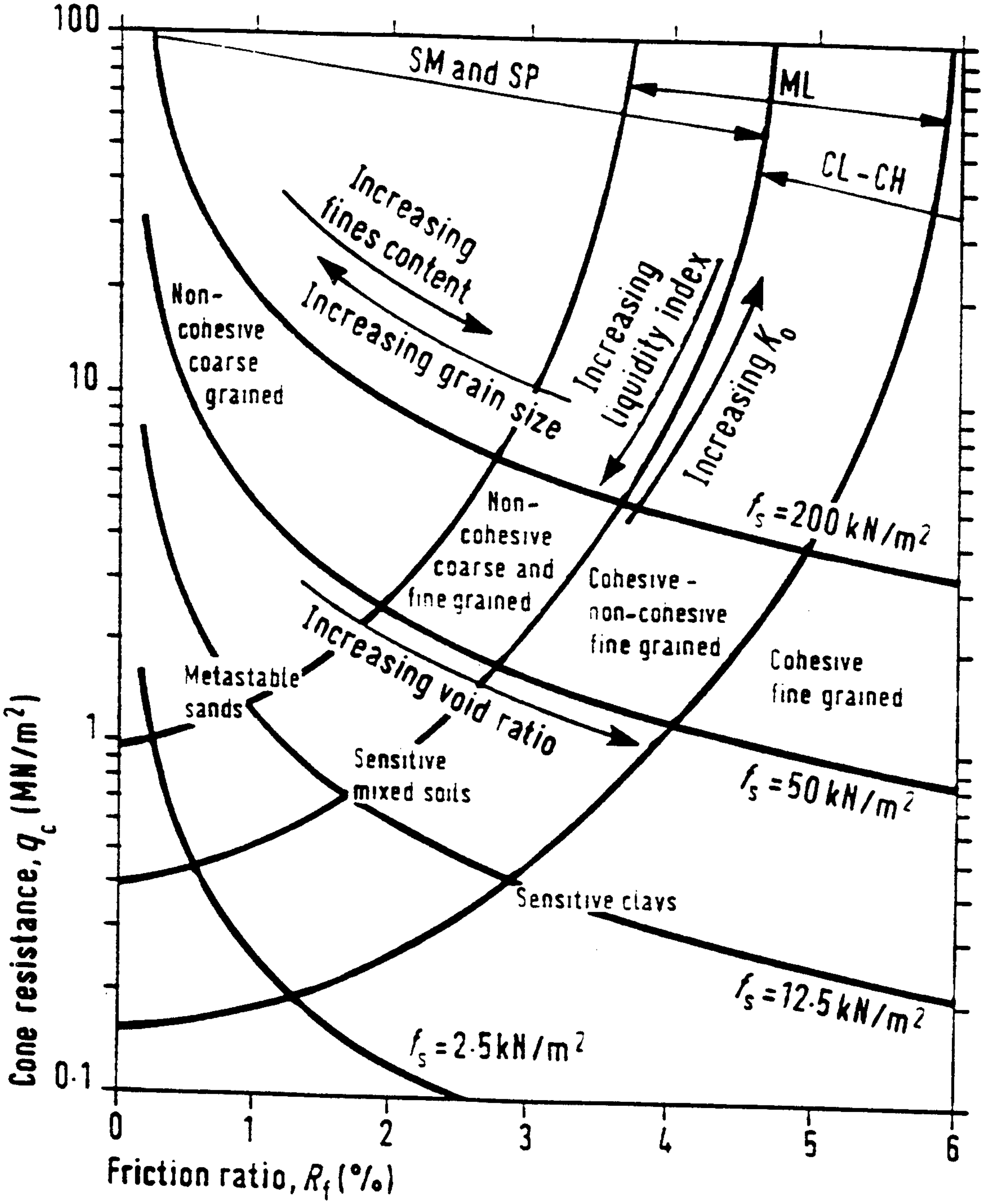
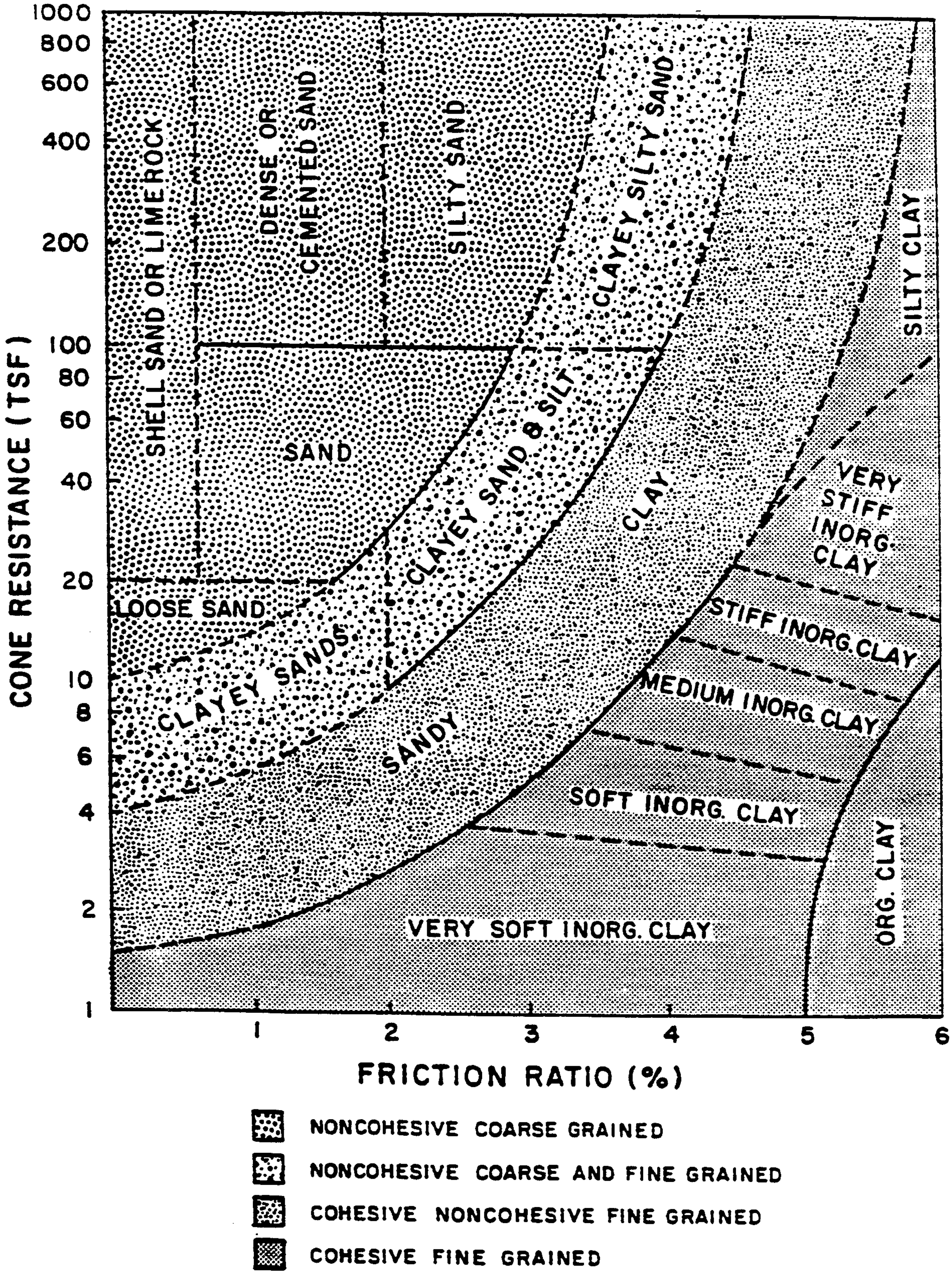
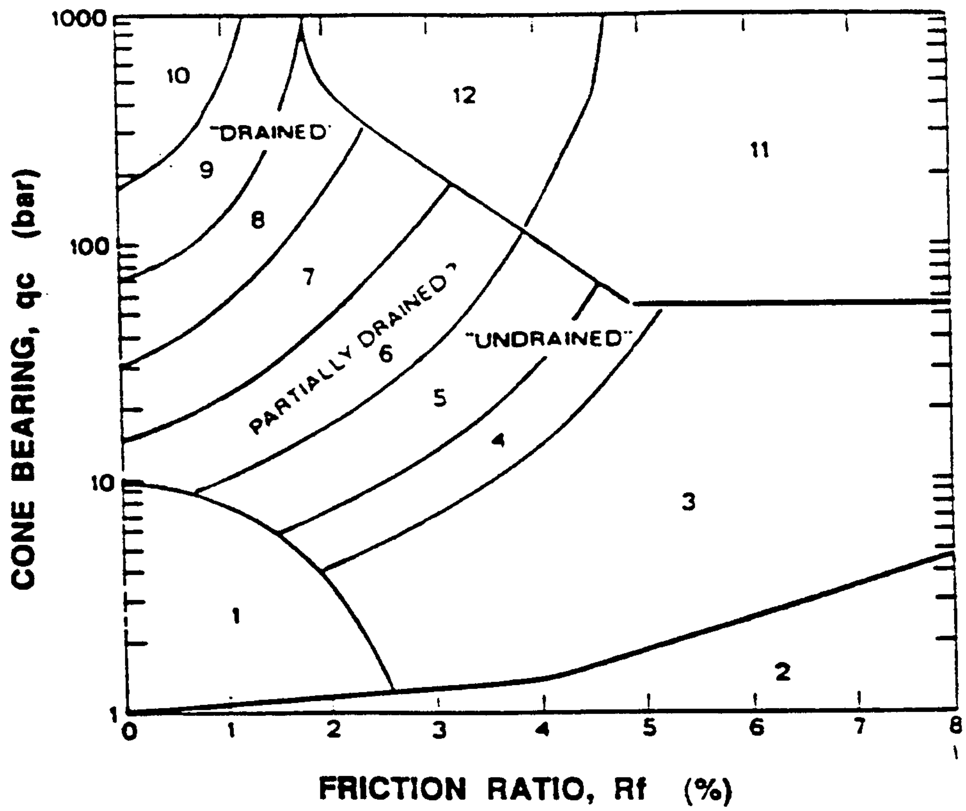
APPENDIX



Soil classification chart proposed by Douglas and Olsen (1981)



Soil classification chart proposed by Tumay (1985)



ZONE	SOIL BEHAVIOUR TYPE
1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

(*) overconsolidated or cemented

Soil classification chart proposed by Robertson et al. (1986)

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