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Dario Cardoso De lima

Louisiana State University and Agricultural & Mechanical College

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Development, fabrication and verification of the LSU in situ Testing Calibration Chamber (LSU/CALCHAS)

de Lima, Dario Cardoso, Ph.D.
The Louisiana State University and Agricultural and Mechanical Col., 1990
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DEVELOPMENT, FABRICATION AND VERIFICATION OF THE LSU IN SITU TESTING CALIBRATION CHAMBER
( LSU/CALCHAS )

A Dissertation
Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
in partial fulfillment of the
requirements for the degree of
Doctor of Philosophy

in

The Department of Civil Engineering

by

Dario Cardoso de Lima
B.S., University of Sao Paulo, Sao Paulo, Brazil, 1975
Specialist, New University of Lisbon, Lisbon, Portugal, 1979
M.S., University of Sao Paulo, Sao Paulo, Brazil, 1981
December 1990
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Finally, this dissertation is dedicated to the foreign side of each one of us, and in particular to mine which I will hopefully visit more frequently in the future.
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<tr>
<td>C</td>
<td>Sample Coefficient of Variation, defined as the ratio between the sample standard deviation and sample mean, in percent</td>
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<td>CBR</td>
<td>California Bearing ratio</td>
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<td>cm</td>
<td>Centimeter</td>
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<td>D̄</td>
<td>Samples Difference Means</td>
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<td>Rₕ</td>
<td>Friction Ratio</td>
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<tr>
<td>K₀</td>
<td>Coefficient of Lateral Earth Pressure at Rest</td>
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<tr>
<td>LL</td>
<td>Liquid Limit</td>
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<tr>
<td>m</td>
<td>Meter</td>
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<td>mm</td>
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<td>M₀</td>
<td>Modulus of Compressibility</td>
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<td>n</td>
<td>Number of Sample Observations</td>
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<td>OCR</td>
<td>Overconsolidation Ratio</td>
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<tr>
<td>Nₚ</td>
<td>Bearing Capacity Factor, relating cone resistance and in situ stress</td>
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<td>Nₚ, Nₜ</td>
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\begin{itemize}
    \item $q_c$: cone resistance
    \item $q_t$: Total Cone Resistance, corrected for unequal end area effects
    \item $S$: Sample Standard Deviation
    \item $S_0^2$: Samples Difference Variance
    \item $u_a$: Pore-Air Pressure
    \item $u_w$: Pore-Water Pressure
    \item $w$: Moisture Content
    \item $w_{opt}$: Optimum Moisture Content
    \item $\bar{X}$: Sample Mean
    \item $\gamma_d$: Dry Density
    \item $\gamma_{d\text{max}}$: Maximum Dry Density
    \item $\mu_1, \mu_2$: Population Means
    \item $\epsilon$: Vertical Strain
    \item $\sigma, \sigma_v$: Vertical Stress
    \item $\phi$: Friction Angle
\end{itemize}
This dissertation introduces the Louisiana State University Calibration Chamber System (LSU/CALCHAS) for calibrating electronic cone penetrometers and other in situ testing equipment. It permits the simulation of the $K_c$ consolidation and the four (4) traditional boundary conditions commonly referred in the literature. The calibration chamber system encompasses a 304 stainless steel cylindrical calibration chamber which can house a soil sample .55 m (20 21/32 in.) in diameter and .79 m (31 1/16 in.) in height. A fully computerized control panel, a data acquisition and reduction system, a large dimension automatic tamper for preparing compacted soil samples, and accessories comprise the complete system. Compacted soil samples can presently be prepared in the LSU/CALCHAS, but capability for testing pluviated sand and preconsolidated clay samples are also available.

In addition, the dissertation includes a preliminary test program to calibrate in the LSU/CALCHAS the 1.27 cm$^2$ Fugro-McClelland miniature electronic cone penetrometer (MQSC). This cone is expected to be used in engineering design and construction control of transportation facilities in the State of Louisiana, U.S.A. The laboratory verification phase of the research involved the compaction of a mixture of 80% of fine sand and 20% of kaolinite by weight, $K_c$ consolidation of the soil sample at 210 kPa (30 psi), and penetration of the sample via the MQSC at the penetration rate of 2 cm/sec. under boundary conditions 1 and 3.
A sister field study is presented in order to examine the scale effect topic when comparing sounding data obtained with the standard 10 cm² (reference) Fugro electronic friction cone penetrometer versus the MQSC and the 15 cm² Fugro electronic friction cone penetrometer. Five (5) representative sites in the State of Louisiana were selected for the field testing program, encompassing the range of sandy, silty and clayey soils. Compacted embankments and natural grade soils were investigated. Sounding 10 m deep were performed in two (2) highway embankments and three (3) natural ground sites. The statistical evaluation of the field testing program data and the corrective measures recommended for their proper cross-correlation are presented.
CHAPTER 1

INTRODUCTION

1.1. Objectives of the Investigation

The objectives of this investigation are to introduce, evaluate the field applicability, and proceed to a preliminary laboratory test program regarded as the basis for the calibration of an in situ testing equipment hereafter referred to as the "Miniature Quasi-Static Cone Penetrometer (MQSC)". The MQSC is a 1.27 cm² cross-section cone penetrometer directed to road and highway engineering applications in the State of Louisiana, such as classification of natural grade soils and determination of engineering properties of compacted embankments. Concisely, the purpose of this project can be stated as follows:
(1) To perform a field testing program in order to evaluate the performance of the MQSC when compared with the performance of the reference 10 cm² cross-section cone penetrometer. The soils involved in the field test program embrace the range of natural grade soils and compacted embankments. The evaluation of performance of a 15 cm² cross-section Fugro friction cone penetrometer which has been accepted by the geotechnical community during the last decade when compared with the reference cone is also accomplished in this field testing program;

(2) To perform a preliminary laboratory test program on a selected type of soil (i.e. artificial mixture of clay, silt and sand) in order to establish the basis for developing correlations between cone penetration resistance and soil parameters of interest to the construction control of embankments, such as: maximum density ($\gamma_{\text{max}}$) and California Bearing Ratio (CBR). This initial laboratory test program encompasses the following steps:

(2.1) To develop a reliable, task-specific laboratory apparatus, a flexible wall calibration chamber, suitable for allowing calibration of the MQSC in different types of soils. These types of soils comprise the range of field soils that are under the scope of applicability of the quasi-static cone penetration test, i.e. sand, silt, clay and combination thereof, as usually reported in geotechnical engineering;

(2.2) To perform an introductory laboratory test program with a selected soil, using the apparatus introduced in item (2.1) to measure the MQSC's cone
penetration resistance (i.e., cone resistance and local side friction resistance) in samples "compacted" at the standard Proctor compaction effort, consolidated at the vertical stress level of 210 kPa (30 psi), and tested under the calibration chamber boundary conditions 1 and 3, which are universally accepted for calibration chamber investigations. Specifics of these boundary conditions are further detailed in Sections 2.3 and 2.5.

(3) To project possible extensions of the work undertaken in this study for future investigations.

1.2. Scope

This research introduces, evaluates the field applicability, and initiates a preliminary laboratory testing program to calibrate a miniature electric cone penetrometer, the MQSC, to be used in the design and construction control of roads and highways in the State of Louisiana. The innovative aspects of this research are, as follows:

(1) Introduction of an in situ testing apparatus, the MQSC, for routine road design and construction control;

(2) Development of a versatile field testing program to analyze and evaluate
the topic of scale effects between the MQSC, and the Fugro 15 cm² cross-section cone, and the reference cone penetrometer;

(3) Development of a fully computerized flexible wall calibration chamber system, hereafter referred to as "The Louisiana State University Calibration Chamber System - LSU/CALCHAS". This task involved fabrication and implementation of a flexible wall calibration chamber, design and construction of a control panel for the calibration chamber, and the development of computer software capable of simulating the traditional calibration chamber boundary conditions commonly referred in the literature (i.e. BC1, BC2, BC3, BC4);

(4) Implementation of compacted sample preparation techniques. This task encompassed the development and fabrication of an automatic tamper of large dimensions for preparation of soil samples to be tested in the LSU/CALCHAS. This equipment enables the simulation of the compaction efforts commonly utilized in construction of transportation facilities.

The purpose of the field testing program was to analyze the performance of the 1.27 and 15 cm² cross-section cones in comparison with the performance of the reference cone penetrometer. Natural grade soils and compacted embankments of roads and highways in the State of Louisiana were investigated in this program of study. The entire spectrum of soil deposits in the State of Louisiana could not conceivably be encompassed as a potential population for sampling purposes. The
site-specific spatial soil variability of the natural grade soils and compacted embankments also brought uncertainties to the problem of defining an optimal sample size. It is therefore not the intention of this research to develop a statistical procedure for sampling, a task which by itself would portray as a complex investigation topic. Consequently, engineering judgement was applied to the selection of soils from sites of natural grade and compacted embankments that could be viewed as representative of materials commonly encountered in transportation engineering in the State of Louisiana.

The laboratory test program of this research provided the physical basis, i.e. the LSU/CALCHAS, for the potential development of correlations between cone penetration resistance and soil parameters of interest to road engineering. Cone and local side friction resistances are expected to be related to factors affecting the compaction characteristics of soils, such as: soil type, water content, compaction effort, and lift thickness. However, the time frame available for this research does not permit to analyze the effect of each and everyone of these parameters on cone penetration resistance. Therefore, these topics are left out of the scope of this investigation.

1.3. Organization of the Dissertation

In order to meet the objectives previously stated, Chapter 2 of this
dissertation presents an extensive literature review regarding aspects of electronic cone penetrometers, cone penetration testing/interpretation, calibration chamber testing/interpretation, principles of capillarity in soils, and statistical techniques historically applied to the analysis of geotechnical data. Chapter 3 describes the field testing program, displays the field test results and the related discussions. Chapter 4 delineates the laboratory testing program, depicts in detail the LSU/CALCHAS and the automatic tamper developed for this research, presents laboratory test data and provides pertinent discussion of the data. Chapter 5 summarizes the results/accomplishments of this research and recommendations for further research. Finally, Chapter 6 embodies the conclusions of this investigation. For clarity, tables and figures related to the chapters of this dissertation are presented at the end of respective chapters. It was opted to present cone penetration resistance in kg/cm² in the field and laboratory testing program of this dissertation. These are the units commonly used in the LSU's Research Vehicle for Geotechnical In Situ Testing and Support (REVEGITS) cone penetrometer sounding output.
CHAPTER 2

LITERATURE REVIEW

2.1. Introduction

There is no standard procedure to classify and determine the engineering properties of subgrade soils in road engineering. However, it is generally accepted that any method should supply enough information to define the geotechnical characteristics and the boundaries of each significant soil layer. Tests for determining the required soil parameters in road engineering may be divided into three groups:

(1) The first group refers to tests such as plate bearing, triaxial and oedometer to define the bearing capacity and consolidation characteristics of soils
supporting roadway foundations. They can be conducted insitu or in laboratory on
the so called "undisturbed samples". However, their application in road engineering
is restricted to bridge-foundation problems and special studies.

(2) The second group includes the procedures to classify, characterize and
determine the trafficcability of disturbed samples in laboratory. They usually
encompass grain size analyses, sand-equivalence, Atterberg limits, Proctor
compaction test and California Bearing Ratio (CBR).

(3) The third group contains the procedures for construction control, involving
in situ determination of density, moisture content and field CBR.

Although laboratory tests are the most commonly used procedures in road
engineering, in situ tests are frequently considered to be more economical, expedient,
reliable and repeatable. It would be advisable to expand the field of application of
in situ tests in road engineering, and undoubtedly it might be valuable to introduce
an in situ testing device for site characterization of natural grade soils and
construction control of embankments.

The geotechnical field of application of various in situ tests as lately
summarized by Jamiolkowski et al. (1985) is depicted in Table 1.1. It is possible to
infer that only the cone penetration test (CPT), in its different versions, has high
applicability for continuous soil profiling and identification. It is also referred in the
literature that the use of CPT would be advisable for judging compaction characteristics of embankments (Schmertmann, 1975; ISSMFE, 1988; and Mlynarek et al., 1988).

The literature review addresses topics related to the cone penetration test, calibration chamber studies, behavior of partially saturated soils, and statistical analyses of geotechnical data. This review seeks to provide the general framework for the present research topic which specifically deals with scale effect on CPT, preparation and testing of partially saturated compacted soil samples in large calibration chambers, and the pertinent statistical analysis of the testing data.

2.2. The Cone Penetration Test

The CPT is usually carried out by using three types of cone penetrometers: the dynamic, the static or mechanic and the quasi-static or electronic cone penetrometers. The range of application of the CPT as a function of soil strength is delineated in Figure 1 (Tumay, 1987). It emerges that the dynamic mode applies to the study of soils exhibiting medium to high strength; the static and quasi-static modes relates to the investigation of soft to high strength soils.

The use of light dynamic cone penetrometers (DCP) to design and construction of roads is extensively reported in the literature (Scala, 1956; Van
Vuuren, 1969; Kleyn, 1975; Kindermans, 1976; Kleyn et al., 1982; De Henau 1982; Rohm, 1984; Chua, 1988; Kleyn and Van Zyl, 1988; Livneh and Ishai, 1988; and Smith, 1988). However, at the actual state-of-the-art, reliability of DCP's test "... has to be improved, compared with static cone penetration testing" (Stefanoff et al., 1988), and DCP's test results still "... cannot be readily and accurately used for quantitative analysis of soil properties" (Tumay, 1987).

The upper range of application of the static and quasi-static modes is governed by the fact that high thrust loads are required to penetrate soils of high strength. In general, high load capacity and relatively heavy field CPT based systems have to be used for testing soil conditions. The mobility of these systems on the natural ground is problematic under moderate to severe field conditions, thus limiting their field application. Considering road engineering applications in the State of Louisiana, and regarding that the required load system to penetrate the soil mass is a direct function of the cone penetrometer dimensions, it is patent that a small size cone penetrometer could expand the field application of the static and quasi-static modes.

The reference electronic cone penetrometer has a 10 cm² cone tip (35.7 mm in diameter) with an apex angle of 60 °. The friction sleeve, located above the conical tip, has a surface area of 150 cm². These are the general accepted standards in Europe (ISSMFE, 1977), and in the United States (ASTM, 1979), and are reported in a suggested international test procedure (ISSMFE, 1988). On the other
hand, many of the small size cone penetrometers used for research purposes exhibit diameters between 10 to 25 mm, friction sleeve length between 50 to 100 mm, apex angle of 30° or 60°. The cone and local side friction resistances are measured externally or internally to the cone penetrometer by means of load cells or similar mechanisms (Melzer, K. J., 1974; Rohani and Baladi, 1981; Parkin and Lune, 1982; Miura et al., 1984; Almeida and Parry, 1985; Bellotti et al., 1985; Rad and Tumay, 1986; Sweeney, 1987; Eid, 1987; Canou et al., 1988; Peterson, 1988). Guidelines for CPT practice is available in Schmertmann (1978b), Robertson and Campanella (1984), and ISSMFE (1988).

For saturated soils, it is generally assumed that cone penetration test at the standard rate of 2 cm/sec functions under drained condition for sands, and under undrained conditions for clays. In silts and clayey silt soils it is expected that a dependence would exist between cone resistance and penetration rate. Campanella and Robertson (1983) have shown that penetration is "...undrained down a penetration speed of about 0.2 cm/s..." for a clayey silt deltaic soil with 70% of silt, 20% of clay and 10% of sand fractions. Konrad et al. (1985) also concluded that cone resistance, local side friction resistance and pore pressure response are dependent on penetration rate when testing in deltaic clayey silts of low plasticity.
2.3. Calibration Chambers

Calibration chambers came into the picture of cone penetration test almost two decades ago. Reasons behind the use of such a calibration device can be stated as follows:

(1) The crescent application of cone penetrometers in geotechnical engineering revealed the necessity of developing a testing device and a procedure to calibrate them under controlled boundary conditions, avoiding the unknown factors common to field calibration;

(2) Cone penetration data can give direct evaluation of bearing capacity (Nottingham, 1975; Vesic, 1977; Schmertmann, 1978b; Tumay and Fakhroo, 1981), but its conversion into basic soil properties such as friction angle and deformation modulus requires calibration under strictly controlled boundary conditions.

Calibration chambers which permit the separation of the effects of increased effective stress level and increased density were introduced by Holden (1971). In general, two types of calibration chambers are used for this purpose: rigid and flexible wall.

A rigid wall calibration chamber imposes a boundary condition of zero lateral strain on the specimen under testing. However, in order to give cone penetration
resistance without the influence of chamber size, a chamber of considerable dimensions is required, and a diameter ratio (i.e., calibration chamber diameter/cone diameter) of 200 is referred in the literature (Holden, 1971; Chapman, 1974). The same authors concluded that a smaller calibration chamber could be used and still give cone resistance that would be compatible with field measurements if the flexible wall option was adopted.

The design of a flexible wall calibration chamber permits accurate control and measurement of the vertical and horizontal stresses and strains. In general, four boundary conditions can be simulated in calibration chamber testing (Bellotti et al., 1985):

- **BC1**: \( \sigma_v = \text{constant} \quad \sigma_h = \text{constant} \)
- **BC2**: \( \varepsilon_v = \varepsilon_h = 0 \)
- **BC3**: \( \sigma_v = \text{constant} \quad \varepsilon_h = 0 \)
- **BC4**: \( \varepsilon_v = 0 \quad \sigma_h = \text{constant} \)

From a historical point of view, the first large flexible wall calibration chamber was built by the Country Roads Board of Victoria (C.R.B.), Australia, housing a cylindrical sample of 760 mm in diameter by 910 mm in height (Holden, 1971). Unavoidable boundary effects in the C.R.B. chamber, when calibrating the 20 cm² C.R.B. penetrometer, led Holden to design a new calibration chamber at the University of Florida, U.S.A., housing a sample of 1,220 mm in diameter by 1,220
mm in height. In spite of this new design, it was not possible to reach a plateau for sleeve friction resistance when testing the 10 cm² Fugro model A penetrometer. In order to solve this problem, a higher chamber was built at the Monash University, Australia, housing a sample of 1,820 mm in diameter by 1,220 mm in height (Chapman, 1974). The Norwegian Geotechnical Institute (N.G.I.), Norway, also constructed a calibration chamber 1,219 mm in diameter and 1,500 mm in height (Parkin and Lunne, 1982) to study boundary effects in the laboratory calibration of cone penetrometers in sand. Another calibration chamber was built by the Italian National Electricity Board - Hydraulic and Structural Research Center, Italy, housing a sand sample of 1,200 mm in diameter by 1,500 mm in height (Bellotti et al, 1982).

The last, and biggest calibration chamber, was designed and assembled at Virginia Polytechnic Institute and State University, Virginia-U.S.A., accommodating a sand sample of 1,500 mm diameter by 1,500 mm height (Sweeney, 1987). A small calibration chamber, also for sand and housing a sample of 760 mm in diameter by 800 mm in height, was built at University of California at Berkeley, U.S.A., (Villet, 1981). The first unique calibration chamber to test clayey soils was developed at Purdue University to calibrate a miniature pressuremeter, and houses a sample 203.2 mm in diameter by 337 mm in height (Huang, 1986).

Primarily, the chambers mentioned above, with the exception of Huang chamber, have similar features and were developed for testing solely sand samples. They differ only on the degree of advanced instrumentation and versatility of controls installed, and can be used for testing a large gamut of in situ testing.
2.4. CPT Interpretation

The high applicability of CPT in soil profiling is well known in engineering practice. The first use of friction ratio, i.e. the ratio of local side friction resistance to cone resistance, to determine the soil subsurface stratigraphy is credited to Begemann (1965). Since then, soil classification charts generally based on the reference cone penetrometer data have steadily been developed and modified as the CPT’s state of the art advanced. Examples of these charts can be found in Schmertmann (1978b), Douglas and Olsen (1981), Tumay (1985), Robertson et al. (1986) and Olsen and Malone (1988).

CPT field data has demonstrated that local side friction resistance changes drastically with soil type. This fact has led to the use of friction ratio and cone resistance as basic parameters in CPT soil classification charts. However, as referred by Parkin (1988), experimental data on sands has shown that local side friction resistance is not only dependent on cone resistance but on cone size as well. The further dependence of friction resistance on cone size certainly could maximize the importance of local side friction resistance on the analysis of soil properties when considering data obtained with different cross-section area cones.
Schmertmann (1978b) showed that the reference cone penetrometer requires a depth of about seven cone tip diameters to fully mobilize the cone penetration resistance in the same sand, after passing from a higher relative density state \(Dr = 100\%\) to a medium relative density state \(Dr = 55\%\); for clays, Schmertmann also mentioned that the complete transition in a two layer clay system could occur between two and four cone diameters. Considering the reference cone penetrometer and a layer of stiff soil embedded in a soft mass and a layer of soft soil embedded in a stiff mass, Jamiolkowski et al. (1985) referred to thicknesses of at least 700 and 200 mm, respectively, in order to fully mobilize the cone penetration resistance at the middle height of each layer.

In order to develop correlations between calibration chamber's cone penetration resistance and soil parameters, particular cone penetration resistance values have to be obtained from the chart records of \(q_c\) and \(f_s\). Working on loose and dense sand samples (i.e., \(Dr = 30\%\) and \(95\%\), respectively), and using a 10 mm diameter cone penetrometer and a diameter ratio of 18, Canou et al. (1988) demonstrated that the 'plateau' condition for cone resistance can be reached at depths around 60 mm. In contrast, Parkin (1988) argues that dense sands samples tested at constant vertical stress and zero lateral strain (i.e., BC3) "do not reach a 'plateau' condition, but have \(q_c\) increasing continuously in response to an increasing lateral stress." If a 'plateau' condition is not attained, generally mid-depth values of cone penetration resistance are used for correlation with soil parameters.
The penetration resistance mechanism of cone penetrometers in soil deposits has been approximated by bearing capacity theories (Durgunoglu and Mitchell, 1973; Janbu and Senneset, 1974; Senneset et al., 1982) and cavity expansion theories (Vesic, 1972; Baligh, 1976; Baligh and Levdoux, 1980). An extensive review of the current practice is available in Keaveny (1985) and Jamiolkowski et al. (1988). An empirical approach based on the so called "state parameter" and describing the relationship between friction angle of sands in terms of effective stress and cone resistance is also available (Been et al., 1986).

Attempts have been made to estimate the strength and deformability of cohesionless soils from traditional laboratory tests and CPT results. For normally consolidated cohesionless soils, Meyerhof (1974) presented an empirical correlation between internal friction angle and cone resistance from the results of investigations developed in Europe. Based on the Durgunoglu and Mitchell theory, Villet and Mitchell (1981) obtained a significant correlation between the internal friction angle of fine sands obtained from CPT results and the measured ultimate triaxial friction angle, for identical stress and void ratio conditions. Considering Toyoura standard sand and various sand fabric characteristics, Miura et al. (1984) concluded that there is a unique relationship between the internal friction angle obtained from drained triaxial compression tests and cone resistance obtained from calibration chamber tests. Also, these authors referred that "...the characteristics of dilatancy due to the cone penetration are intimately connected to those in the static triaxial test." Based on laboratory triaxial and calibration chamber tests on a large range of sand types,
Robertson and Campanella (1984) proposed an empirical correlation between a factor called "bearing capacity number" (i.e., Nq) and peak friction angle obtained from drained triaxial tests "...performed at confining stresses approximately equal to the horizontal effective stress in the calibration chamber before cone penetration."

From the analysis of a large range of sand types, Been et al. (1986) recommended empirical correlations between the normalized cone resistance and the so-called "state parameter" which is a concept developed on a data base of triaxial tests. Also from laboratory tests on Monterrey sand No. 0, Rad and Tumay (1986) gave empirical correlations between cone penetration resistance and friction angle obtained from triaxial tests. From field cone penetration testing on a dune deposit of fine to medium silty sand, Johnson (1986) proposed regression equations relating triaxial friction angle to cone penetration resistance. Relationships between the drained secant Young's modulus and cone resistance are also available (Robertson and Campanella, 1984; Bellotti et al., 1985).

The influence of compaction characteristics on the undrained behavior of a compacted clay has been studied by Pastor and Uriel (1983). These authors concluded that a compacted clay behaves like an overconsolidated clay, and the equivalent OCR is a function of the compaction effort and water content. These authors also suggested the existence of a "...well defined Critical State for the compacted, saturated and consolidated clay tested.\text{,}" and concluded that the assumption of a parallel virgin consolidation line to the critical state line could lead to the estimation of the OCR generated by the compaction procedure used.
2.5. Boundary Effect in Calibration Chamber Tests

A major problem in calibration chamber tests is the definition of an equivalent boundary condition when comparing field to results obtained in the laboratory calibration. Holden (1971) considered that the actual boundary condition in the field would lie somewhere between the laboratory calibration chamber test BC1 and BC3. He also assumed that the true field cone resistance "...would lie at the third point of the range between the two extremes obtained from 'zero-stress-change' and 'zero-deformation' tests...", i.e. BC1 and BC3, respectively. From comparison of data from field and calibration chamber tests performed on sands, Harman (1976) concluded that the third point between the range of BC1 and BC3 is in fact the best approximation of in situ conditions. Schmertmann (1978.a) has also attempted an interpolation between calibration chamber test results obtained under boundary conditions BC1 and BC3 when developing correlations between relative density and cone penetration resistance in sands.

Parkin and Lunne (1982) understood that for a sample of ideal dimensions, cone resistance measured in a flexible chamber should be independent of boundary conditions. These authors suggested that for a sample size smaller than the ideal one, i.e. a sample that should not be subject to any boundary effect in calibration chamber tests, cone resistance obtained from tests under boundary conditions BC1 and BC3 simulated in flexible wall calibration chambers could represent boundaries for the actual in situ cone resistance. This in agreement with Holden's assumption.
Bellotti et al. (1985) considered that the relationship between the true field boundary conditions and those feasibly simulated in flexible wall calibration chambers is still not well understood because of complex interactions between the so-called "chamber size effect" and boundary conditions.

From experimental data, it was concluded that in order to prevent boundary effects on cone penetration resistance in a flexible wall calibration chamber, diameter ratios of 20, 50 and 100 would be required for loose (Dr = 15 - 30%), normally consolidated dense (Dr = 90%) and overconsolidated sands (OCR ≥ 8), respectively (Parkin and Lunne, 1982). These experimental conclusions have evolved along of the years. Considering a large range of new experimental data Parkin (1988) reports that even for a diameter ratio of 60, calibration chamber tests performed in dense sand (Dr between 80 and 90%) could still be affected by boundary conditions. Tumay et al. (1985), in their treatise of the flow field around a cone penetrating an inviscid and incompressible fluid, concluded that a diameter ratio around 20 was required to dissipate the strain rates induced during the penetration of a 60 ° apex angle cone. In order to prevent boundary effects when developing his laboratory pressuremeter experiments in clay soils, Huang (1986), based on Carter et al. (1979) solution for the expansion of a cylindrical cavity in a semi-infinite medium, adopted a diameter ratio of 18.

Correction factors for chamber size effects are available for sands (Bellotti et al., 1985; Been et al., 1986).
2.6. Scale Effect in CPT

The term "scale effect" is generally associated with the influence of cone penetrometer dimensions on penetration resistance. Although this topic has been repeatedly analyzed in the literature during the last two decades, a comprehensive literature survey revealed that it still is an unsettled issue.

The factors affecting the cone penetration resistance of electronic cone penetrometers can be summarized as soil type and its environment, equipment type, and test procedures. Changes in cone penetrometer dimensions may influence the measured cone penetration resistance (i.e., cone resistance and local side friction resistance). Available correlations developed between the reference penetration resistance and related soil properties certainly need to be verified for possible scale effect(s). Consequently, the use of small size cones for assessing soil engineering properties may require to carry on a laboratory and a field test program in order to evaluate possible scale effects between the reference and miniature cone data.

Kerisel (1961) reported decrease in penetration resistance with increase in pile diameter in homogeneous compacted sand samples, considering pile diameters ranging from 45 to 320 mm. Sanglerat (1972) addressed the scale effect topic stating that cones with different cross-section areas (5 to 40 cm²) give almost the same cone resistance in all soils. Holden (1977) suggested that a small size cone penetrometer could give higher penetration resistance than the reference probe under normal field conditions. Schmertmann (1978b) referred to "no significant variation" in the...
measured penetration resistance of cones with cross-section areas in the range of 5 to 20 cm², for all soil types. Shields (1981) implied that the influence of cone diameter in the measured cone resistance would be little in an isotropic, homogeneous and uniform soil, and that this influence could be expected to increase in layered soil systems. Considering sounding data accumulated along the years, De Ruiter (1982) reported "...that there are no significant differences in qₑ and fₑ for cone sizes varying from 5 to 15 cm²." Based on field tests performed with the 10 and 15 cm² McClelland piezocones at Brent Cross Hendon (London clay), Lunne et al (1986b) stated that "...there are no significant differences in the cone resistance and sleeve friction values between the 15 and 10 cm² piezocones." Sweeney (1987) referred to scale effects between a 4.1 cm² and a 10 cm² cone penetrometers. From field tests in two sites at the Imperial Valley in Southern California, soil profiles classified as "soft silt to clayey silt" / "loose to compact silty sand" and "loose sandy silt", this author noticed that the scale effect "was more noticeable in the tip resistance than in the sleeve friction; also, the effect became more pronounced as higher values of qₑ are compared." Same conclusion was reached by the referred author from his laboratory penetration testing program on sands in a large size calibration chamber (Monterrey #0/30 Sand, Dr = 24% and 65%), for cone resistance. Although the laboratory test program was not conclusive about scale effects on local side friction resistance, Sweeney reported that "...it appears no pronounced scale effects exists for sleeve friction...". From cone penetration tests performed at Dunkerque with 10 cm² and 15 cm² cones (QCPT, PCPT and DPCPT), Juran and Tumay (1989) concluded that cone diameter does not affect cone
resistance and excess pore water pressure measurements, but can affect the measured local side friction resistance. These authors indicated that the 10 cm² cone systematically gave sleeve friction measurements 20% higher than the 15 cm² cone penetrometer. The Earth Technology Corporation reported comparison between the reference penetrometer and a 1.27 and a 2.85 cm² cross-section cones from tests performed "... under stress-controlled soil conditions in a 30 inch diameter drum." They mentioned that "... the tip resistance measurements were essentially the same as for full size cones and the friction measurements were consistently about 10 % higher than measured by full size cones."

The topic scale effect was also analyzed by Parkin (1988), in his special lecture addressed to the First International Symposium on Penetration Testing, ISOPT-1. From a theoretical approach and considering the Terzaghi bearing capacity equation for a circular footing, the referred author stated that "...any scale effect would have to be embodied in the term involving Nₗ, which is effectively insignificant at D/B ratios above, say, 10 (i.e., all practical situations involving the CPT)." Complementing, Parkin suggested that "Perhaps the only remaining source of a scale effect (as opposed to chamber size effect, discussed elsewhere) is the relationship between cone size and the size of sand grains..", stressing however that "...this possibility is unlikely to be one of serious practical consequence."

It has been reported that the wear of the friction sleeve and cone tip would influence friction ratio in sands. From an extensive field testing program carried out
in 1982, using an electric cone penetrometer, Jekel (1988) referred to a 45% decrease in friction ratio after a penetration length of 500 m in sand. From a recent field soil investigation performed in a sand soil profile, the same author reported local side friction resistance reduction of approximately 30%, and credited the decrease in friction resistance to the wear of the friction sleeve and cone tip.

2.7. Capillarity in Soils

A feature of interest of subgrade soils, at least at the usual design depths of interest, is their partially saturated condition. Generically, this statement can be extended to almost all compacted soils which translates to considerable amount of natural soils used in road engineering construction activities.

Analysis and design involving partially saturated soils have been often based on fully saturated soil properties. One reason for adopting this procedure is the lack of data and reliable analytical methods to predict behavior of unsaturated or partially saturated soils.

Basic theoretical considerations on partially saturated soils have been given by Bishop and Blight (1963) and Fredlund (1985). For a given compactive effort, Bishop and Blight (1963) noticed that the pore-pressure and suction characteristics of compacted soils are dependent mainly on four factors: soil type, compaction water
content, difference between total normal stress and pore-air pressure \((\sigma - u_0)\), and 
ithe applied shear stress. Fredlund understood that the shear strength and volume 
change behavior of an unsaturated soil could be described in terms of two independent 
stress variables, as follows: total normal stress and pore-water pressure \((\sigma - u_w)\), and 
pore-air and pore-water pressure \((u_a - u_w)\). Fredlund suggested that when the 
degree of saturation approaches 100\%, the pore-air pressure approaches the 
pore-water pressure, and a smooth transition to the saturated case is expected to 
occur.

Typical relationships between suction and compaction water content for 
cohesive soils have been given by Blight (1967), and for cohesionless soils by 
Lambe (1950) and Wu et al. (1984). Blight (1967) analyzed the effect of saturation 
on failure zone void ratio and maximum shear stress in triaxial tests on an unsaturated 
clayey sand. He showed that the saturated soil presents a noticeable change in 
shear strength parameters when compared to the unsaturated soil. He also concluded 
that clayey sands compacted dry of the optimum water content show a dependence 
between shear strength parameters and water content, in disagreement with Bishop 
et al. (1960)'s assumption. Wu (1983) showed that the magnitude of capillary stress 
developed in the Glacier Way Silt with a void ratio of 0.58 and degree of saturation 
of 80\% is approximately 6.5 kPa (9 psi).

The relationship between optimum moisture content and degree of saturation 
as defined in compaction tests has been referred in the literature. Lambe (1951)

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displayed compaction curves for a well-graded sandy silt, considering various upper limits of grain size. From these curves it is possible to conclude that, for cohesionless soils compacted around the optimum water content, the degree of saturation is at least 80%. Martin (1983) also reported that "...the optimum moisture content for compaction of fine grained soils is in the range of 80% to 90% saturation."

Shear strength of partially saturated soils is in general understood as a suction controlled parameter and highly dependent on changes in moisture content induced eventually by climatic loading variation. Wu (1983) referred that "...capillary pressure is often neglected in soil mechanics analysis when the air voids are separated. During consolidation, there is a successive occlusion of individual voids as effective stress increases." Barden (1965) reported in the case of a Vicksburg silty clay that presented fully continuous voids at a moisture content 4% below the optimum and occluded voids at moisture content 3% above the optimum.

Considering road engineering applications, a topic of interest in the present research is the influence of suction on the shear strength behavior of Louisiana compacted soils. Taking into account the suction measurements performed by Wu (1983) in a silt soil, it seems that the compaction of cohesionless soils for road engineering purpose should not generate suction levels in order to affect significantly the effective stress. Taking into account that construction of fills in humid regions tend to be on the upper side of the optimum moisture content, and considering that the case of the Vicksburg silty clay reported by Barden (1963) could represent the
state resulting from compaction of fine grained soils for road engineering purposes, it would be fair to assume that the generation of capillary pressure during compaction of fine grained soils in road engineering in Louisiana could not affect significantly the soil behavior in terms of effective stresses.

2.8. Statistical Analysis of Geotechnical Data

The present research envisions the statistical analysis of field testing data in order to examine the scale effect topic among cones with different diameters. It also involves the application of statistics techniques to calibration chamber and traditional geotechnical laboratory test data in order to develop meaningful correlations between CPT data and soil parameters of interest to road engineering. From this point of view, it would be of interest to present some comments on soil populations, and to expose a brief history of the application of statistical techniques in geotechnical engineering in the last two decades.

Engineering properties of soil masses can be understood as unknown functions of two or three spatial coordinates (Alonso and Krizek, 1975), and it is frequently assumed that "...most soil properties can be regarded as random variables conforming to the 'normal' or 'Gaussian' theoretical distribution." (Lumb, 1986). It seems that there is a tendency to describe the random heterogeneity of a soil mass on the grounds of probabilistic models (Alonso and Krizek, 1975; Vanmarcke, 1977). In
addition, it is referred that "...the type of problem, its symmetry conditions, and a knowledge of the soil at each site will dictate the simplifications that might be employed" in the model (Alonso and Krizek, 1975). Therefore, it is latent that engineering judgement is a requirement, but sometimes a pitfall, of these models. Engineering judgment is required, for example, in order to estimate the structure of the spatial soil variability of soil populations. Lumb (1975) considered that, for a single soil population, "...the minimum number of test results needed in order to give reasonably precise estimates is of the order of 10^4." However, the same author concluded that "...the best that can be done in practice is to study variability over one dimension, either laterally or vertically, using sample sizes of the order of 20 to 100." Also, Arman and McManis (1977), when studying the effects of disturbance caused by sampling and handling on the engineering properties of cohesive soils in Louisiana, stated that "...the selection of sites was based on an attempt to find soils conducive to a study on sampling, and representative of material encountered by soil engineers in the area."

Statistical techniques have been applied to increase the reliability of geotechnical engineering data in the last decades. Holtz and Schrode (1975) and Deer (1984) have shown the importance of Factor Analysis on the evaluation of soil test data. Harman (1976) applied Analysis of Variance and Regression Analysis in developing static cone bearing versus relative density correlations for fine sands. Sanglerat et al. (1982) used Regression Analysis to study the influence of soil properties on cone resistance during static sounding of cohesive soils. Also,
statistical procedures such as Path Analysis, Cluster Analysis, Factor Analysis, Factorial Analysis and Regression Analysis have been used by Johnson (1986) for developing a methodology to correlate cone penetration test data with other geotechnical parameters.
2.9. Tables and Figures
### Table 2.1. Present In Situ Capabilities for Soil Modelling (After Jamiolkowski et al., 1985)

<table>
<thead>
<tr>
<th>Soil Behaviour-Parameter</th>
<th>Equipment and/or Procedures</th>
<th>Comments - Remarks</th>
</tr>
</thead>
</table>
| 1. Soil profiling and identification | 1.1. CPTU | 1.1.a. Simultaneous measurement of qA and qN at very short penetration has great potential for soil profiling and identification.  
1.1.b. Essentially rigid and extremely well suited system with very quick response for reliable qN measurements.  
1.1.c. Correlation of qA and qN for unequal and axes effects. |
| | 1.2. CPT | 1.2.a. Good for soil profiling but less sensitive to strata changes in comparison to CPTU.  
1.2.b. Friction ratio qa/qc a poor soil type identifier in especially sensitive clays.  
1.2.c. Potential may be increased by improving resolution of qa measurements and more reliable and repeatable qc measurements. |
| | 1.3. DMT | 1.3.a. A sensitive soil identifier but, since performed discontinuously, generally every 20 cm, less sensitive to strata changes.  
1.3.b. Mainly for soil profiling and identification; needs further field and laboratory validation. |
| | 1.4. Acoustic cone | 1.4.1. Measure monodimensional electrical "formation factor" which reflects sand structure, hence the anisotropy, particle shape, void ratio, and cementation; may be relevant for liquefaction studies.  
1.4.b. Needs further validation, especially in the field. |
| | 1.5. Electric conductivity probe | 1.5.a. Measure monodimensional electrical "formation factor" which reflects sand structure, hence the anisotropy, particle shape, void ratio, and cementation; may be relevant for liquefaction studies.  
1.5.b. Needs further validation, especially in the field. |
| 2. In situ σho hence σo | 2.1. SPB (measures σho) | 2.1.a. "Proven" to be successful in soft clay; less experience in stiff clays; poor experience in sands.  
2.1.b. Greatest potential among in situ methods but still some problems with equipment compliance and probe insertion procedures.  
2.1.c. Based on empirical correlations; promising, but requires further research to assess reliability.  
2.1.d. New device; requires further intensive laboratory and in situ validation.  
2.1.e. Limited positive experience only in soft to stiff clays; successful use in other soils unlikely.  
2.1.f. In stiff clay overestimates σho; requires correction for bedding error.  
2.1.g. Vertical installation essential.  
2.1.h. Applicable only to cohesive soils having σo < σho.  
2.1.i. Interpretation uncertain. |
| | 2.2. DMT (assesses σo) | 2.2.b. Possible applications limited to relatively homogeneous cohesionless deposits at shallow depths in which tests are performed under fully drained conditions.  
2.2.c. For SPB the influence of plate shape and disturbance due to its installation on the load-settlement relationship not well understood.  
2.2.d. Eq 3.4.6. may be correlated to OCR in homogeneous cohesive deposits; reflects OCR changes.  
2.2.e. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| | 2.3. ISB (assesses σho) | 2.3.a. Limited experience for assessment of σo.  
2.3.b. Possible applications limited to relatively homogeneous cohesionless deposits at shallow depths in which tests are performed under fully drained conditions.  
2.3.c. For ISB the influence of plate shape and disturbance due to its installation on the load-settlement relationship not well understood.  
2.3.d. Eq 3.4.6. may be correlated to OCR in homogeneous cohesive deposits; reflects OCR changes.  
2.3.e. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| | 2.4. Spade-like TSC (measures σho) | 2.4.b. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| | 2.5. Hydraulic fracturing, (assesses σho) | 2.5.a. "Proven" in cohesionless deposits in which can determine average drained young stiffness E', within the depth of influence of the plate.  
2.5.b. In cohesive soils, despite uncertainty about drainage conditions, it is assumed to yield average undrained young stiffness E.  
2.5.c. Since E is obtained from load-displacement measurements, an a priori assumption regarding soil constitutive model is necessary.  
2.5.d. Very difficult to refer the E obtained from PL and SPL tests to the behaviour of a soil macro-element, hence to strain or stress levels.  
2.5.e. Great potential for direct measurement of shear modulus G in horizontal direction.  
2.5.f. G, describing the "elastic" soil behaviour can be assessed from small unloading-reloading cycles whose role is to minimize the soil disturbance due to probe insertion. |
3.1.b. Possible applications limited to relatively homogeneous cohesionless deposits at shallow depths in which tests are performed under fully drained conditions.  
3.1.c. For PL the influence of plate shape and disturbance due to its installation on the load-settlement relationship not well understood.  
3.1.d. For SPL the influence of plate shape and disturbance due to its installation on the load-settlement relationship not well understood.  
3.1.e. Through empirical correlations yields values of tangent constrained modulus in sands and clays.  
3.1.f. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| | 3.2. CPTU | 3.2.a. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| 4. Deformability characteristics | 4.1. PL and SPL | 4.1.a. Application limited to shallow depths.  
4.1.b. "Proven" in cohesionless deposits in which can determine average drained young stiffness E', within the depth of influence of the plate.  
4.1.c. In cohesive soils, despite uncertainty about drainage conditions, it is assumed to yield average undrained young stiffness E.  
4.1.d. Since E is obtained from load-displacement measurements, an a priori assumption regarding soil constitutive model is necessary.  
4.1.e. Through empirical correlations yields values of tangent constrained modulus in sands and clays.  
4.1.f. In any case applicable only to predominantly quartz sands un cemented sands in which penetration occurs under fully drained conditions. |
| | 4.2. SPB tests | 4.2.a. Through empirical correlations yields values of tangent constrained modulus in sands and clays.  
4.2.b. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| | 4.3. DMT | 4.3.a. Through empirical correlations yields values of tangent constrained modulus in sands and clays.  
4.3.b. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| | 4.4. CPT | 4.4.a. Through empirical correlations yields values of tangent constrained modulus in sands and clays.  
4.4.b. Possible applications limited to cohesive deposits; further laboratory and field validation needed. |
| 4.5. Shear wave velocity measurements | 4.5.a. "Proven" potential to evaluate small strain (v<0.5%) G in horizontally layered soil deposits.  
4.5.b. The value of G is calculated after assumption are made concerning the constitutive soil model, the travel path and the soil homogeneity.
Table 2.1. CONT.

<table>
<thead>
<tr>
<th>Soil behaviour-Parameter</th>
<th>Equipment and/or Procedure</th>
<th>Comments - Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>5. Flow and consolidation</td>
<td>5.1. Borehole</td>
<td>5.1. Outflow tests at constant head preferred; interpretation above water level extremely complex</td>
</tr>
<tr>
<td>5.2. Large scale pumping</td>
<td>5.2. Very reliable but also very expensive test; accurate well installation and drawdown measurements with piezometers are required</td>
<td></td>
</tr>
<tr>
<td>5.3. Plimeters</td>
<td>5.3. Constant head tests with as small to avoid fracturing are preferred; parameters from outflow tests relevant to OC conditions; inflow tests appropriate for NC conditions</td>
<td></td>
</tr>
<tr>
<td>5.4. Self-boring permeameter</td>
<td>5.4. Careful installation required; only outflow tests available, hence the derived parameters are relevant to OC conditions</td>
<td></td>
</tr>
<tr>
<td>5.5. Building test (Cakometer)</td>
<td>5.5. Careful installation required; difficult interpretation due to non-monotonic changes of effective stress</td>
<td></td>
</tr>
<tr>
<td>5.6. Piezcone or piezometer probe</td>
<td>5.6. Very economical and great repetitibility; great care required when performing test and interpreting field measurements</td>
<td></td>
</tr>
<tr>
<td>5.7. Back-analysis of full-scale structures</td>
<td>5.7. Uncertainties related to initial excess pore pressure or to final consolidation settlements; methods based on consolidation rate need careful analysis of experimental data</td>
<td></td>
</tr>
</tbody>
</table>

**SYMBOLS:**
- CPTU = Quasi-static cone penetration test with pore pressure measurements
- CPT = Quasi-static cone penetration test
- DMT = Marzhefski's flat dyanometer
- SB = Iowa tapped blade
- TSC = Total stress cell
- Clean = screw plate loading
- ESP = Self boring pressuremeter
- PD = Plate loading
- $\sigma_{HH}$ and $\sigma_{HP}$ = respectively, total and effective in situ horizontal stresses
- $\sigma_{top}$ = Effective overburden stress

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Figure 2.1. Range of Application of Different Cone Penetration Test (After Tumay, 1987)

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CHAPTER 3

FIELD TESTING PROGRAM

3.1. Introduction

The quasi-static cone penetration test (QCPT or CPT for short), in its different versions, is commonly used in geotechnical engineering in preliminary and advanced phases of subsurface soil investigations. The simplicity and cost-effectiveness of the test, the quality of the measured data and the reliability of the developed interpretation procedures have made the CPT an outstanding tool to determine the soil conditions in general, such as classification and stratification, and basic geotechnical parameters. Soil classification and soil strength parameters charts based on cone resistance ($q_c$) and friction ratio ($R_f$) obtained with the standard 10 cm² cross-section (reference) cone penetrometer are readily available to the practicing engineer.
The primary purpose of the field testing program was to analyze the adequacy of a lightweight, portable, electronic CPT based system using the Miniature Quasi-Static Cone Penetrometer (MQSC) to field conditions related to road engineering, i.e. natural grade soil classification and embankment construction control. A weakness in using this idea could be the possible existence of scale effect between the MQSC and the reference cone penetrometer data, endangering the applicability of the 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 15 cm² cross-section cone penetrometer in use by the Louisiana Transportation Research Center (LTRC) and the Department of Civil Engineering-LSU when compared with the reference cone penetrometer.

The field testing program implemented to examine the scale effect topic involved a comparative study of field performance between the MQSC and the 15 cm² cone, and the reference cone penetrometer. The reference cone penetrometer was used as the basis for comparison of performance. Statistical correlations between the MQSC and the 15 cm² cones 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 in road engineering.
Two more questions were concisely addressed in this field testing program. They were related to the influence of the pore pressure generated during cone penetration (i.e., unequal end area ratio) and wear of cone tip and friction sleeve on cone penetration resistance.

3.2. Equipment and Procedures

3.2.1. Penetration System

3.2.1.1. The LSU REVEGITS Penetration System

Penetration tests with the reference and the 15 cm² probes were performed using the Louisiana State University Research Vehicle for Geotechnical In-situ Testing and Support hereafter referred to as REVEGITS. This is a 20 tonne all wheel-drive vehicle which incorporates the state-of-the-art technology for in situ subsurface soil exploration for civil and geo-environmental engineering purposes. 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. REVEGITS' design includes hydraulic levelling and CPT operation with a 1 meter stroke penetration chucking system. The hydraulic leveling system consists of three jacks; two mounted behind the driver's cab,
connected to two .5 m (around 20") reaction pads and one jack at the rear of the vehicle frame with a .5 m reaction pad (see Figure 3.1).

REVEGITS penetration thrust system consists of two double acting hydraulic cylinders (Hyson 200 kN, Type III) with a cross beam, support columns with platen for loading head, and columns aligned by roller bearings. The maximum drive and pulling loads are 200 kN and 260 kN, respectively. The thrust system is attached to a support subsystem mounted to the vehicle frame. The clamping device is capable of penetrating and extracting rods of 35.6 mm and 55 mm in diameter. It can manipulate both sizes of the rods without changing the push-in head and without changing the electrical cable of the cone penetrometer. 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).

Considering Louisiana soils, general sounding depths on the range of 45 to 50 m can be reached, using 1 m high tensile strength seamless steel rods. One operator can perform all the testing tasks, although experience suggests two operators as an economic-time optimum number.
3.2.1.2. The MQSC Truck Penetration System

The MQSC truck system developed by Fugro-McClelland Engineers consists of a 1.27 cm$^2$ cross-section cone penetrometer, 10.6 m (35 ft) long coiled push rod, grabbing system and roller unit working on the push rod, and hydraulics. This equipment is mounted as a unit in the front of a modified standard pick-up truck. The pick-up truck provides mobility for the light utility cone, reaction force during penetration, and power for the hydraulics and data acquisition system.

The miniature cone push rod is a 9.53 mm (3/8 in) diameter 304 stainless steel rod with 4.76 mm (3/16 in) internal diameter. It is connected to the cone tip, down hole, and to a 10 pin electric plug on the up hole end. The rod is coiled into approximately 60 cm (2 ft) diameter coils.

The hydraulic system consists of three drive jacks. Lowering a transverse steel plate to the ground and raising the front of the truck, the two outer jacks provide for the reaction force during penetration. The center jack is the push jack. It allows the cone and push rod to pass through, has a stroke of around 15 cm (around .5 ft), and can deliver a set push rate of 2 cm/sec. The push jack provides the force needed to run the push rod through a roller unit and to penetrate the soil. The push jack is connected to a sleeve which contains the chucking system. The rollers in the roller unit combined with the chucking system straightens the coiled push rod. The rod then passes through a support guide before entering the soil.
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. Figure 3.3. shows details of the Fugro-McClelland miniature cone penetration system.

3.2.2. Data Acquisition and Reduction System

3.2.2.1. The LSU REVEGITS Data Acquisition and Reduction System

REVEGITS data acquisition hardware embodies a signal conditioning unit (PCU-M) manufactured by Fugro-McClelland of the Netherlands, a Compaq Portable III micro computer with a 640x400 high resolution screen, running at 12 MHz, a forty megabyte internal hard disk drive for data storage, and the Data Translation DT-2801A analog to digital conversion and digital I/O board. Signals coming from the cone penetrometer are amplified and scaled by the PCU-M unit 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 offline high quality CPT hardcopy. The data acquisition and reduction software in REVEGITS are programmed around the Turbo Pascal version 4.0 language environment by Borland International and the HALO '88 graphics library by Media Cybernetics. Figure 3.4 depicts the REVEGITS' data acquisition and reduction hardware. The plotter showed in this figure is HP 7475A.
The REVEGITS' 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. Figure 3.6 depicts the depth measurement system set up.

Further information about REVEGITS' data acquisition software is available in Chen (1990).

3.2.2.2. The MQSC Truck Data Acquisition and Reduction System

An automated data acquisition technique was developed for the MQSC truck system by Fugro-McClelland, Inc. One operator can perform all testing tasks, eliminating the personnel and time required in conventional data logging.

Cone resistance and the combined cone and local side friction resistance readings are taken at 2 cm intervals, or about 15 readings per foot. The electrical signal is continuously transmitted to an on board portable Grid 386 personal computer, running at 20 MHz. In this computer, the signal is simultaneously digitized, displayed on the screen in the form of a line graph, and stored in the sounding data file. Plotting and printing capabilities are available. Figure 3.6 shows a general view of the on board computer system.
3.2.3. Cone Penetrometers

Figure 3.7 shows the three different electronic friction cone penetrometers used in the field testing program. Calibration data for all cone penetrometers used in the field testing program are presented in Appendix 2. A brief description of all cones is given below:

(1) **The Reference Friction Cone Penetrometer:** is a 35.7 mm nominal diameter Fugro cone penetrometer (cross-sectional area of 10\(^2\) cm\(^2\)), with a friction sleeve area of 150 cm\(^2\) and a cone apex angle of 60°. 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 areas (area ratio = .45), built-in amplifiers and an incorporated slope sensor.

(2) **The MQSC Penetrometer:** is a Fugro-McClelland small size cone penetrometer which can be viewed as a scaled down version of the full size 10 cm\(^2\) reference Fugro penetrometer. It is a 1.27 cm nominal diameter subtraction type penetrometer (cross-sectional area of 1.27 cm\(^2\)), with a friction sleeve area of 25.14 cm\(^2\), cone apex angle of 60°, and unequal end area ratio of .75. It measures cone resistance and combined cone and local side friction resistance.

(3) **The 15 cm\(^2\) Cone Penetrometer:** is a 43.7 mm nominal diameter Fugro cone penetrometer (cross-sectional area of 15 cm\(^2\)), with a friction sleeve area of...
200 cm² and a cone apex angle of 60°. It measures cone and local side friction resistance. The Fugro cone is a subtraction type with unequal end area ratio of .59, built-in amplifiers and an incorporated slope sensor.

(4) The 15 cm² Dual-Piezocone Penetrometer: is similar to the 15 cm² Fugro friction penetrometer with the additional capability of measuring pore pressure behind the friction sleeve in addition to the pore pressure measurement at the cone tip.

3.3. Description of Test Sites

The sites sounded in order to evaluate scale effects 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. Only one of the sites, the Big River Industries, is not a part of the State highway system under the jurisdiction of the Louisiana Department of Transportation. The names given to the places indicate only their general location. Relevant information about the sites studied are given below.

(1) Big River Industries is a recent alluvium soil with 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 West of Baton Rouge, Louisiana. The ground water was observed at 1 m.

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

(3) Iowa (Natural Ground) is a terrace soil with predominance of sandy silts to clayey silts, clayey silts to silt clays, sands to silty sands, sands, and of silty clays and clays 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 SE of Baton Rouge, Louisiana.
3.4. Field testing Program and Test Results

3.4.1. Field Testing Program

Sounding generally 10 m deep were performed in three natural grade and two highway embankment soils. At each chosen location, five sounding apiece were executed with the reference and 15 cm$^2$ cone penetrometers, and three sounding were performed with the MQSC. Continuous penetration tests spaced 2 meters from each other and at a penetration rate of 2 cm/sec were a standard in the field test program, in accordance with the International Reference Test Procedure (ISSMFE, 1988).

A question of concern was the influence of the generated pore pressures during penetration on cone penetration readings for the three different cones (i.e., problem of unequal end areas) led to the use of piezocone in at least one of the sites studied. The Big River Industries site was chosen for piezocone penetration tests, and two sounding were executed with the LSU/Fugro 15 cm$^2$ dual-piezocone penetrometer.

The influence of wear of cone tip and friction sleeve on cone resistance was also addressed in this research. The Iowa (natural ground) site was chosen for this purpose, and three sounding were performed with a brand new tip and friction sleeve attached to the MQSC. Considering that the same cone penetrometer body
and load cells were previously used at this site with a 1000 m penetration length tip and friction sleeve, the sounding performed with the brand new cone allowed for comparison between the MQSC cone penetration resistance measurements before and after wear.

Figure 3.8 depicts the field testing program layout adopted in this investigation. It was intended to perform sounding 10 m deep at each site with the three different cross-section cone penetrometers. However, limitations in the MQSC thrust system capacity prevented that depth to be reached at Highland Road (natural ground).

3.4.2. Field Testing Program Results

The results of all the cone penetration tests performed in the field testing program at the five chosen sites are given in Appendix 1. A total of seventy sounding were executed in the field testing program. The test numbers, cone types and penetration depths of tests performed in each site are given below.

(1) Big River Industries Site (Natural Ground):

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Cone Type</th>
<th>Penetration Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>MQSC</td>
<td>10.6</td>
</tr>
<tr>
<td>C2</td>
<td>MQSC</td>
<td>10.6</td>
</tr>
<tr>
<td>C3</td>
<td>MQSC</td>
<td>10.6</td>
</tr>
<tr>
<td>Test Number</td>
<td>Cone Type</td>
<td>Penetration Depth (m)</td>
</tr>
<tr>
<td>------------</td>
<td>-------------------</td>
<td>----------------------</td>
</tr>
<tr>
<td>LSU 001</td>
<td>10 cm²</td>
<td>11.2</td>
</tr>
<tr>
<td>LSU 002</td>
<td>10 cm²</td>
<td>10.4</td>
</tr>
<tr>
<td>LSU 003</td>
<td>10 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 004</td>
<td>10 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 005</td>
<td>10 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 006</td>
<td>15 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 007</td>
<td>15 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 008</td>
<td>15 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 009</td>
<td>15 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 010</td>
<td>15 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 011</td>
<td>15 cm² (dual-piezocone)</td>
<td>9.0</td>
</tr>
<tr>
<td>LSU 012</td>
<td>15 cm² (dual-piezocone)</td>
<td>9.0</td>
</tr>
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</table>

(2) Highland Road Site (Natural Ground):

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Cone Type</th>
<th>Penetration Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>MQSC</td>
<td>9.5</td>
</tr>
<tr>
<td>B2</td>
<td>MQSC</td>
<td>9.1</td>
</tr>
<tr>
<td>B3</td>
<td>MQSC</td>
<td>9.1</td>
</tr>
<tr>
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<td>15 cm²</td>
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<td>LSU 003</td>
<td>15 cm²</td>
<td>11.0</td>
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<td>LSU 004</td>
<td>15 cm²</td>
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<td>LSU 005</td>
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<td>10 cm²</td>
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</tr>
<tr>
<td>LSU 008</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 009</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 010</td>
<td>10 cm²</td>
<td>11.0</td>
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</tbody>
</table>

(3) Iowa Site (Natural Ground):

<table>
<thead>
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<th>Test Number</th>
<th>Cone Type</th>
<th>Penetration Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>MQSC</td>
<td>10.2</td>
</tr>
<tr>
<td>C2</td>
<td>MQSC</td>
<td>10.2</td>
</tr>
<tr>
<td>C3</td>
<td>MQSC</td>
<td>10.3</td>
</tr>
<tr>
<td>LSU 001</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 002</td>
<td>10 cm²</td>
<td>10.3</td>
</tr>
<tr>
<td>LSU 003</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>Test Number</td>
<td>Cone Type</td>
<td>Penetration Depth (m)</td>
</tr>
<tr>
<td>-------------</td>
<td>-----------</td>
<td>-----------------------</td>
</tr>
<tr>
<td>LSU 004</td>
<td>10 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 005</td>
<td>10 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 006</td>
<td>15 cm²</td>
<td>10.1</td>
</tr>
<tr>
<td>LSU 007</td>
<td>15 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 008</td>
<td>15 cm²</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 009</td>
<td>15 cm²</td>
<td>10.4</td>
</tr>
<tr>
<td>LSU 010</td>
<td>15 cm²</td>
<td>10.0</td>
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</table>

(4) Highland Road Site (Embankment):

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Cone Type</th>
<th>Penetration Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>MQSC</td>
<td>10.0</td>
</tr>
<tr>
<td>A2</td>
<td>MQSC</td>
<td>10.0</td>
</tr>
<tr>
<td>A3</td>
<td>MQSC</td>
<td>10.0</td>
</tr>
<tr>
<td>LSU 001</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 002</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 003</td>
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<td>11.0</td>
</tr>
<tr>
<td>LSU 004</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 005</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 006</td>
<td>15 cm²</td>
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<tr>
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<tr>
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<td>11.0</td>
</tr>
<tr>
<td>LSU 010</td>
<td>15 cm²</td>
<td>11.0</td>
</tr>
</tbody>
</table>

(5) McElroy Swamp Site (Embankment):

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Cone Type</th>
<th>Penetration Depth (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>MQSC</td>
<td>10.2</td>
</tr>
<tr>
<td>D2</td>
<td>MQSC</td>
<td>10.0</td>
</tr>
<tr>
<td>D3</td>
<td>MQSC</td>
<td>10.2</td>
</tr>
<tr>
<td>LSU 001</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 002</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 003</td>
<td>10 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 004</td>
<td>10 cm²</td>
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</tr>
<tr>
<td>LSU 005</td>
<td>10 cm²</td>
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</tr>
<tr>
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<td>15 cm²</td>
<td>11.0</td>
</tr>
<tr>
<td>LSU 007</td>
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<tr>
<td>LSU 008</td>
<td>15 cm²</td>
<td>11.0</td>
</tr>
</tbody>
</table>

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Averaged sounding data plots for each site are shown in Figures 3.9 to 3.25. Figures 3.9 and 3.10 depict plots of average cone resistance and friction ratio for all the sites investigated in the field testing program, using the reference cone penetrometer. Figures 3.11 to 3.25 show plots of average cone resistance, local side friction resistance and friction ratio of each site, considering the MQSC, the reference and the 15 cm$^2$ cone data plotted in the same profile.

Figures 3.26 to 3.30 display plots of the MQSC, reference and 15 cm$^2$ cone penetrometers data when considering all sites plotted sequentially as an adjoining continuous sounding. In this fictitious profile, the averaged CPT data from each site investigated are displayed sequentially, creating a fictitious profile with an imaginary sounding depth and a substantial size data file to be used in computational analysis. Figures 3.26 to 3.28 present plots of cone resistance, local side friction resistances and friction ratio; and, Figures 3.29 and 3.30 introduce ratio plots of cone and local side friction resistance between the MQSC and reference cone, and between the 15 cm$^2$ and reference cone, respectively.

The influence of unequal end area ratio on cone resistance is analyzed in Figures 3.31 to 3.34. Figure 3.31 depicts excess pore pressure generated at the tip of the 15 cm2 dual-piezocone from two sounding performed at the Big River site. The sounding were performed with a pre-punch of 1 meter. Figures 3.32 to 3.34
show cone resistance ($q_c$) and cone resistance corrected for excess pore pressure ($q_t$), for the MQSC, the reference and the 15 cm² cone penetrometers. It was assumed that all cones would generate similar excess pore pressure distribution during penetration.

The influence of the wear of the cone tip and friction sleeve on cone penetration resistance is analyzed in Figures 3.35 to 3.44. These figures introduce data obtained at Iowa (natural ground) from three sounding performed with the MQSC with a "brand new" cone tip and friction sleeve, and three more sounding performed using a worn cone tip and friction sleeve that had 1000 meters of penetration history. The sounding performed with the "brand new" and "worn" tips and friction sleeves were performed on the same alignment, 1 meter away from each other. Figures 3.35 and 3.36 present cross-sections of the MQSC with a "brand new" cone tip and friction sleeve, and the worn cone tip and friction sleeve, respectively. Figures 3.37 to 3.40 display cone and local side friction resistances for the MQSC with the "brand new" and "worn" tip and friction sleeve. Figures 3.41 to 3.43 introduce plots of averaged cone resistance, local side friction resistance and friction ratio when considering the MQSC with the "brand new" tip and friction sleeve, and with the "worn" tip and friction sleeve. The averaged values of cone penetration resistance were obtained for a sand layer between the depths of 2.2 and 3.8 m. Finally, Figure 3.44 shows a plot of cone penetration resistance ratios between the "brand new" and the "worn" tips and friction sleeves.
3.5. Statistical Evaluation of the Field Testing Program Data

The statistical analysis was directed to the comparative study of the pooled profiles. It considers the pooled data from all sites, accounting for c. 2400 observations with each cone penetrometer type. A simple linear regression model (Neter et. al., 1985) was used to determine the relationships between the reference, MQSC and 15 cm² cone penetrometer data. The MQSC and 15 cm² cone penetrometer measurements were regarded as the response variables, and the reference penetrometer measurements as the independent variable. The model was stated as follows:

\[ Y_i = \beta_0 + \beta_1 X_i + \epsilon_i \]  

where:

- \( Y_i \): Value of the response variable in the \( i^{th} \) trial;
- \( \beta_0, \beta_1 \): Regression parameters;
- \( X_i \): Value of the independent variable in the \( i^{th} \) trial;
- \( \epsilon_i \): error terms which are independent \( N(0, \sigma^2) \);

Results of the linear regression analysis applied to the MQSC and 15 cm² penetrometer data versus the reference penetrometer data are summarized below. The units of \( q_c, f_s \) and \( R_t \) are given in kg/cm², kg/cm² and percentage, respectively. 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) \( f_{1}, R_{1} \) - soils with \( q_{c} \) smaller or equal to 80 kg/cm², and (2) \( f_{2}, R_{2} \) - soils with \( q_{c} \) higher than 80 kg/cm².

MQSC versus Reference Penetrometer:

\[
\begin{align*}
q_{c10 \text{ cm}^2} &= -0.359 + 0.861 \times q_{c15 \text{ cm}^2} \\
q_{f10 \text{ cm}^2} &= 0.234 + 0.836 \times q_{f11 \text{ MQSC}} \\
q_{f20 \text{ cm}^2} &= 0.497 + 1.115 \times q_{f21 \text{ MQSC}} \\
R_{f10 \text{ cm}^2} &= 3.196 + 0.511 \times R_{f11 \text{ MQSC}} \\
R_{f20 \text{ cm}^2} &= 1.270 + 1.330 \times R_{f21 \text{ MQSC}}
\end{align*}
\]

15 cm² versus Reference Penetrometer:

\[
\begin{align*}
q_{c10 \text{ cm}^2} &= -0.729 + 1.055 \times q_{c15 \text{ cm}^2} \\
f_{a10 \text{ cm}^2} &= 0.0197 + 1.150 \times f_{a11 \text{ cm}^2} \\
R_{f10 \text{ cm}^2} &= 0.812 + 0.931 \times R_{f11 \text{ cm}^2}
\end{align*}
\]

Table 3.1 shows descriptive statistics for each cone penetrometer from averaged sounding data. Table 3.2 gives descriptive statistics of differences between cone resistance, local side friction resistance and friction ratio of the MQSC and 15 cm² cone penetrometers, and the reference penetrometer.

Figures 3.44 to 3.48 depict scatter plots of cone and local side friction resistances of the MQSC and 15 cm² cone penetrometers versus the reference probe data.
Figures 3.49 to 3.53 show cone resistance, local side friction resistance and friction ratio plots of the MQSC and 15 cm$^2$ penetrometers corrected via linear regression equations and correction factors, and of the reference cone penetrometer. These results will be used in the next section to make inferences on scale effects between the reference and the MQSC and 15 cm$^2$ cone penetrometer.

The influence of soil variability on cone penetration resistance is briefly addressed in this dissertation via the coefficient of simple determination ($r^2$) between pairs of sounding. Table 3.3 depicts $r^2$ of five sounding performed with the reference cone penetrometer at Highland Road (natural ground). An arithmetic averaged sounding called "average" (X) is also added to the table. The upper diagonal part of the table shows $r^2$ for cone resistance, and the lower diagonal part depicts $r^2$ for local side friction resistance.

3.6. Comments on the Field Testing Program Data

The statistical analysis of the data on field testing was directed to the estimation of the scale effect between the MQSC and 15 cm$^2$ penetrometers and the reference probe.

Figures 3.9 and 3.10 reveal that the field testing program covered ranges of
soils commonly encountered by CPT, and referred in the literature.

Table 1 shows mean (X), standard deviation (S) and coefficient of variation (C) values of cone penetration resistance obtained from sounding performed with the MQSC, the reference and the 15 cm² penetrometers at five different sites in the State of Louisiana. This table suggests that cone resistance’s mean and standard deviation decrease with increase in cone dimensions. However, it is not possible to infer any conclusion regarding local side friction resistance. Although the relatively high values of standard deviation could minimize the strength of conclusions about scale effects between the MQSC and 15 cm² penetrometers and the reference, the decrease in cone resistance with increase in cone diameter could be viewed as a pattern that would reflect the existence of scale effects among the cones studied. If there is a pattern, it seems that it can not be explained solely by soil variability, taking into account that the adopted field testing program layout minimized the soil variability influence on cone penetration resistance. Also, the decrease in standard deviation with increase in cone diameter could reinforce the thesis of higher capability of a miniature cone to capture more of the soil variability than a large dimension cone penetrometer.

Inferences about difference between the MQSC and the reference penetrometers, and the 15 cm² and the reference penetrometers’ population means, matched large samples, were also addressed in the statistical analysis, taking into account the pooled field data. Considering the α risk to be controlled at .001 level
when the difference between the population means is zero (i.e., \( \mu_1 - \mu_2 = 0 \)), and assuming that the population differences are approximately normally distributed, the standardized test statistics turned out as follows (Neter et al., 1988):

. Test Alternatives:
   . \( H_0: \mu_1 - \mu_2 = 0 \)
   . \( H_1: \mu_1 - \mu_2 \neq 0 \)

in which \( \mu_1 = 10 \text{ cm}^2 \) cone penetrometer population mean, \( \mu_2 = \text{MQSC} \) and \( 15 \text{ cm}^2 \) cone penetrometer population means.

. Test Statistic:
   . \( z' = \frac{D}{S_D} \)
   . \( S_D^2 = \frac{S_0^2}{n} \)

in which \( z' = \) test statistic, \( D = \) samples difference mean, \( S_0^2 = \) samples difference variance, \( n = \) number of observations in each sample.

. MQSC versus Reference Penetrometer:
   . Cone Resistance: \( z' = -17.51 \)
   . Local Side Friction Resistance: \( z* = 23.28 \)
   . Friction Ratio: \( z* = 39.20 \)

. \( 15 \text{ cm}^2 \) versus Reference Penetrometer:
   . Cone Resistance: \( z' = 6.02 \)
. Local Side Friction Resistance: $z' = 37.7$

. Friction Ratio: $z' = 24.29$

$. z(1 - \alpha/2) = z(.9995) = 3.29$  (From Cumulative and percentiles of the Standard Normal Distribution; Neter et al., 1988)

Since all $z'$ in absolute value are higher than $z(.9995)$, it can be concluded that alternative $H_1$ governs (i.e., that there are significant differences in the population means).

Visual analysis of the Figures 3.11 to 3.30 also suggest presence of scale effects between the MQSC and 15 cm$^2$ and the reference cone penetrometer data. Figures 3.49 to 3.53 depict the MQSC and 15 cm$^2$ penetrometers' cone resistance, local side friction resistance and friction ratio corrected via linear regression equations. From an engineering point of view, as depicted in Figure 3.50, 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 (i.e., $q_{c(10\ text{cm}^2)} = .85 \times q_c^{MQSC}$). A division factor of .85, as showed in Figure 3.52, also can be used to correct the 15 cm$^2$ penetrometer local side friction resistance in order to obtain the reference local side friction resistance (i.e., $f_{l(15\ text{cm}^2)} = (1/.85) \times f_{l(15\ text{cm}^2)}$). The MQSC's local side friction resistance and friction ratio should be corrected via linear regression equations considering two ranges of cone resistance: (1) soils with $q_e$ equal or smaller than 80 kg/cm$^2$, and (2) soils with $q_e$ higher than 80 kg/cm$^2$. No
significant correction is necessary for cross-correlating cone resistance of the reference and 15 cm² cross-section penetrometers. An optional linear regression equation is presented, however, for academic purposes.

The influence of unequal cone end area ratios on cone penetration resistance is addressed in Figures 3.31 to 3.34. The analysis is based on the assumption that cone diameter does not affect the excess pore pressure distribution during cone penetration (Juran and Tumay, 1989). These figures suggest that the different cone end area ratio do not significantly affect the measured cone resistance. From these figures, it is evident that all cones show similar $q_t$ and $q_c$ trends with penetration depth. Also, considering that the unequal end area ratios of the MQSC, the reference and the 15 cm² cones are .75, .45 and .59, respectively, it was expected that the MQSC could give higher cone resistance than the 10 and 15 cm² cones. Also, it was anticipated that the reference penetrometer could give smaller cone resistance than the 15 cm² penetrometer. Although the first part of the expectation came to be true, the second one was not verified, giving the reference higher cone resistance than the 15 cm² penetrometer. However, these conclusions can be biased by the relative shallow sounding depths (9 m) and type of soil (soft clay), which generated relatively low excess pore pressure during cone penetration.

Wear of cone tip and friction sleeve is analyzed in Figures 3.35 to 3.44. Figures 3.41 to 3.43 clearly illustrates that the cone with the "brand new" tip and friction sleeve gave lower friction ratio, cone and local side friction resistances than
the cone with the "worn" tip and friction sleeve. This fact contradicts finding associated to cone penetrometers following the Dutch Standard NEN 3680, and reported by Jekel (1988). This author related that "...for sand the friction ratio (the ratio of the recorded local friction and cone resistance) decreased for a particular cone with increasing penetration length. After a penetration length of 500 m the friction ratio was only 55 % of its original value." Apart of soil variability influence, an explanation for the decrease in friction ratio with increase in penetration length observed in this investigation, in contrast with Jekel's reported results, could be based on a different wear pattern of the friction sleeve of the MQSC and the reference penetrometer. Jekel refers that "...the wear pattern of the sleeve and tip appears to be conical. The top edge of the sleeve wears more than the bottom edge." A view of a cross-section of the MQSC with the "brand new" and "worn" cone tips and friction sleeves is showed in Figures 3.35 and 3.36. From these figures, it is apparent that the conical wear pattern verified for the standard cone is not verified for the MQSC. It seems that the MQSC sleeve wear pattern has changed to a cylindrical form. Also, an accentuated wear of the MQSC's tip on the tip section close to the friction sleeve end is verified.

The influence of soil variability in CPT is briefly analyzed in Table 3, using the $r^2$ between pairs of sounding performed at Highland Road (natural ground) with the reference cone penetrometer. Assuming that the influence of soil variability on CPT can be inferred in a primary basis from the coefficient of determination, and considering that are significant increases in $r^2$ when comparing the paired...
individual sounding with the individual sounding versus the arithmetic averaged sounding (X), it is possible to conclude that the use of averaged sounding data substantially improves the capability of the CPT in capturing site soil variability.
3.7. Tables and Figures
### Table 3.1. Descriptive Statistics: Field Testing Program Data (all sites, pooled data, n = 2351 observations)

<table>
<thead>
<tr>
<th>Cone Area (cm²)</th>
<th>Cone Resistance (kg/cm²)</th>
<th>Friction Resistance (kg/cm²)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>X</td>
<td>S</td>
</tr>
<tr>
<td>1.27</td>
<td>37.59</td>
<td>36.98</td>
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<tr>
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<td>31.64</td>
<td>35.39</td>
</tr>
<tr>
<td>15</td>
<td>30.77</td>
<td>32.86</td>
</tr>
</tbody>
</table>

- X - sample mean
- S - sample standard deviation
- C - sample coefficient of variation

### Table 3.2. Descriptive Statistics on Differences: Field Testing Program Data (all sites, pooled data, n = 2351 observations)

<table>
<thead>
<tr>
<th>Differences</th>
<th>q_d (kg/cm²)</th>
<th>f_r (kg/cm²)</th>
<th>R_r (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D</td>
<td>S_d²</td>
<td>D</td>
</tr>
<tr>
<td>10CM²-1.27CM²</td>
<td>-.4894</td>
<td>1.8358</td>
<td>.1897</td>
</tr>
<tr>
<td>10CM²-15CM²</td>
<td>.0759</td>
<td>.3737</td>
<td>.1463</td>
</tr>
</tbody>
</table>

- D - samples difference mean
- S_d² - samples difference variance
Table 3.3. Coefficient of Determination ($r^2$) Matrix: Highland Road (natural ground), Reference Cone Penetrometer. Upper diagonal part of table shows $r^2$ for cone resistance, and lower diagonal part shows $r^2$ for local side friction resistance.

<table>
<thead>
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<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>X</th>
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<tbody>
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<td>.32</td>
<td>.33</td>
<td>.34</td>
<td>.62</td>
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<td>.38</td>
<td>1</td>
<td>.76</td>
<td>.62</td>
<td>.77</td>
</tr>
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<td>1</td>
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<td>1</td>
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<td>.73</td>
<td>.75</td>
<td>.76</td>
<td>.76</td>
<td>1</td>
</tr>
</tbody>
</table>

X - arithmetic average of 5 sounding performed at Highland Road (natural ground)
Figure 3.1. General View of the LSU REVEGITS
Figure 3.2. Details of the REVEGITS Penetration Thrust System
Figure 3.3. General View of the MQSC Penetration System
Figure 3.4. REVEGITS Data Acquisition and Reduction System
Figure 3.5. REVEGITS Depth Measurement System
Figure 3.6. MQSC Data Acquisition and Reduction System
Figure 3.7. Cone Penetrometers Used in the Field Testing Program
FIELD TESTING PROGRAM

3 FRICTION CONE PENETROMETERS (1 PIEZOCONES):

- 1.27 CM² FUGRO-McCLELLAND FRICTION CONE
- 10 CM² FUGRO FRICTION CONE
- 15 CM² FUGRO FRICTION CONE (15 CM² FUGRO PIEZOCONES)

SITES IN LOUISIANA - U.S.A.

- 3 NATURAL GRADE SOILS
- 2 EMBANKMENTS

FIELD LAYOUT:

- SOUNDINGS 2 M FROM EACH OTHER
  - 15 CM² (5 SOUNDINGS; PIEZOCONE: 2 SOUND.)
  - 10 CM² (5 SOUNDINGS)
  - 1.27 CM² (3 SOUNDINGS)

SOUNDING INFORMATION:

- NUMBER OF SOUNDINGS PERFORMED: 70 SOUNDINGS
- AVERAGE DEPTH: 10 METERS

Figure 3.8. Field Testing Program Layout
Figure 3.9. "Reference" Cone Resistance: Average of 5 sounding at each of the five sites in Louisiana
Figure 3.10. "Reference" Friction Ratio: Average of 5 sounding at each of the five sites in Louisiana

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Figure 3.11. Averaged Cone Resistance: Big River (natural ground)
Figure 3.12. Averaged Local Side Friction Resistance: Big River (natural ground)
Figure 3.13. Averaged Friction Ratio: Big River (natural ground)
Figure 3.14. Averaged Cone Resistance: Highland Road (natural ground)
Figure 3.15. Averaged Local Side Friction Resistance: Highland Road (natural ground)
Figure 3.16. Averaged Friction Ratio: Highland Road (natural ground)
Figure 3.17. Averaged Cone Resistance: Iowa (natural ground)
Figure 3.18. Averaged Local Side Friction Resistance: Iowa (natural ground)

15, 10 AND 1.27 CM² CONES DATA

LOCAL SIDE FRICTION RESISTANCE (KG/CM²)

DEPTH (METER)

1.27 CM²
10 CM²
15 CM²
15, 10 AND 1.27 CM2 CONES DATA

IOWA (NATURAL GROUND)

Figure 3.19: Averaged Friction Ratio: Iowa (natural ground)
Figure 3.20. Averaged Cone Resistance: Highland Road (embankment)
Figure 3.21. Averaged Local Side Friction Resistance: Highland Road (embankment)
15, 10 AND 1.27 CM2 CONES DATA

HIGHLAND ROAD (EMBANKMENT)

DEPT (METER)
Figure 3.23. Averaged Cone Resistance: McElroy Swamp (embankment)
Figure 3.24. Averaged Local Side Friction Resistance: McElroy Swamp (embankment)
3.25. Averaged Friction Ratio: McElroy Swamp (embankment)
Figure 3.26. Averaged Cone Resistance (pooled profiles)
Figure 3.27. Averaged Local Side Friction Resistance (pooled profiles)
Figure 3.28. Averaged Friction Ratio (pooled profiles)
Figure 3.29. Cone Resistance Ratios (pooled profiles)
Figure 3.30. Local Side Friction Resistance Ratios (pooled profiles)
Figure 3.31. Excess Pore Pressure Measured at Tip of the 15 cm² Dual-Piezocone: Big River (natural ground)
Figure 3.32. Averaged Cone Resistance ($q_c$): Big River (natural ground)
Figure 3.33. Total Cone Resistance ($q_t$): Big River (natural ground)
Figure 3.34. Averaged Cone Resistance ($q_a$) and Total Cone Resistance ($q_t$): Big River (natural ground)
Figure 3.35. Transverse Section of the MQSC with Brand New Cone Tip and Friction Sleeve
Figure 3.36. Transverse Section of the MQSC with a 1000 m Penetration Length Cone Tip and Friction Sleeve

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Figure 3.37. Cone Resistance from Three Soundings Performed at Iowa (natural ground): Brand new cone tip and friction sleeve
Figure 3.38. Local Side Friction Resistance from Three Sounding Performed at Iowa (natural ground): Brand new cone tip and friction sleeve
Figure 3.39. Cone Resistance from Three Sounding Performed at Iowa (natural ground): 1000 m penetration length cone tip and friction sleeve.
Figure 3.40. Local Side Friction Resistance from Three Sounding Performed at Iowa (natural ground): 1000 m penetration length cone tip and friction sleeve.
Figure 3.41. Averaged Cone Resistance at Iowa (natural ground): Brand New and 1000 m penetration length cone tips and friction sleeves
Figure 3.42. Averaged Local Side Friction Resistance at Iowa (natural ground): Brand new and 1000 m penetration length cone tips and friction sleeves

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Figure 3.43. Averaged Friction Ratio at Iowa (natural ground): Brand new and 1000 m penetration length cone tips and friction sleeves
Figure 3.44. Ratios Between Cone Resistance, Local Side Friction Resistance and Friction Ratio at Iowa (natural ground)
Figure 3.45. Scatter Plot of Cone Resistance (pooled profiles): MQSC versus reference cone penetrometer
Figure 3.46. Scatter Plot of Local Side Friction Resistance (pooled profiles): MQSC versus reference cone penetrometer

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Figure 3.47. Scatter Plot of Cone Resistance (pooled profiles): 15 cm$^2$ cone penetrometer versus reference cone penetrometer
Figure 3.48. Scatter Plot of Local Side Friction Resistance (pooled profiles): 15 cm² Cone penetrometer versus reference cone penetrometer.

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Figure 3.49. Cone Resistance (pooled profiles): MQSC and 15 cm² penetrometer data corrected via simple linear regression equations
Figure 3.50. Cone Resistance (pooled profiles): MQSC data corrected via simple linear regression equation and correction factor.
Figure 3.51. Local Side Friction Resistance (pooled profiles): MQSC and 15 cm² data corrected via simple linear regression equations

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Figure 3.52. Local Side Friction Resistance (pooled profiles): 15 cm² penetrometer data corrected via simple linear regression equation and correction factor.
Figure 3.53. Friction Ratio (pooled profiles): MQSC and 15 cm² penetrometer data corrected via simple linear regression equations.
CHAPTER 4

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 other strength and compressibility tests (unconfined, triaxial and oedometer). 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 ($\gamma_{dmax}$) 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 in order to establish the basis for calibrating a miniature cone penetrometer, the MQSC, to be employed in construction control of road embankments in the State of Louisiana. Calibration is to be understood in the context of generating correlations between cone penetration resistance and soil parameters of interest such as CBR, $w_{opt}$ and $\gamma_{dmax}$ for the construction control of embankments. The preliminary laboratory test program was intended to encompass the following steps:

(1) To prepare a laboratory reliable apparatus, i.e. a flexible wall calibration chamber, suitable for allowing the calibration of MQSC in different types of soils. The types of soils selected for the laboratory testing program would comprise the range of soils in the field that are under the scope of applicability of the quasi-static cone penetration test (i.e., sand, silt and clay), as generally reported in geotechnical engineering;

(2) To perform traditional laboratory tests on the soils selected for this study. These tests were to include the determination strength and deformation characteristics of the soils under study;
(3) To perform a laboratory testing program using the apparatus introduced in item (1) to measure cone resistance (MQSC) over a range of maximum dry density and vertical stress levels;

(4) To develop empirical correlations between cone penetration resistance measurements, e.g. cone tip resistance \( (q_c) \) and friction ratio \( (R_f) \), and soil parameters of interest to road engineering design and construction control, such as: maximum dry density \( (\gamma_{\text{d,max}}) \), California Bearing Ratio (CBR) and internal friction angle \( (\phi) \);

(5) To indicate possible extensions of the work undertaken to future analysis, such as: influence of the changes in the soil type, compactive effort and lift thickness.

In an attempt to include a wide experimental range, five (5) types of soils were envisioned for the laboratory testing program. Three of these soils were to be well documented materials that would portray a calibration basis. The first soil candidate for this study was to be Monterrey sand No. 0, a sand of nationally known geotechnical engineering properties; the second soil was to be silt from a construction site near Terre Haute in West Indiana; and, the third candidate was to be a mixture of 50% of Georgia kaolinite (a soil acquired from Georgia Kaolin Company of Elisabeth, New Jersey) and 50% of Terre Haute Silt. The second and third soils have been extensively used in researches at Purdue University, and are
well documented by Huang (1986). The two other soils selected for this laboratory test program were to be representative of those commonly encountered and/or used in road engineering for the construction of embankments in the State of Louisiana. These two soils were to have grain size distributions between sand-silt and silt-clay, respectively, and would be furnished by the Louisiana Transportation Research Center (LTRC).

Initially a cylindrical calibration chamber approximately 14" high and 10" in diameter, and housing a sample around 50 lb, was expected to be prepared and used for testing compacted soil samples in the present study. Also, all penetration tests were supposed to be carried out under boundary condition 1, i.e. constant vertical stress and constant lateral stress. It was expected that this program of study would furnish guidelines for further studies encompassing the range of soils in the field which are commonly used in road engineering as material for embankment construction.

As the project developed, the possibility of building a larger calibration chamber, and therefore the chance of testing soil samples without the extreme constraints of the boundary effects generated by the small size chamber previously suggested, became a reality. A larger chamber housing a sample around 800 lb was then built for testing purpose. This fact necessitated a substantial change in the scope of the laboratory testing program. An elaborate crane system for moving soil samples and calibration chamber parts had to be implemented and a large dimension
automatic tamper and compaction system hat to be designed and fabricated for preparation of compacted soil samples. A brand new conception of panel of controls-computer software were also introduced in order to fully control the calibration chamber test via a personal computer in such a way that all the significant test data generated during sample consolidation and penetration phases could be displayed in the computer screen at real time, saved in a file after each reading, and plotted immediately after conclusion of the test in a personalized format.

The unexpected changes in the initial program of study introduced complex problems to be solved in the course of this research. Considering the original proposed research, it soon became apparent that the time frame required for laboratory equipment design and fabrication, computer software development and soil sample testing was quite immense. Consequently, the study aims were narrowed. It was then concluded that the laboratory program of study could have to be restricted only to the development of a laboratory calibration chamber system that would ascertain proper testing of compacted soil samples under four boundary conditions. The goals finally adopted were as follows:

(1) Development of a reliable apparatus, the Louisiana State University Flexible Wall Calibration Chamber System (LSU/CALCHAS), for calibration of the MQSC in types of soils comprising the field range of materials that are under the scope of applicability of the quasi-static cone penetration test, i.e. sand, silt and clay.
This task would involve the construction of a flexible wall calibration chamber, and a fully automated calibration chamber panel of controls ran by computer software capable of simulating typical calibration chamber boundary conditions (i.e., BC1, BC2, BC3 and BC4) commonly referred in the literature;

(2) To perform a representative miniature cone penetration test in a compacted soil sample prepared from a selected soil using the equipment introduced in item (1). Cone penetration resistance should be measured under boundary conditions 1 and 3 in a sample prepared at the AASHTO "standard" compaction effort and consolidated under $K_s$ condition at a specified vertical stress level;

(3) To demonstrate feasibility of extensions of the work undertaken for future research (i.e., influence of the changes in the soil type, compactive effort and lift thicknesses).

In order to cover all the items involved in the laboratory test program, the present chapter is divided into the following sections: equipment, data acquisition and reduction software, test procedure and calibration chamber test results.
4.2. Equipment

The Louisiana State University Calibration Chamber System involved in the laboratory phase of the research program, and hereafter referred to the LSU/CALCHAS is presented in the following sections: calibration chamber, panel of controls, cone penetrometer, hydraulics and chucking system, compactor and compaction mold, penetration depth measurement system, auxiliary system, calibration chamber operation. Calibration data for the equipment related to this phase is presented in Appendix 3 of this dissertation.

4.2.1. Calibration Chamber

The LSU/CALCHAS was primary built for testing cone penetrometers under controlled boundary conditions. It permits the simulation of the $K_0$ consolidation phase and the four (4) traditional cone penetration boundary conditions commonly referred in the literature. The chamber is 1.78 m (70 in) high and .64 m (25 in) in overall diameter. It is divided in two sections: the sample cell, and the piston cell. Overall, five penetration tests can be performed in the same soil sample in the LSU/CALCHAS via five different penetration positions in the chamber top plate. The stresses in the vertical and horizontal directions can be controlled.
independently. A cross-section of the calibration chamber is shown in Figure 4.1, and pictures of the unit are depicted in Figure 4.2.

The sample cell is a double flexible wall device that can house a soil sample .53 m (20.66 in) in diameter and .79 m (31.06 in) in height. It walls are made out of 6.35 mm (.25 in) stainless steel 304 plates rolled into cylindrical shells that are .91 m (36 in) high. These shells are designed to withstand a maximum pressure of around 14 kg/cm² (200 psi). The internal diameter of the outer and inner walls are .58 m (23 in) and .56 m (22 in), respectively. During testing, the soil sample is isolated from the cell water by a 1.59 mm (.06 in) thick rubber membrane fixed to the sample top and bottom plates via four (4) O-rings. The sample top and bottom plates are made of 6061 T-6 aluminum, with dimensions .53 m (20.66 in) in diameter and 38.1 mm (1.5 in) and 63.5 mm (2.5 in) in height, respectively. The sample top plate is bolted to the chamber top plate which is a .64 m (25 in) in diameter and 38.1 mm (1.5 in) in height 6061 T-6 aluminum plate. The sample bottom plate rests on a .635 m (25 in) piston cell ring built in 6061 T-6 aluminum. The chamber top plate, sample cell inner and outer walls, and the piston cell ring are kept together via twelve (12) stainless steel 304 rods 12.7 mm (.5 in) in diameter tightened with a torque of 70 newton-meter (around 60 foot-pounds). These twelve rods also constitute the chamber's reaction frame during the penetration phase. The annular space between the soil sample and the sample inner wall and between the sample inner and outer walls will be referred as "inner sample" and "outer sample" cells, respectively. For testing purpose, the inner and outer sample cells are filled with
deaired water via two water lines connected to the chamber top plate. Under testing conditions, water in the inner and outer cells are pressurized generating changes in horizontal stresses over hydrostatic.

The piston cell is also a double wall cylinder similar in diameter and material to the sample cell. It is .43 m (17 in) long and the inside cell space is kept free for various types of testing instrumentation. A 76.2 mm (3 in) .406 m (16 in) hollow cylinder is attached to the bottom of the inner cell. The piston inner cell rests on the chamber bottom plate which is .64 m (25 in) in diameter and 38.1 mm (1.5 in) in height. The piston cell bottom plate carries a bearing shaft that houses the piston and allows for its vertical movement during testing. The annular space between the inner and outer cell walls and some grooves at the bottom of the piston inner cell are filled with deaired water furnished via a water line through the walls of the piston cell. Under testing conditions, when this water is pressurized, the inner piston moves upward and generates the required vertical stress. The piston cell ring, the piston cell and the chamber bottom plate are kept together using 12 stainless steel 304 rods 12.7 mm (.5 in) in diameter tightened with a torque of 47 newton-meters (around 40 foot-pounds).

4.2.2. Panel of Controls

The controls that regulate the operation of the chamber are grouped together
within reach of the operator on a vertical rectangular wood panel 1.22 m x 1.96 m (48 in x 77 in). In order to minimize volume changes in the system, the pressure lines are made of 3.18 mm (.125 in) walled copper tubing. The air and water pressure lines are 6.35 mm (.25 in) and 9.53 mm (.375 in) in overall diameter, respectively. The water pressure lines are also used for filling the piston and sample cells with water, and that is the reason why larger diameter tubing were utilized. Three quick-connectors link the three water pressure lines from the panel of controls to the piston and sample cells. A schematic drawing and pictures of the panel are shown in Figures 4.3 and 4.4. The basic components in the panel of controls are:

. **Fairchild Model 10 BP Precision Back Pressure Regulator:** The actual function of this pressure regulator is to provide protection against over pressure in the downstream portion of the pneumatic system. In the LSU/CALCHAS this pressure regulator acts as a relief valve for the inner sample and outer piston cells in order to keep their water pressure constant during testing (as would be required by the boundary condition being applied to the sample during the penetration phase). The pressure regulator, capable of handling flows up to 40 SCFM with a sensitivity of .125" W.C., vents to atmosphere when the downstream pressure exceeds a set point. The chamber panel of controls houses two units in the range of 2 to 150 psi.

. **Fairchild Model T 5700 Electro-Pneumatic Transducer:** This transducer converts electrical signals to linear pneumatic signals. It is immune to problems of supply pressure changes in the range of 18 to 150 psi, and works in the input range
of 0 to 10 volts DC. The chamber control panel houses four transducers covering the output pressure ranges of 5.6 to 30 psi and 3 to 120 psi, in a lower/upper range option. Two of these transducers are used in the piston cell operation during the Ko consolidation phase, and the two others provide pressure compensation between the inner and outer sample cells during Ko consolidation and penetration phase, if required.

. SenSym ST 2000 Pressure Transducer: It is a fully temperature compensated and signal conditioned transducer that provides a high level voltage output. Accuracy is within -.5 to .5 percent with an output voltage range of 1 to 6 V. The transducer's rugged stainless steel package provides excellent resistance to shock and vibration. The chamber control panel houses five transducers in the ranges 0 to 30 psi and 0 to 100 psi, in a lower/upper range option. Two transducers are connected to the water line related to the piston cell operation, two others are in the water line directed to the inner sample cell, and one in the range 0 to 30 psi is connected to the outer sample cell water line.

. Marsh Process Gauge: The gauge is 114.3 mm (4.5 in) dial size with phenolic construction in a safe case design. Five process gauges in the range of 0 to 100 psi and accuracy in the range of -.5 to .5 percent are connected to the chamber panel of controls. Three gauges are used to provide a visual check on pressure in the lines, sample cell and piston cell during testing. The gauges measure air pressure going into the air-water cylinder and water pressure in the annular space
between the sample and the sample inner wall. Two others Marsh pressure gauges are connected to the back pressure regulators in order to allow for their calibration, if necessary.

Air-Water Systems: The air-water systems used in the control panel are two PVC Schedule 240 cylinders, with high pressure caps glued to the top and bottom. These cylinders are connected to the piston and the sample cells, respectively. The cylinder connected to the sample cell is 12.7 cm (5 in) overall diameter and 30.48 cm (12 in) high, and the one associated to the piston cell is 30.48 cm (12 in) overall diameter and 78.74 cm high (31 in). The cylinders are filled with water and air in a 90% to 10% proportion, respectively, with an oil interface to minimize air/water absorption.

4.2.3. Cone Penetrometer

The MQSC is the same miniature cone penetrometer used in the field testing program. It is a 1.27 cm² cross-sectional area subtraction type Fugro-McClelland cone penetrometer, with a friction sleeve 6.3 cm long and an apex angle of 60°. It measures cone resistance and the combined cone and local side friction resistances. The MQSC push rod is a 9.53 mm (.375 in) overall diameter 204 stainless steel rod 1.82 m (6 ft) long. It is connected to the cone tip, down hole, and to a ten pin electric plug on the up hole end. The calibration procedure for the MQSC uses a
dumb tip and a doughnut for calibrating tip and sleeve, respectively. Pictures of the MQSC and its calibration set up developed for this research are shown in Figures 4.5 and 4.6.

4.2.4. Hydraulics and Chucking System

The hydraulic system consists of a drive jack that acts as a push jack for the cone penetrometer. It allows the cone and push rod to pass through it, has a stroke of around 15 cm (.5 ft), and a push rate controlled via a flow valve. In the LSU/CALCHAS, the push rate is set for 2 cm/sec. The push jack is connected to a special sleeve which contains the chucking system. The chucking system applies a grabbing force to the push rod and advances or extracts the cone from the soil sample, depending on the selected direction of movement. Figure 4.7 depicts the push jack, grabbing system and hydraulics, individually. A new hydraulic and push jack system that allows for penetrating the chamber soil sample in one single stroke, i.e. push jack with .79 m (31 in) stroke, as well as for testing different cone penetrometer diameters in the LSU/CALCHAS was also designed and fabricated for this research and is shown in Figure 4.8.
4.2.5. 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 .53 m (20.66 in) in diameter and .79 m (31.06 in) in height which will be tested in the LSU/CALCHAS.

The automatic tamper is composed of a 2 HP/1710 rpm Dayton adjustable speed motor drive resting on a 63.5 cm x 63.5 cm x 1.27 cm (25 in. x 25 in. x 1/2 in.) steel plate, a fly-wheel 63.5 cm (25in.) in diameter, a 2.44 m (96in.) high steel frame, steel cable and pulley system, and 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, rises 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 every time. 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 205 cm² (31.8 sq. in.) is attached to a tubular steel shaft 3.81 cm (1 1/2in.) in diameter and 2.34 m (92 in.) long. The tamping head and shaft form a 70.64 kg (155.6 lb) rammer. A split compaction mold and extension collar 81.9 cm x 52.86 cm x .64 cm (32 1/4 in. x 20 13/16 in. x 1/4 in.) and a 20.32 cm x 81.9 cm x .64 cm (8 in. x 32 1/4 in. x 1/4 in.), respectively, in steel are used in conjunction with a 52.39 cm x 7.62 cm (20 5/8 in. x 3in.) spacer disk in T-6
aluminum. The two sides of the compaction mold and extension collar are fastened with nine .64 cm (1/4 in) bolts. The split compaction mold is attached to a 63.5 cm x 63.5 cm x 1.27 cm (25in. 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 of compaction. During compaction, two vacuum pumps of 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 always touching the internal perimeter of the compaction mold and preventing any damage. A seal is applied between the compaction mold half cylinders in order to allow for applying vacuum during sample compaction. Figure 4.9 depicts a cross-section of the automatic tamper, and Figure 4.10 pictures the automatic tamper, compaction mold and membrane, base plate, collar, spacer disk and vacuum pumps used during the compaction of a soil sample in the LSU/CALCHAS. A general view of the automatic tamper is also depicted in Figure 4.12.

4.2.6. Penetration Depth Measurement System

In order to obtain an accurate depth measurement during the penetration phase, an analog to digital converter depth decoding system was developed and incorporated to the LSU/CALCHAS. The depth decoder is composed of a metal disk, a light emitting diode and an optical sensor. Holes are drilled at equal
distances on the circumference of the metal disk. As the cone penetrates the sample, a cable connected from the drill rod to the shaft of the disk mechanically turns the disk. The distance between two consecutive holes on the circumference of the metal disk represents a penetration of 2 cm. The light emitting diode and the optical sensor are installed on either side of the disk. When the light emitted by the diode passes a hole, the optical sensor senses the light and thus generates a pulse to the control unit that triggers the multiplexer to switch on the channels for analog to digital conversion. This process continues until the penetration motion is stopped. Figure 4.11 shows details of the depth decoding system developed for the present research.

4.2.7. Auxiliary System

The LSU/CALCHAS auxiliary system consists of a trolley crane system, a hanging scale and a mixer. Figure 4.12 depicts the crane system, hanging scale and mixer.

The crane system is comprised of a one ton crane moving in a wing beam, a two ton trolley crane moving on a horizontal beam supported on two transversalis beams in such a way that the beams-crane movement covers the total area of the LSU/CALCHAS. In addition, a one ton crane moving on an upper transversa beam provides higher working versatility. The one ton wing beam crane allows for moving
soil samples and calibration chamber parts in and out of the laboratory; the two ton crane permits transfer of the soil sample from the compactor's base plate to the calibration chamber and, after test, from the chamber to the floor; and, the one ton upper beam crane allows for accurate placement of the sample cell walls from the floor to the chamber and vice-versa.

The hanging scale is the MSI-3260 Challenger digital crane manufactured by Measurement Systems International, with a maximum load capacity of 2000 lb and a reading precision of 1 lb. It is integrated to the two-ton crane and used for determining the soil sample weight after compaction.

The mixer is a utility concrete mixer manufactured by Olympia Industrial Inc. with 1/3 HP motor, drum rpm of 30 - 32 and drum capacity of 3.5 cu.ft.

4.2.8. Calibration Chamber Operation

The LSU/CALCHAS allows for $K_p$ consolidation of the soil sample and application of the four traditional boundary conditions commonly referred in the literature to the sample during a CPT. The calibration chamber operation can be subdivided in three steps: the consolidation phase, the penetration phase, and instrumentation.
4.2.8.1. Consolidation Phase

In the LSU/CALCHAS, soil samples can be consolidated under the condition of zero lateral strain \((K_o)\). Other loading paths can be simulated in the chamber consolidation phase, but they may require minor changes in the panel of control operation as well as in the computer software developed for data acquisition.

Boundary condition \(K_o\) compresses the sample under conditions of zero lateral strain. It simulates the loading history of a soil layer undergoing unidimensional compression, which can be considered as the simplest loading case for analysis, and is usually adopted for sample consolidation in calibration chamber research.

During \(K_o\) consolidation, the piston cell, the inner and outer cells of the sample are entirely filled with water. The vertical consolidation stress level is given via the computer keyboard, in psi. The computer software developed for this research allows for reaching the given consolidation stress level using successive stress increments via D/A conversions in the DT 2801A series data translation board in connection with the Fairchild model 5700 electro-pneumatic transducer mounted in the panel of controls. The output air pressure from the electro-pneumatic transducer is transmitted via an air-water system to the piston outer cell, and the soil sample is vertically compressed. As the sample is compressed, the increases in the water pressure in the sample inner cell are monitored via a SenSym pressure transducer installed in the panel of controls, and the computer software allows for
an equivalent pressure to be applied to the water in the sample outer cell via D/A conversions using the DT 2801A data translation board and another Fairchild model 5700 electro-pneumatic transducer from the control panel. Therefore, at all times the pressures in the sample inner and outer cells are matched. Since the pressures acting on both sides of the sample inner wall can be considered the same, this wall can not move. The diameter of the piston cell top plate equalizes the soil sample diameter, and since the sample inner cell wall can not move, the sample is compressed under zero average lateral strain.

During the loading phase, the vertical stresses applied, the vertical deformation and the lateral stresses developed are automatically recorded, saved in a file and displayed in the computer screen in real time during consolidation. Values such as stress ratio ($K_o$), constrained modulus ($M_o$), among others, can be calculated.

After vertical compression of the soil sample, the boundary condition for the penetration phase of the consolidated sample is selected. Overconsolidated soil samples can be obtained by lowering the vertical stress from a desired preconsolidation pressure while still maintaining zero lateral strain conditions.
4.2.8.2. Penetration Phase

Four boundary conditions can be applied to the soil sample in the LSU/CALCHAS penetration phase, as follows:

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

For boundary condition 1, the water pressures in the piston cell and sample inner cell are kept constant via two Fairchild model 10 BP precision back pressure regulators. These regulators work like pressure relief valves for the outer piston and inner sample cells. Therefore, any increase in the vertical and horizontal water pressures generated during penetration is vented out of the system. In this boundary condition, cone and local side friction resistances, vertical and horizontal pressures in the outer piston and inner sample cells, and vertical displacement are automatically recorded, saved in a file and displayed at real time in the computer screen during penetration.

Under boundary condition 2, the soil sample should be resting on a rigid base, the piston cell. This can be accomplished by closing the valve between the
piston cell and the panel of controls air-water system. Zero lateral strain is maintained in a similar way to the consolidation phase. Cone and local side friction resistances, water pressures in the sample inner and outer cells are automatically recorded, saved in a file and displayed in the computer screen in real time as the penetration test develops.

Boundary condition 3 requires that the outer piston cell water pressure generated during penetration be released, as well as the sample inner and outer cells water pressures be kept equal. This is accomplished via procedures already explained in BC1 and $K_c$ consolidation. Cone and local side friction resistance, piston outer cell water pressure, sample inner and outer cell water pressures, and vertical displacement are automatically recorded, saved in a file and displayed in real time in the computer screen as the test advances.

Boundary condition 4 uses a rigid sample bottom base, the piston cell as in BC2, and requires that all water pressure generated during penetration be released. This is accomplished via procedures described in BC1 and BC2.

During the penetration phase, the cone penetrometer is forced into the soil sample by a hydraulic system connected to a grabbing device. The push jack and grabber are mounted on the top of the chamber top plate, and they use the chamber structure as a reaction frame when penetrating or removing the cone penetrometer from the soil sample. The speed of penetration is controlled by a hydraulic flow.
valve and is kept constant at 2 cm/sec.

Of the four possible boundary conditions for penetration, the two extremes are BC1 and BC3. As referred in the literature review, it is advocated that a soil sample in the field, if subjected to the same loading history as a calibration chamber sample, would have a penetration resistance which would lie between the penetration resistance measured from chamber boundary conditions BC1 and BC3.

4.2.8.3. Chamber Instrumentation

Instrumentation of the LSU/CALCHAS allows for all of the pertinent test data to be automatically recorded. The data to be 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.

- **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.

For boundary conditions 1 and 4, the soil sample volume change can be measured visually by measuring the volume of water flowing into and out of the inner sample cell via a sight glass tube installed in the air-water system connected to the chamber sample cell.

Both the vertical and lateral stresses acting on the sample are measured using the mid sample height as a datum. The vertical stress on the base of the soil sample is measured by monitoring the water pressure developed in the piston cell. The lateral stresses are measured by monitoring the water pressure developed in the sample inner and outer cells.

The lateral and vertical pressures are measured using electrical SenSym pressure transducers. These transducers are located in the panel of controls, outside the chamber, at mid piston cell and mid sample height, respectively. To provide a visual check on pressures developed during testing, Marsh process gauges are used to monitor the lateral and vertical pressures in the panel of controls.
The vertical deflection of the soil sample is measured by measuring the movement of the piston at the piston cell. A linear variable differential transducer (LVDT) manufactured by Trans-Tek, model number 353-000, mechanical travel of 1.25 in, is connected to the piston in the piston cell and touching the chamber bottom plate. This allows the LVDT to measure the movement of the piston cell piston in relation to the chamber bottom plate, and hence the vertical deformation of the soil sample.

4.3. Data Acquisition and Reduction Software

A major task in the present research was to develop a data acquisition and reduction software capable of fully controlling the operational process involved in the consolidation and penetration phases of the LSU/CALCHAS. A computer code in excess of 7500 lines was written for this purpose. A 3.5 in floppy diskette which contains this software is given in Appendix 5. Besides taking into account the particularities common to the \( K_o \) consolidation phase and each possible boundary condition during the penetration phase, the software is able to handle the following chores:

1. Straight forward operation without further intervention of the LSU/CALCHAS operator, besides the required keyboard input for initiating the consolidation and penetration phases;
(2). To acquire data from eight electric transducers via an analog to digital conversion (A/D), and to have flexibility for acquiring data from up to sixteen electric transducers;

(3). To operate two transducers via digital to analog conversion (D/A);

(4). To acquire the data, append it to a file, display the pertinent readings in a computer screen in a graphic form at real time at their actual units of measurement, close the file after each batch of reading is taken, and to respect the time increment of 1 second for all operations;

(5). To produce offline high quality output using a printer or the HP 7475A plotter;

(6). To be capable of developing the graphic part of the software in a device-independent environment in such a way that any change in the output device configuration would require only the installation of the appropriate device driver and minimal reprogramming;

(7). To be written in a structured language in such a way that future implementation of the LSU/CALCHAS should require only minor reprogramming.
This task was accomplished by selecting the following data acquisition hardware and computer language:

(1). Gateway 2000 PC 386 microcomputer, with 640 K Ram memory, 1.2 Meg 5.25 in floppy drive, 1.44 Meg 3.5 in floppy drive, 16 bit VGA with 256 K, 80 Meg 28 ms RLL hard disk drive, monochrome monitor;

(2). Data translation DT-2801A analog to digital conversion and digital I/O board, by Data Translation;

(3). The software was developed in the Turbo Pascal version 4.0 environment by Borland International, and the HALO'88 graphics library by Media Cybernetics.

The data translation board is capable of performing A/D conversion, D/A conversion, digital I/O transfers, report errors in the operation board, set the period of the on-board clock, stop board operations in process, reset some of the board's programmable parameters and perform simple tests on the board. It is a complete single board data acquisition system for the IBM Personal Computer, with sixteen channels 12-bit A/D, 2 channels 12-bit D/A, on-board microprocessor, IBM PC interface, sixteen lines digital I/O, programmable clock and power supply. In this system, the user's signal lines are transmitted to the board via the output connector of the screw terminal panel.
The HALO'88 library displays a collection of high performance graphic subroutines that enables the user to create and incorporate sophisticated computer generated images. The library encompasses more than two hundred functions which eliminate the need to generate the source code needed to implement graphics. Using this library, the user can create and develop the image and then control the placement, color, and size of that image via an access library. HALO'88 was used in the data acquisition and reduction software as the basic graphics source.

The computer program was written in the Turbo Pascal version 4.0 environment. Turbo Pascal allows for structured programming, closely follows the definition of Standard Pascal, and combines the editing, compiling and linking capabilities in the same environment.

A directory named CHAMBER, with the data acquisition and reduction software is available in the Gateway 2000 PC allocated for the LSU/CALCHAS operation.

4.3.1. Data Acquisition Software

The data acquisition software encompasses five computer programs written in Turbo Pascal version 4.0 around the HALO'88 graphics library environment. For clarity and considering the software compiler limitation, it was decided to approach
the consolidation phase and each of the boundary conditions during the penetration phase via an independent software, as follows:

(1). Consolidation Phase: CHAMBKo.EXE;

(2). Penetration Phase:
   2.1. Boundary Condition 1: CHAMBC1.EXE;
   2.2. Boundary Condition 2: CHAMBC2.EXE;
   2.3. Boundary Condition 3: CHAMBC3.EXE;
   2.4. Boundary Condition 4: CHAMBC4.EXE.

A general description of the operational steps involved in the consolidation and penetration phases can be found in items 2.8. Calibration Chamber Operation, 4.3. Consolidation Phase, and 4.4. Penetration Phase of this dissertation. The total programming involved in the data acquisition software accounts for nearly six thousand lines.

4.3.2. Data Reduction Software

The data reduction software environment is similar to the one used for developing the data acquisition software. Clarity and software compiler limitation also prompted to develop two distinct plot/printer programs for the consolidation
and penetration data, as follows:

(1). Consolidation Phase: CHAMBKoP.EXE;

(2). Penetration Phase: CHAMBERP.EXE.

The plotting/printing computer software developed for the consolidation phase, program CHAMBKoP.EXE, requires compliance with the following steps:

1. Input of filename for display/plot: This is the filename that was generated in the $K_0$ consolidation phase;

2. Selection of graphic to plot: The graphic options horizontal stress versus vertical stress and vertical displacement versus time are available in the computer screen;

3. Input of pressure transducer range used during testing (psi): The pressure ranges of 0 to 30 psi and 0 to 100 psi are available in the computer screen window;

4. Option for a print or a plot?: The computer screen displays a Y/N option. Choosing Y, it sends the user to a printer hardcopy, and selecting N, allows for a plotter hardcopy;
The computer software developed for the penetration phase, program CHAMBERP.EXE, requires conformity to the following steps:

1. Input of filename for display/plot: This is the filename that was generated in the penetration phase, under a specific boundary condition;

2. Selection of total depth for plotting: Three depths of sounding are available as follows: 40 cm, 60 cm and 80 cm;

3. Option for a print or a plot?: The computer screen displays a Y/N option. Choosing Y, it sends the user to a printer hardcopy, and selecting N, allows for a plotter hardcopy;

4. Computer screen with graphic to be printed/plotted;

5. Printing/plotting in progress;
6. Press any key to finish printing/plotting.

The computer programs developed for data reduction account for about seventeen hundred lines.

4.4. Test Procedure

The laboratory testing program of the present research is related to the preparation and testing of compacted soil samples in the LSU/CALCHAS. 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.
4.4.1. Sample Preparation

A requisite in the present research was the preparation of compacted soil samples in such a way to reproduce the maximum dry density and optimum moisture content determined with the Proctor compaction test for a specific compaction effort. To fulfill this requirement a large dimension automatic tamper and compaction mold were designed and fabricated with the following characteristics:

1. Weight of the sliding hammer (P): Two rod lengths are available, leading to the total weights of 65.40 kg (144 lb) and 70.60 kg (155.6 lb);
2. Drop height: adjustable;
3. Volume of compaction mold: 170,000 cm³.

For the same type of soil and compaction technique, using equipment of different dimensions should result in acceptable soil characteristics, i.e. similar maximum dry density and optimum moisture content, when the compactive efforts or the work accomplished in operating the rammer on different soil sample sizes are the same. The compaction effort can be estimated as follows (Head, 1984):

\[ E = \frac{(P \cdot L \cdot N \cdot n)}{V} \]

where:

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

In order to reproduce a specific compactive effort in the large dimension automatic tamper developed for this research, adjustments in the drop height, number of blows per layer and number of layers have to be made. For instance, considering the standard Proctor compactive effort, ASTM D 698-78 recommends the following values for the energy equation:

\[
P = 2.49 \text{ kg}; \\
L = 305 \text{ mm}; \\
N = 56 \text{ blows per layer}; \\
n = 3 \text{ layers}; \\
V = 2124 \text{ cm}^3
\]

resulting in

\[
E = 589 \text{ kJ/m}^3
\]

The standard compactive effort can be reproduced with the large dimension automatic tamper if the following adjustments are made:

\[
L = 711.2 \text{ mm (28 in)}; \\
N = 41 \text{ blows per layer, when using the 70.64 kg rammer; and, 44}
\]
blows per layer, when using the 65.376 kg rammer;

\[ n = 5 \text{ layers; } \]

resulting in

\[ E = 590 \text{ kJ/m}^3. \]

For other compactive efforts, the same computational method can be used in order to determine the modified rammer drop height and the number of blows per layer, keeping the number of layers the same.

The first step in sample preparation is to produce an homogeneous soil mass with the desired moisture content determined in the compaction test. A mixer with capacity of 3.5 ft³ is used for this purpose in this investigation. It is advisable to prepare the whole soil sample in series of ten small samples or more, because of limitations in mixer capacity. After the compaction mold is fixed to the baseplate and the membrane is affixed to its interior, the whole unit is weighed using the electronic hanging scale hooked to the two-ton trolley crane. Then the extension collar is added to the compaction mold. Vacuum is applied between the compaction mold and membrane in order to keep the membrane as flush as possible to the compaction mold wall. The next step is to compact the soil sample. Before compaction of each soil layer, three samples were taken for determination of moisture content. After completion of the compaction of the five soil layers in the compaction mold, the extension collar is removed and the sample is trimmed. At this stage the compaction mold, sample, baseplate and membrane are weighed again.
The difference between this measurement and the previous one gives the sample weight which allows for the determination of the sample dry density.

Figures 4.13 and 4.15 show various stages of the soil mixing process, compaction of the sample, and sample transfer from the compactor baseplate to the chamber.

4.4.2. Chamber Preparation

The first step in sample preparation is to fill completely the piston outer cell with water from the reservoir at the control panel. This is done by directing the water flow from the reservoir to the piston cell water line via valve A4 and closing valve B15 in the control panel. After the outer piston cell is filled with water, any air bubbles that might be present shall be removed via the piston cell bleeding valve V3. Once the outer piston cell if filled with water and free of any air bubbles, there is no need to fill it again for further tests in the LSU/CALCHAS.

After determining the combined weight of the compaction mold, baseplate, membrane and the sample, the whole unit is moved and lowered on top of the chamber piston cell. The compaction mold is taken apart, the baseplate is kept as the sample bottom plate, the sample top plate is carefully added to the top of the sample, and the membrane is fixed to the sample bottom plate and sample top plate.
via two O-rings. The chamber top plate is then carefully lowered into position on the top of the sample, and the chamber top plate is fixed to the piston cell ring via twelve steel 304 rods with 12.7 mm in diameter tightened at a torque of 70 newton-meters (around 60 foot-pounds). Water from the control panel reservoir is then introduced into the sample inner and outer cells, filling them completely. Any air bubbles that might be present are removed by flushing the chamber water lines and opening the bleeding valves V1 and V2 at the top of the chamber top plate. At this stage, close valve A4. The chamber is then ready for testing.

Various stages of chamber preparation for testing are depicted in Figures 4.15 and 4.16.

4.4.3. Consolidation Phase

This phase involves consolidation of the soil sample under zero lateral strain. Figure 4.17 illustrates the LSU/CALCHAS control panel configuration developed for simulating this situation. The dotted lines represent the inactive part of the panel. The electric-pneumatic transducers in the control panel have an offset of around 5.6 and 3 psi for the ranges 5.6 to 30 psi (transducers F1 and F3) and 3 to 120 psi (transducers F2 and F4), respectively. Closing valves B11 and B17 and opening valves B6 and B13 it is possible to have all the SenSym pressure transducers in the control panel at zero pressure. Zero readings are then taken for transducers
S1, S2, S3, S4 and S5, as well as for the LVDT measuring deformation of the piston cell, cone tip and friction sleeve. After opening valves B11 and B17, and closing valves B6, B8 and B13, the system is ready for testing.

The computer software developed for this phase, program CHAMBKo.EXE, requires compliance with the following steps:

1. Input of filename for $K_c$ consolidation data storage;

2. Laboratory testing information on Ko consolidation phase, as follows: soil sample, compactive effort, consolidation stress in psi, sample dry density, sample water content and test number;

3. Selection of pressure transducer range: The pressure ranges of 0 to 30 psi and 0 to 100 psi are available in the computer screen window;

4. Optional use of a default equilibrium pressure: A YES/NO option is available. More details about this step is given at the end of this section;

5. Selection of electric-pneumatic transducer range: The pressure ranges of 6 to 30 psi and 3 to 120 psi are available in the computer screen window;
6. Input of consolidation stress (psi);

7. Computer will flash a screen with graphics which will feature real time development of cone resistance, local side friction resistance sample vertical deformation, vertical stress, and horizontal stresses in the sample inner and outer cells. This screen is depicted in Figure 4.18;

8. Press any key to finish test.

The equilibrium pressure is the vertical pressure that has to be applied to the inner piston cell water in order to equilibrate the pressure generated by the weights of sample, sample bottom plate, membrane and inner piston cell. Any pressure acting on the inner piston cell water which is higher than this value will cause an upward movement, starting the consolidation phase.

The equilibrium pressure can be estimated dividing the total weight by the inner piston cell bottom plate area which is 2200 cm². A default pressure of .21 kg/cm² (3 psi) based on a soil sample with 80% of fine sand and 20% of kaolinite, compacted at the standard Proctor compaction effort around the optimum water content is available in the computer program. If the user's answer is NO, he opts to determine the sample equilibrium pressure via the LSU/CALCHAS control panel. In this case, the computer program will read the equilibrium pressure from the transducer S4 or S5 and incorporate it into the proper calculations. In this situation,
before entering the keyboard corresponding to answer NO, the operator should adjust the control panel in order to reach the sample equilibrium pressure. It can be done by directing the compressed air flow from the air filter to the transducer F1, keeping valves B13 and B16 open in the control panel, and controlling the pressure in the vertical pressure line via the back pressure regulator BP2. When the sample equilibrium pressure is reached, and the sample touches the sample top plate, the user should press the keyboard corresponding to answer NO, close valves B3, B13 and B16, and direct the compressed air flow from the air filter to the E/P transducer F1/F2 chosen for testing. At this stage, the computer program will take zero readings for all Sensym transducers, cone tip and friction sleeve. These zero readings should be taken with the horizontal and vertical pressure lines in the panel under zero pressure. This can be accomplished via the back pressure regulators BP1 and BP2 by closing valves B10/B11, B16/B17, and opening valves B6 and B13 in the panel of controls. Immediately after the zero readings are taken and displayed in the computer screen, and before going to the next computer screen, the operator should close valves B6 and B13, and open valves B10/B11 and B16/B17, in order to have the outer sample cell horizontal pressure line and the vertical pressure line at the F1/F2 and F3/F4 transducer offsets, respectively.

After completion of the consolidation phase, the sample is ready to be penetrated.
4.4.4. Penetration Phase

4.4.4.1. Boundary Condition 1

Boundary condition 1 requires to penetrate the soil sample under conditions of zero lateral and vertical stress increments. A control panel configuration set for this boundary condition is portrayed in Figure 4.19, where the dotted lines represent the inactive part of the panel.

The first step is to set the Fairchild back pressure regulators BP1 and BP2 to the vertical and horizontal stress levels attained at the end of the consolidation phase, respectively. This can be done by closing valves B6 and B13 and opening valves B18 and B19. The desired horizontal and vertical pressures are achieved by turning the adjusting screw on the top of the respective pressure regulator. The pressure is checked via the Marsh process gauges G4 and G5.

After the back pressure regulators G1 and G2 are set to the consolidation horizontal and vertical stress levels, respectively, close valves B7, B11 and B17 on the control panel, and the system will be ready for testing.

The computer software developed for this phase, program CHAMBC1.EXE, requires compliance with the following steps:
1. **Enter filename for zero readings input:** This is the file that was opened for zero readings at the consolidation phase;

2. **Enter filename for cone data storage;**

3. **Laboratory testing information on Ko consolidation phase:** soil sample, compactive effort, boundary condition, consolidation stress, dry density, water content and test number;

4. **Option to change display setting of CPT:** An Y/N option is available. Three sounding depths are available: 40 cm, 60 cm and 80 cm;

5. **Selection of pressure transducer range:** The pressure ranges of 0 to 30 psi and 0 to 100 psi are available in the computer screen window;

6. **Computer will flash a screen with graphics which will feature real time development of cone resistance, local side friction resistance, sample vertical deformation, vertical stress, horizontal stress in the inner sample cell. This screen is depicted in Figure 4.20;**

7. **Press any key to finish test.**
4.4.4.2. Boundary Condition 2

Boundary condition 2 refers to a condition of zero vertical and lateral strains in the sample during cone penetration. This situation can be accomplished via the configuration of the control panel layout shown in Figure 4.21. In this figure, the dotted lines represent parts of the panel which are inactive during penetration. For this boundary condition, valves B6, B8 and B15 on the control panel shall be in the closed position.

The computer software developed for this phase, program CHAMBC2.EXE, requires compliance with the following steps:

1. Enter filename for zero readings input: This is the file that was opened for zero readings at the consolidation phase;

2. Enter filename for cone data storage;

3. Laboratory testing information on Ko consolidation phase: soil sample, compactive effort, boundary condition, consolidation stress, dry density, water content and test number;

4. Option to change display setting of CPT: A Y/N option is available. Three sounding depths are available: 40 cm, 60 cm and 80 cm;
5. Selection of pressure transducer range: The pressure ranges of 0 to 30 psi and 0 to 100 psi are available in the computer screen window;

6. Selection of electric-pneumatic transducer range: The pressure ranges of 6 to 30 psi and 3 to 120 psi are available in the computer screen window;

7. Computer will flash a screen with graphics which will feature real time development of cone resistance, local side friction resistance, vertical stress, horizontal stresses in the inner and outer sample cells. This screen is depicted in Figure 4.22;

8. Press any key to finish test.

4.4.4.3. Boundary Condition 3

Boundary condition 3 reflects a state of constant vertical stress and zero lateral strain during cone penetration. The control panel configuration adjusted to this boundary condition is presented in Figure 4.23, where the dotted lines delineate the part of the panel which is inactive during testing. Before starting the test, valves B6 and B17 on the control panel shall be in the closed position.
The computer software developed for this phase, program CHAMBC3.EXE, requires compliance with the following steps:

1. Enter filename for zero readings input: This is the file that was opened for zero readings at the consolidation phase;

2. Enter filename for cone data storage;

3. Laboratory testing information on Ko consolidation phase: soil sample, compactive effort, boundary condition, consolidation stress, dry density, water content and test number;

4. Option to change display setting of CPT: A Y/N option is available. Three sounding depths are available: 40 cm, 60 cm and 80 cm;

5. Selection of pressure transducer range: The pressure ranges of 0 to 30 psi and 0 to 100 psi are available in the computer screen window;

6. Selection of electric-pneumatic transducer range: The pressure ranges of 6 to 30 psi and 3 to 120 psi are available in the computer screen window;
7. Computer will flash a screen with graphics which will feature real time development of cone resistance, local side friction resistance, vertical stress, horizontal stresses in the inner and outer sample cells. This screen is depicted in Figure 4.24;

8. Press any key to finish test.

4.4.4.4. Boundary Condition 4

Boundary condition 4 demands to penetrate the soil sample under zero vertical strain and constant lateral stress. Figure 4.25 introduces the control panel configuration layout set for this boundary condition, where the dotted lines represents parts of the panel which are inactive during testing. Before starting the test, valves B7, B11 and B15 shall be placed in the closed position.

The computer software developed for this phase, program CHAMBC4.EXE, requires compliance with the following steps:

1. Enter filename for zero readings input: This is the file that was opened for zero readings at the consolidation phase;

2. Enter filename for cone data storage;
3. Laboratory testing information on Ko consolidation phase: soil sample, compactive effort, boundary condition, consolidation stress, dry density, water content and test number;

4. Option to change display setting of CPT: A Y/N option is available. Three sounding depths are available: 40 cm, 60 cm and 80 cm;

5. Selection of pressure transducer range: The pressure ranges of 0 to 30 psi and 0 to 100 psi are available in the computer screen window;

6. Computer will flash a screen with graphics which will feature real time development of cone resistance, local side friction resistance, vertical stress, horizontal stress in the inner sample cell; This screen is depicted in Figure 4.26;

7. Press any key to finish test.

4.4.5. Control Panel Shut-Down and Removal of the Sample

After completion of the penetration phase, the three water lines of the chamber must be disconnected via the quick-connectors C1, C2 and C3, isolating the chamber from the control panel.
To shut off the control panel, disconnect the compressed air line at the bottom of the panel, turn off the computer system and open valves B6 and B13 carefully. This will bring the panel pressure lines to zero.

To remove the sample from the chamber, carefully open the bleeding valves V1 and V2 at the top of the chamber. When pressures in the inner and outer sample cells are atmospheric, dismantle the chucking system, dismount the cone penetrometer and chamber top plate, drain the water from the inner and outer cells, move the inner and outer sampler cell walls to the floor using the one-ton crane installed in the upper beam, remove the sample top plate, strip out the membrane, assemble the split compaction mold onto the sample and piston cell top plate, move the sample from the chamber frame to the floor using the two-ton trolley crane, take apart the split compaction mold and sample bottom plate, and finally dispose of the sample.

4.5. Laboratory Test Results

The laboratory testing program of this investigation envisaged the development of a calibration chamber system (LSU/CALCHAS) capable of performing cone penetration tests in compacted soil samples, under specified boundary restraints. A large dimension compactor fabricated for sample preparation and a chamber servo panel which enables computer controlled execution of cone
penetration tests using the MQSC complete the system.

This section describes the experimental work carried out for the verification of the laboratory testing equipment. An artificially prepared mixture of 80% of fine sand and 20% of kaolinite in dry weight was used in all laboratory tests. The tests performed can be divided in two subsections, namely traditional laboratory tests and calibration chamber tests.

4.5.1. Traditional Laboratory Tests

Traditional laboratory tests were conducted on a mixture of 80% of fine sand and 20% of kaolinite in dry weight. The sand and kaolinite used in this research were furnished by the Feldspar Corporation from Edgar, Florida. The sand is a "Glass-Grade" type, with mean grain diameter between .425 and .106 mm (sieves #40 and #140, respectively) obtained from a sedimentary deposit in Florida. The kaolinite is regarded to be the fruit of eolic transportation from the mountains of North Carolina. After excavation of the deposits in Edgar, the sand and kaolinite are processed and packaged in 50 lb. sacks.

Atterberg limits performed on the soil mixture resulted in liquid limit of 25% and plastic limit of 18% (plastic index of 7). Compaction tests on the mixture performed at the Standard Proctor Compactive Effort resulted in optimum moisture
content of 10.9% and maximum dry density of 18.48 kN/m³ (117.65 lb/cu ft). Figures 4.27 to 4.29 depict the sieve analyses performed on the sand, kaolinite and sand-kaolinite mixture. Figure 4.30 shows Standard and Modified Proctor compaction tests performed in the sand-kaolinite mixture.

4.5.2. Calibration Chamber Tests

The LSU/CALCHAS is capable of consolidating soil samples under Ko condition and simulating the four traditional boundary conditions commonly referred in the literature, i.e. BC1, BC2, BC3 and BC4, during the penetration phase.

In the calibration chamber tests, the mixture of 80% of fine sand and 20% kaolinite was compacted to reproduce the maximum dry density and optimum water content of the Standard Proctor Compaction Effort. After compaction, the soil sample was consolidated at Ko state under a vertical stress of 30 psi, before penetration. Considering that in the LSU/CALCHAS the capability of simulating boundary conditions 1 and 3 implies automatically in the possibility of reproducing boundary conditions 2 and 4, the penetration phase tests were conducted under BC1 and BC3, exclusively.

A virgin cone penetration test was attempted in the completed soil sample in order to verify the final stage of development of the data acquisition software. For
this test, the sample was consolidated at 68.9 kPa (10 psi) and then penetrated under BC1. The consolidation phase was performed via eight stress increments applied for 600 seconds each, totaling 4800 seconds. Readings were taken at each 10 seconds increments. However, because of unintentional measurement errors in the vertical stress and cumulative time during the consolidation phase, this preliminary phase data are not discussed in the text, but only shown in Appendix 4 for demonstrative purposes. After final adjustments in the software structure were completed, the same soil sample was consolidated at the vertical stress 207 kPa (30 psi) and then penetrated two additional times (at different entry locations) under boundary conditions 1 and 3, respectively. The location coordinates of the cone penetration tests performed in the soil sample are displayed in Figure 4.31.

The compaction, Ko consolidation and penetration phases are discussed in the next subsections.

4.5.2.1. Sample Compaction

The Standard Compaction Proctor characteristics of the sand-kaolinite mixture is depicted in Figure 4.30, and can be summarized as follows:

. Optimum Moisture Content = 10.9%

. Maximum Dry Density = 18.46 kN/m³ (117.65 lb/cu ft)
In an attempt to simulate the Standard Proctor Compaction Effort, the sand-kaolinite mixture was compacted using the large dimension automatic tamper designed and fabricated for this research. The soil sample was compacted in five lifts of 15.8 cm (6.22 in) each, adopting a rammer drop height of 71.12 cm (28 in). Limitations in the ceiling height of the LSU Geotechnical Laboratory required the last three layers to be compacted using a shorter rammer rod. Therefore, the first two sample layers were compacted with a rammer weight of 70.64 kg (155.6 lb) and the last three layers with a rammer weight of 65.37 kg (144 lb), resulting in 41 blows for the first two and 44 blows for the last three soil layers to keep the compaction effort consistent.

The moisture content of the soil sample (w) was determined from three samples taken from each soil layer, resulting in an average value of 11.46%. Table 5 shows the pertinent sample moisture content data. It should be indicated that the sand-kaolinite mixture was purposely prepared at a water content around 11.7%, expecting that this higher water content could take into account possible loss due to the ambient conditions in the laboratory and the time required for mixing and compacting the soil sample. This presumption came out to be partially true, but the average sample water content resulted to be somewhat higher than the ideal optimum moisture content determined in the Standard Proctor Test.

The sample dry density ($\gamma_d$) was determined as follows:

1. Total weight (sample, former, membrane, sample bottom plate): $W_t$,
\[ W_t = 502.85 \text{ kg (1107.60 lb)} \]

Weight of former, membrane and sample bottom plate: \( W_t \)

\[ W_t = 152.50 \text{ kg (336 lb)} \]

Sample volume: \( V = 170,000 \text{ cm}^3 \)

Sample dry density:

\[
\gamma_d = \frac{(W_t - W_i)/(1+w))}{V}
\]

\[ \gamma_d = 18.14 \text{ kN/m}^3 \ (115.57 \text{ lb/ cu ft}). \]

The sample water content of 11.46\% is higher than the expected optimum moisture content of 10.9\%, consequently resulting in a dry density of 18.13 kN/m\(^3\) which is slightly lower than the anticipated sample maximum dry density of 18.46 kN/m\(^3\). However, from Figure 4.30, Standard Proctor Test, at a water content of 11.46\% it gives a dry density of 18.29 kN/m\(^3\). If it is considered that this dry density translates only .87\% higher than the target dry density \( \gamma_d = 18.13 \text{ kN/m}^3 \), it can safely be assumed that the compaction phase of the present research was satisfactory.

\textbf{4.5.2.2. Consolidation Phase}

The soil sample was consolidated under \( K_o \) condition at the vertical stress level of 207 kPa (30 psi). Ten stress increments were applied during the consolidation process. Each stress increment lasted for 30 seconds, totalling a sample consolidation time of 300 seconds. For each stress increment, a reading was
taken at the very beginning, followed by three more readings at intervals of 10 seconds between two consecutive readings.

During the consolidation phase, the vertical stresses applied to the sample, the generated horizontal stresses and the vertical sample deformations were truly recorded. This data was used in the determination of the $K_0$ value and the modulus of vertical compression of the soil sample.

Figures 4.32 and 4.33 show, respectively, the applied vertical stress versus generated horizontal stress and sample vertical deformation versus time relationships developed during the consolidation phase. Figure 4.32 introduces the $K_0$ ratios generated during the consolidation phase. It seems that a $K_0$ value of 25 can be associated to the sample consolidation phase.

Figure 4.34 depicts the evolution of vertical and horizontal stresses with time during the consolidation phase. A question concerning the shape of the stress curves could be raised. It would be expected that the applied vertical stress increments should generate increases in the stress curves in the vertical direction with time. However, this is not deduced in Figure 4.34. Apart from the response time required by the LSU/CALCHAS in order to change the horizontal stresses due to incremental increases in the applied vertical stress, the true stress-time relationship in the system shows in fact movement in the vertical direction when a vertical stress increment is applied to the soil sample. This unrealistic pattern of the stress-time...
curves depicted in Figure 4.34 is due to the way the data acquisition software developed for the consolidation phase handles its task. Immediately after the software has sent a vertical stress increment signal to the control panel via a D/A conversion, readings are taken from all electric transducers involved in the penetration phase via A/D conversions. A delay of 1 seconds is introduced in order to allow for the LSU/CALCHAS response time. After this delay, a new reading is taken from the sample inner cell pressure line via transducer S2/S3 and, instantaneously, this impulse is sent to the outer cell pressure line via a D/A conversion in order to keep the sample inner and outer cell at the same pressure, i.e. to maintain the sample under $K_u$ state. Ten seconds after the first set of readings are taken from the system and stored in the consolidation phase file, a new set of readings are also taken and stored. This procedure attempts to keep the consolidation phase file in a manageable size. The time between consecutive readings, presently 10 seconds, can be changed as a function of the soil type being tested. Therefore, the procedure adopted for the software development generates an unreal evolution of the consolidation phase stress curves when plotted against time. However, this fact does not affect the integrity of the measured data, because the deformation, vertical and horizontal stresses readings are always taken at the same time. Same explanation applies to the sample deformation versus time curve depicted in Figure 4.33.

The development of the horizontal stresses in the sample inner and outer cell can be also addressed in Figure 4.34. This figure shows that the outer cell has an
initial pressure offset, and only after around 160 seconds the inner and outer cell show a close increase on the horizontal stresses. It could be argued that this situation does not represent the hypothesized double flexible wall chamber behavior during the $K_o$ consolidation, i.e. the inner and outer cell to be kept under the same pressure in order to allow for an average zero lateral strain situation. However, if it is taken into account that for all situations where the pressure in the inner cell is lower than the pressure in the outer cell it will implies a condition of zero lateral strain, it is possible to conclude that the calibration chamber actually works as postulated.

Figure 4.35 depicts the variation of the measured vertical stresses plotted against vertical strain for the compacted soil sample. The shape of the curve is relatively uniform, suggesting a mathematically related expression. A polynomial equation of third order is used in order to estimate the sample stress-strain relationship, as follows:

$$\sigma_v = A_o + A_1 \varepsilon + A_2 \varepsilon^2 + A_3 \varepsilon^3$$

where

$\sigma_v$: measured vertical effective stress (kPa);

$\varepsilon$: sample vertical strain

$A_o$, $A_1$, $A_2$, $A_3$: constants.

The fitted stress-strain relationship is also plotted in Figure 4.35 as a solid line, and it closely follows the experimental data points.
The modulus of vertical compression \( (M_v) \) is the slope of the stress-strain curve at any given point. It is clear that the slope of the stress-strain curve increases with increasing vertical stress. Therefore, it can be expected that \( M_v \) is not constant. The modulus of compression can be estimated from the fitted stress-strain relationship, as follows:

\[
M_v = \frac{d\sigma}{d\epsilon} = A_1 + 2A_2\epsilon + 3A_3\epsilon^2
\]

Figure 4.36 depicts the estimated compression modulus \( M_v \) versus the measured vertical stress of the compacted sand-kaolinite mixture.

4.5.2.3. Penetration Phase

The LSU/CALCHAS has capability of performing computer-controlled cone penetration tests under boundary conditions 1 and 3 which automatically indicates its competence for executing penetration tests under boundary conditions 2 and 4 also. Boundary conditions 1 and 3 were chosen for penetrating the compacted soil samples.

Two cone penetration tests were performed in the same soil sample, but at different entry locations, as depicted in Figure 4.31. The standard penetration rate of 2 cm/sec. was utilized in the penetration phase, and readings were taken and
stored in the penetration phase file for each second of penetration.

4.5.2.3.1. Boundary Condition 3

After the soil sample was consolidated at 207 kPa, it was penetrated under boundary condition 3. Figures 4.37 and 4.38 display test results of a CPT performed at entry location 1.

Figure 4.37 introduces cone and friction sleeve resistance, and friction ratio. The sudden decrease in cone resistance showed at some penetration depths in this figure can be attributed to the layered character of the compacted soil sample, and to limitations in the chucking system used. The chucking system has a stroke of only 15 cm (6 in.), requiring at least five strokes for a complete penetration of the soil sample, which is 78.9 cm high. During penetration, the chucking system applies a grabbing force to the cone push rod in order to advance the cone into the soil sample. In the penetration position, at each stroke, the chucking sleeve containing the grabbing jaws should be able to move freely from the bottom of the push rod to the next upper grabbing position. However, it appears that a grabbing occurs at the beginning of the up movement, somewhat reducing the mobilized cone resistance at that point. This problem can be solved by a chucking system capable of penetrating
the soil sample in one single unique stroke. Such a system has been already implemented and added to the LSU/CALCHAS. However, present limitations on its hydraulic pump capacity is such that it is not possible to keep the standard penetration rate of 2 cm/sec. Also, the soil sample was compacted in five layers 15.78 cm thick. It could be expected that the resulting layered system would be responsible for an irregular evolution of the mobilized cone penetration resistance with depth. This expectation came to be true when considering cone resistance; however, as depicted in Figure 4.37, the local side friction resistance presented a regular development with depth, and the friction ratio resulted in an almost constant value. After 52 cm, the effect of the rigid bottom boundary generated by the sample bottom plate appears to be dominant in the cone penetration resistance. Cone penetration resistance and friction ratio came out as expected ($q_c = 130 \text{ kg/cm}^2$, $f_s = .6 \text{ kg/cm}^2$, FR = .75%), considering that the soil mixture is predominantly a compacted fine sand.

Figure 4.38 shows the progress of the vertical and horizontal stresses with penetration depth, for boundary condition 3. From this figure it is evident that a condition of constant vertical stress during penetration is attained. It can be also acceptable the condition of zero lateral strain, taking into account the similar development of the horizontal stresses in the inner and outer sample cells with penetration depth. However, it was reported the small difference of 2 kPa between the values of the horizontal stress on the sample inner and outer cell lines, at the end of penetration. An explanation could reside in the fact that the horizontal
stress readings from the sample inner and outer cell lines are taken only milliseconds apart, not allowing for a complete response of the compensation pressure system adopted in the LSU/CALCHAS. Therefore, there is not enough time for fully mobilizing the system response in order to keep both lines truly at the same pressure. The difference of 2 kPa can be considered acceptable for testing purposes.

4.5.2.3.2. Boundary Condition 1

After performing penetration of the soil sample under boundary condition 3 at position 1, the penetration set up was moved to position 2 on the top plate, and a new test was executed under boundary condition 1 in the same soil sample. Figures 4.39 and 4.40 display the test results.

Data from Figure 4.39 confirm the grabbing system problem referred in the previous cone penetration test. It is also evident that the influence of the layered system on cone penetration resistance is more accentuated on cone than on local side friction resistance. This fact could stress the importance of cone resistance for the development of correlations between cone penetration resistance and properties such as CBR, dry density and water content.

Figure 4.40 shows the progress of the vertical and horizontal stresses with penetration depth. An analysis of the test results shows that the hypothesized
condition of constant vertical and horizontal stresses can be considered acceptable, since both vertical and horizontal show clearly a constant development with penetration depth.
4.5. Tables and Figures
Table 4.1. Compacted Soil Sample Water Content (three determinations for each soil layer)

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<th>LAYER (Bottom to Top)</th>
<th>WATER CONTENT (%)</th>
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SAMPLE AVERAGE WATER CONTENT: 10.46%
Figure 4.1. Cross-Section of the LSU/CALCHAS
Figure 4.2. Details of the LSU/CALCHAS
Figure 4.3. Schematic of the LSU/CALCHAS Control Panel

SCHEMATIC LAYOUT OF LSU'S CALIBRATION CHAMBER SYSTEM

A: 3-WAY BALL VALVE
B: ON-OFF BALL VALVE
C: QUICK-CONNECTOR
D: FAIRCHILD BACK PRESSURE REGULATOR (BP1,2: 0-150 PSI)
E: FAIRCHILD E/P TRANSDUCER (F1,2: 0-30 PSI; F2: 0-120 PSI)
F: MARSH PROCESS PRESSURE GAUGE (G1,2,3,4,5: 0-100 PSI)
G: SENSYM TRANSDUCER (S1,2,4: 0-30 PSI; S3,5: 0-100 PSI)
H: BLEEDER-PLUG VALVE
Figure 4.4. Details of the LSU/CALCHAS Control Panel
Figure 4.5. Details of the Laboratory MQSC and Its Calibration Set Up
Figure 4.6. Details of Laboratory Calibration of MQSC
Figure 4.7. Details of the Hydraulics, Push Jack and Grabbing System
Figure 4.8. Details of the New Hydraulic System (79 cm stroke)
Figure 4.9. Cross-Section of the Large Dimension Automatic Tamper
Figure 4.10. Details of the LSU/CALCHAS Automatic Tamper and Compaction Mold

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Figure 4.11. Details of the LSU/CALCHAS Depth Measurement System
Figure 4.12. Details of the LSU/CALCHAS Auxiliary Systems

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Figure 4.13. Details of the Soil Mixing and Compaction Processes
Figure 4.14. Details of the Soil Sample After Compaction, Before and After Trimming
Figure 4.15. Details of Sample Movement from Compactor to Top of Chamber Piston Cell

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Figure 4.16. Details of Preparation of Calibration Chamber for the Penetration Phase

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Figure 4.17. Control Panel for the Consolidation Phase

CONSORTIAL PHASE: \( K_0 \) CONSOLIDATION

SCHEMATIC LAYOUT OF LSU'S CALIBRATION CHAMBER SYSTEM
4.18. Computer Screen for $K_o$ Consolidation
Figure 4.19. Control Panel for Penetration Phase: Boundary condition 1

CONE PENETROMETER

CHUCKING SYSTEM

JACK

PORE PRESSURE MEASUREMENT

SAMPLE CELL

WATER 18 GALLONS

PISTON CELL

SCHEMATIC LAYOUT OF LSU'S CALIBRATION CHAMBER SYSTEM

A: 3-WAY BALL VALVE
B: ON-OFF BALL VALVE
C: QUICK-CONNECTOR
D: FAIRCHILD BACK PRESSURE REGULATOR (8P1,2: 2-150 PSI)
E: FAIRCHILD E/P TRANSUCER (F1,3: 0-30 PSI; F2,4: 3-120 PSI)
F: MARSH PRESSURE GAUGE (G1,2,3,4,5: 0-100 PSI)
G: SENSYS TRANSDUCER (S1,2,4: 0-30 PSI; S3,5: 0-100 PSI)
H: BLEEDER-PLUG VALVE

BOUNDARY CONDITION 1:
CONSTANT VERTICAL STRESS
CONSTANT LATERAL STRESS
Figure 4.20. Computer Screen for Penetration Phase: Boundary condition 1
Figure 4.22. Computer Screen for Penetration Phase: Boundary condition 2
Figure 4.22. Control Panel for Penetration Phase: Boundary condition 3

PENETRATION PHASE: BOUNDARY CONDITION 3

SCHEMATIC LAYOUT OF LSU'S CALIBRATION CHAMBER SYSTEM

A: 3-WAY BALL VALVE
B: ON-OFF BALL VALVE
C: QUICK-CONNECTOR
D: FAIRCHILD BACK PRESSURE REGULATOR (BP/L: 2-150 PSI)
E: FAIRCHILD E/P TRANSDUCER (FL3: 6-30 PSI; F24: 3-120 PSI)
G: MARSH PROCESS PRESSURE GAUGE (G1: 0 - 100 PSI)
S: SENSYS TRANSDUCER (S1,2,4: 0-30 PSI; S3,5: 0-100 PSI)
V: BLEEDER-PLUG VALVE

BOUNDARY CONDITION 3:
- CONSTANT VERTICAL STRESS
- ZERO LATERAL STRAIN
Figure 4.24. Computer Screen for Penetration Phase: Boundary condition 3
Figure 4.25: Control Panel for Penetration Phase: Boundary condition 4

CONE PENETROMETER

CHUCKING SYSTEM

JACK

PORE PRESSURE MEASUREMENT

WATER
18 GALLONS

SAMPLE CELL

PISTON CELL

3-WAY BALL VALVE
ON-OFF BALL VALVE
QUICK-CONNECTOR
FAIRCHILD BACK PRESSURE REGULATOR (BP1, 2-150 PSI)
FAIRCHILD E/P TRANSDUCER (FL1, 0-30 PSI; FL2, 3-120 PSI)
MARSH PROCESS PRESSURE GAUGE (GL2, 3, 5, 0-100 PSI)
SENSYM TRANSDUCER (S1, 2, 4: 0-30 PSI; S3, 5: 0-100 PSI)
BLEEDER-PLUG VALVE

BOUNDARY CONDITION 4:
- Zero Vertical Strain
- Constant Lateral Stress

SCHEMATIC LAYOUT OF LSU'S CALIBRATION CHAMBER SYSTEM
Figure 4.26. Computer Screen for Penetration Phase: Boundary condition 4
Figure 4.27. Sieve Analysis: Kaolinite
Figure 4.28. Sieve Analysis: Fine sand
Figure 4.29. Sieve Analysis: Mixture of 80% of fine sand and 20% of kaolinite
Figure 4.30. Standard and Modified Proctor Compaction Test Results: Mixture of 80% of fine sand and 20% of kaolinite

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Figure 4.31. Position of Penetration Tests Performed in the Soil Sample
Figure 4.32. $K_0$ Consolidation Data: Vertical stress versus horizontal stress

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Figure 4.33. $K_o$ Consolidation Data: Sample vertical deformation versus time
Figure 4.34. $K_o$ Consolidation Data: Evolution of vertical and horizontal stresses with time
Figure 4.35. Sample Vertical Stress versus Sample Vertical Strain During K₀ Consolidation
Figure 4.36. Modulus of Compressibility ($M_0$): $K_o$ consolidation
LSU/CALCHAS
1.27 sqm Cone Penetration Test: Penetration Phase

Figure 4.37. LSU/CALCHAS CPT Profile: CPT at boundary condition 3

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Figure 4.38. Evolution of Sample Vertical and Horizontal Stresses During Penetration: Boundary condition 3
Figure 4.39. LSU/CALCHAS CPT Profile: CPT at boundary condition 1
Figure 4.40. Evolution of Sample Vertical and Horizontal Stresses During Penetration: Boundary condition 1
CHAPTER 5

SUMMARY OF RESULTS/ACCOMPLISHMENTS
AND RECOMMENDATION FOR FUTURE RESEARCH

5.1. Introduction

This chapter summarizes the results and accomplishments of this investigation. In addition, it suggests implementations which could improve the overall performance of the LSU/CALCHAS, and highlights areas for future research.
5.2. Summary of Results/Accomplishments of this Investigation

5.2.1. Field Testing Program

(1) The MQSC, 10 and 15 cm² cone penetrometers generated similar trends of cone resistance, local side friction resistance and friction ratio with penetration depth.

(2) Analysis of plots of cone penetration resistance versus penetration depth, considering each site individually and all sites pooled in a large file, suggested following observations:

- Existence of scale effects between the MQSC & 15 cm² penetrometer's local side friction resistance and the reference penetrometer's local side friction resistance;
- Existence of scale effects between the MQSC's cone resistance and the reference penetrometer's cone resistance;
- From an engineering point of view, there is no significant difference between the 15 cm² and the reference penetrometer's cone resistance and friction ratio.

(3) Inferences about difference between the MQSC & 15 cm² and the reference penetrometers data (matched large samples, \( \alpha \) risk controlled
219

.001 level when \( \mu - \mu_0 = 0 \) indicate differences in the cone resistance, local side friction resistance and friction ratio's population means, suggesting the existence of scale effects.

(4) Simple linear regression equations can be used to correct the MQSC & 15 cm²'s local side friction resistance and friction ratio, and the MQSC's cone resistance to the reference cone penetration resistance. Section 3.5 specifies regression equations for this purpose. The MQSC's local side friction resistance and friction ratio should be corrected via linear regression equations considering two ranges of cone resistance: (1) soils with cone resistance equal or smaller than 80 kg/cm², and (2) soils with cone resistance higher than 80 kg/cm².

(5) A multiplying factor of .85 could also be used to correct the MQSC's cone resistance in order to obtain the reference penetrometer's cone resistance.

(6) A division factor of .85 could also be used to correct the 15 cm² penetrometer's local side friction resistance in order to obtain the reference penetrometer's local side friction resistance.

(7) The higher capability of small size cones to capture more of the soil profile variability than large size cone penetrometers, as depicted in
Figure 3.14 is evident.

(8) The influence of different cone end area ratios on cone resistance was addressed in this investigation, considering sounding performed with the MQSC, the reference cone penetrometer and the 15 cm$^2$ Fugro dual-piezocone penetrometer at the Big River Industries site. The analysis was based on the assumption that cone penetrometers of different dimensions generate similar excess pore pressure distribution during soil penetration. The study permitted to conclude that different cone end area ratios do not affect significantly the measured cone resistance. However, this conclusion can be biased by the relative shallow depth sounded (9 m) and type of soil penetrated (soft clay) which generated relatively low excess pore pressure. Further verification of the validity of the preliminary hypothesis that cone penetrometers with different dimensions could generate similar excess pore pressure distribution during cone penetration would be essential. Future research encompassing different types of soils and piezocone dimensions is recommended to clarify this topic.

(9) The influence of wear of cone tip and friction sleeve on cone resistance was also addressed in this research. MQSC tests were performed at the Iowa (natural ground) site using a brand new cone tip and friction sleeve and a 1000 m penetration depth cone tip and friction sleeve.
Examination of the test results indicated that the MQSC with the brand new cone tip and friction sleeve gave lower friction ratio, cone and local side friction resistance than the same cone penetrometer with a tip and friction sleeve exposed to 1000 m of penetration. A cylindrical wear pattern of the cone friction sleeve was detected in opposition to a previous conical wear pattern reported by Jekel (1988). Considering the contradictory conclusion about the influence of wear of cone tip and friction sleeve by Jekel, and taking into account limitations on the number of sounding and field sites investigated in this research, it would be advisable to study this topic further.

5.2.2. Laboratory Testing Program

(1) The large dimension automatic tamper designed, fabricated and implemented in this research for sample preparation proved to be a valuable piece of equipment for reproducing the standard Proctor compaction effort. Considering the satisfactory performance of the equation depicted in Section 4.4.1. to represent the work done in operating the rammer of the automatic tamper at the standard Proctor compaction effort, it could be anticipated that this equation would also give satisfactory results for other compactive efforts.
(2) The depth measurement system developed for the laboratory cone penetration phase of this research proved to be reliable for measuring precise depth of cone penetration.

(3) The laboratory sample consolidation phase of this study resulted in a well defined $K_o$ line, as expected. A $K_o$ of .25 can be associated to the compacted/consolidated sand-kaolinite mixture.

(4) The hydraulics and chucking system used in this research showed an overall satisfactory performance. However, when in the penetration mode, the grabbing system seems to introduce some grabbing force in the up movement to the next cone rod grabbing sequence. This grabbing force somewhat diminishes the mobilized cone resistance, and generates a sudden drop in cone resistance when starting the new penetration step. Therefore, in future penetration tests at the LSU/CALCHAS, it would be advisable to use the new hydraulics and chucking system developed in this research for penetrating the soil sample in one stroke.

(5) The LSU/CALCHAS cone penetration resistance profile confirmed the characteristic profile obtained in previous calibration chamber worldwide tests. Three distinct sections can be visualized from the cone penetration resistance measurements. The upper and lower
portion of the profile clearly reflects the close proximity of the top and bottom plates, respectively; the middle section, except for the influence of the grabbing system, shows an almost constant cone penetration resistance.

(6) From the consolidation and penetration phase tests performed in this research, it was possible to conclude that the compensation pressure system used between the sample inner cell and sample outer cell was accurate enough for testing purpose.

(7) Cone penetration tests performed in the soil sample at boundary condition 1 illustrated the satisfactory performance of the Fairchild model 10 BP back pressure regulator on keeping the vertical and horizontal stresses constant during cone penetration.

5.3. Implementation of the LSU/CALCHAS

With the LSU/CALCHAS developed and made operational, it is hoped that future researchers will concentrate not only on using but also on implementing further modifications to the system. It is our understanding that the priority topics for implementation of the LSU/CALCHAS are, as follows:
(1) Design and construction of a reaction frame that could allow for the testing of the 10 and 15 cm² cone penetrometers in the LSU/CALCHAS;

(2) Introduction of other techniques for sample preparation, namely pluviation for sands and preconsolidation for clays;

(3) Addition of capability for saturating cohesionless soil samples, and pore pressure measurement during the consolidation and penetration phases along the sample cross-section.

5.4. Recommendation for Future Research

Based on the results of this investigation and on the suggested implementations of LSU/CALCHAS, following topics are recommended for future studies:

(1) Influence of different cone end area ratio on cone resistance: This theme was briefly addressed in the field testing program of this investigation. Conclusion about this question suggested that further study encompassing a field testing program with different types of soils and piezocone dimensions would be undertaken in order to clarify this
Influence of wear of cone tip and friction sleeve on cone resistance:
This topic was also briefly covered in the field testing program of this investigation. Conclusions reached on this subject suggest that a field testing program should be undertaken in order to improve our understanding on this theme. Different sandy soils and cone diameters should be considered in this program of study. Implementation of LSU/CALCHAS with a stronger reaction frame would provide an ideal tool for developing a similar test program under laboratory controlled conditions.

A laboratory testing program encompassing MQSC penetration of compacted soil samples in the LSU/CALCHAS would be implemented in order to develop empirical correlations between cone penetration resistance and soil parameters of interest to road and highway design and construction: maximum dry density, water content and California Bearing Ratio. Different soil types, consolidation pressures, compactive efforts and lift thickness should be considered in the study.

Introduction of capability of obtaining pluviated sand and preconsolidated clay samples in the LSU/CALCHAS would give the versatility required to study topics such as: effect of cementation on
cone resistance on sands, consolidation aspects of soft soils during cone penetration, scale effect on sands at high levels of overburden pressure.

(5) From a bearing capacity perspective, scale effects could become an important issue in the study of cone penetration resistance of layered systems. This is because enlarged cone tips would be expected to increase the number of layers influencing the soil resistance in the path of the penetrometer.
CHAPTER 6

CONCLUSIONS

A laboratory testing program is in progress at the Louisiana State University to examine the existence of scale effects among the cone penetrometers investigated in this research. A fully computerized calibration set up, the Louisiana State University Calibration Chamber System (LSU/CALCHAS), housing a sample .53 m in diameter and .79 m in height which permits the simulation of the $K_v$ consolidation and the four traditional penetration boundary conditions is already operational.

The field testing program was directed to the estimation of the scale effect between the MQSC and 15 cm² penetrometers and the reference penetrometer. The analysis of the field data permitted to draw the following conclusions:

(1) The MQSC, the reference 10 cm² and the 15 cm² cone penetrometers
generate similar trends of cone resistance, local side friction resistance and friction ratio with penetration depth;

(2) Inferences about difference between the MQSC, 15 cm² and the reference penetrometers' population means (matched large samples) support the existence of scale effect.

(3) Linear regression equations are furnished for proper cross-correlation. Scale effects on cone resistance do not seem to be dependent on soil type. On the contrary, considering the local side friction resistance and friction ratio of the MQSC and the reference penetrometer, the level of scale effect increases with increase on cone resistance. Therefore, it is recommended that the statistical evaluation of the local side friction resistance and friction ratio performance of the MQSC considers two ranges: (1) soils with $q_c$ equal to or smaller than 80 kg/cm², and (2) soils with $q_c$ higher than 80 kg/cm².

(4) Analyses of the CPT data suggest that a multiplication factor of 0.85 can be effectively used to correct the cone resistance of the MQSC to reference penetrometer scale. A division factor of 0.85 can also be effectively used to correct the local side friction resistance of the 15 cm² penetrometer to reference scale.
(5) From a practical engineering point of view, there is no significant difference between the cone resistance and friction ratio obtained by the 15 cm$^2$ and the reference penetrometers.
REFERENCES


MIT, Cambridge, MA, 368 pp.


EARTH TECHNOLOGY CORPORATION, "Cone Penetration Testing Services: Geotechnical Site investigation", Testing Service Group, Hunting Beach, CA.


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VAN VUUREN, D.J. (1969), "Rapid Determination of CBR with the Portable Dynamic Cone Penetrometer", The Rhodesian Engineer, 3 pp.


APPENDIX 1

FIELD TESTING PROGRAM: SOUNDING PLOTS
In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sp/cm) | Friction (Kg/Sp/cm) | Ratio(s)

Depth (inches)

Job Description: Field Testing Program 10 cm2 Friction Cone Penetrometer
Job Location/No.: Big River Industries/Baton Rouge/AIA/LSU01
Probe ID: 5035/V 538
Date/Time: 04-11-1990/11:50am
Elevation/GMT: Unknown/Unknown
Remarks: LSU Civil Engineering Geotechnical Research

Reveqit

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Seqm)  Friction (Kg/Seqm)  Ratio(R)

0  80  150  240  330  0  1  2  3  4  0  2  4  6  8  10

Depth (Meters)

Job Description: Field Testing Program: 11 and 2 Friction Cone Penetrometer
Job Location/No.: Big River Industries/Baton Rouge/LA/LSU002
Probe ID: FSDC/h 538
Remarks: LSU Civil Engineering Geotechnical Research

Data/Time: 04-16-1988/11:47am
Elevation/STD: Unknown/Unknown

Reveails
In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm)

Fricion (Kg/Sqcm)

Ratio(f)

Depth (Meters)

Job Description: Field Testing Program: 10 cm2 Friction Cone Penetrometer
Job Location/Nr.: Big River Industries/ Baton Rouge/LA/ LSUCO4
Probe ID: F30E/V-538
Date/Time: 04-15-1999/12:12am
Elevation/Time: Unknown/ Unknown
Remarks: LSU Civil Engineering Geotechnical Research

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sp/cm)  Friction (Kg/Sp/cm)  Ratio(d)

Depth (Meters)

Job Description: Field Testing Program 1D cm2 Friction Cone Penetrometer
Job Location/No: Big River Industries Station Range/PA  LSU005
Probe D: FS05/530
Data/Time: 04-19-1996/12/PM
Elevation/Off: Unknown/Unknown
Remarks: LSU Civil Engineering Geotechnical Research

Reveqits
In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Specm)  Friction (Kg/Specm)  Ratio
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0  0  0  0  0  0  0  0  0  0
0  0  0  0  0  0  0  0  0  0
0  0  0  0  0  0  0  0  0  0
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0  0  0  0  0  0  0  0  0  0
0  0  0  0  0  0  0  0  0  0
0  0  0  0  0  0  0  0  0  0
0  0  0  0  0  0  0  0  0  0

Job Description: Field Testing Program. 15 cm2 Friction Cone Penetrometer
Job Location/No.: Big Fiber Industries, Baton Rouge, LA / LSU0001
Probe D: F1525E/S 37
Date/Time: 04-10-2000/2:34pm
Elevation/QT: Unknown / Unknown
Remarks: LSU Civil Engineering Geotechnical Research Revegits

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm)  Friction (Kg/Sqcm)  Ratio (K)

Depth (Meters)

Job Description: Field Testing Program, 15 cm2 Friction Cone Penetrometer
Job Location/No.: Big River/Baton Rouge/AA/ LSU003
Probe ID: PT5204/ 307

Date/Time: 01/18/1980/ 2:58pm
Elevation/ALT: Unknown

Remain: LSU Civil Engineering Geotechnical Research

Revegits

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In-Situ Cone Penetration Test Result

Job Description: Field Testing Program 15 cm2 Friction Cone Penetrometer
Job Location/No.: Big Bear Industries/Station Range/LJ / LSU/DJ
Probe ID: F7.50NE/3 37
Remarks: LSU Civil Engineering Geotechnical Research

Reveqits
In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) Friction (Kg/Sqcm) Ratio

Depth (Meters)

Job Description: Field Testing Program 15 cm2 Over-penetrated
Job Location/No: Big River Industries Baton Rouge, LA / LSU 912
Probe D: E1501298-A-206
Remarks: Pre-punched 1m

Date/Time: 04-15-2003 / 1:34 pm
Elevation/GMT: Unknown / Unknown

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) | Friction (Kg/Sqcm) | Ratio \( \frac{2}{3} \)
--- | --- | ---
240 | 0 | 0
320 | 1 | 1
400 | 2 | 2
480 | 3 | 3
560 | 4 | 4
640 | 5 | 5
720 | 6 | 6
800 | 7 | 7
880 | 8 | 8
960 | 9 | 9
1040 | 10 | 10

Depth (Meters)

Job Description: Field Testing Program 15 cm² Friction Cone Penetrometer
Job Location/No.: Highland Road (Natural Ground)/Butler R/ LSU001
Probe ID: F7552F/V 337
Remarks:

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In-Situ Cone Penetration Test Result

![Graph showing cone penetration test results with depth in meters on the y-axis, and tip resistance and friction in kg/cm² on the x-axis.](image)

Job Description: Field Testing Program: 15 cm2 Friction Cone Penetrometer
Job Location/No.: Highland Road (Natural Ground)/Baton R/ LSN002
Probe ID: FZ1056/V 317
Remarks: LSU Civil Engineering Geotechnical Research

Date/Time: 11-13-1986/ 9:16am
Elevation: unknown/ unknown

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In-Situ Cone Penetration Test Result

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Job Description: Field Testing Program 15 cone Friction Cone Penetrometer
Job Location/No.: Highland Road (Natural Ground)/Datum R/ LS3003
Probe D: F7501E/v 301
Remarks: LSU Civil Engineering Geotechnical Research

Data/Time: 11-17-1993/10:30am
Elevation/Diff: unknown/ unknown

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In-Situ Cone Penetration Test Result

Job Description: Field Testing Program 16 cone Friction Cone Penetrometer
Job Location/No.: Highland Road (Natural Ground) Proctor 8/ LSU 004
Probe No.: F150E/N 317
Remarks:

Date/Time: 11-01-2009/10:41am
Elevation/DTM: unknown/ unknown

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In-Situ Cone Penetration Test Result

Job Description: Field Testing Program to Conduct Cone Penetrometer Test Location/No.: Highland Road (Natural Ground) Date: 11/51-8999 Test Site: LSO005
Probe No.: 77050.57 J317 Elevation/GMT: unknown/unknown
Remarks: LSU Civil Engineering Geotechnical Research

Depth (Meters)
Tip Resistance (Kg/Secm) Friction (Kg/Secm) Ratio(h)
In-Situ Cone Penetration Test Result

Job Description: Field Testing Program: ID 03 and Friction Cone Penetrometer
Job Location/Res.: Highland Road (Natural Ground)/ Baton R. / LSC008
Probe ID: F305/4-308
Remarks: Louis

LSU Civil Engineering Geotechnical Research

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In-Situ Cone Penetration Test Result

Depth (Meters)

Tip Resistance (Kg/Sqcm)

Friction (Kg/Sqcm)

Ratio(s)

Job Description: Field Testing Program 10 cm2 Friction Cone Penetrometer
Job Location/No.: Highland Road (Natural Ground)/Station II: CS0009
Probe ID: F5001/530
Remarks:

LSU Civil Engineering Geotechnical Research

Reveqits

Date/Time: 11-15-999/ 154pm
Elevation/Depth: Unknown/ Unknown

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) Friction (Kg/Sqcm) Ratio(4)

Depth (Meters)

Job Description: Field Testing Program 10 cm2 Friction Cone Penetrometer
Job Location/No.: Jame Natural Ground, USA/ LSU001
Probe D: 1 SOE/F 530
Date/Time: 12-13-2008/ 2:32pm
Elev/GMT: Unknown/Unknown
Remarks:

LSU Civil Engineering Geotechnical Research
Reveqits
In-Situ Cone Penetration Test Result

Job Description: Field Testing Program: 10 cm2 Friction Cone Penetrometer
Job Location/No: 1003 (Natural Ground)/LA1/LSU003
Probe ID: F50E/V 506
Date/Time: 12-13-2001/12.56m
Remarks:

LSU Civil Engineering Geotechnical Research

Revegits

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) Friction (Kg/Sqcm) Ratio(f)

Depth (Meters)

Job Description: Field Testing Program 10 cm2 Friction Cone Penetrometer
Job Location/Misc. Iowa (Natural Ground) IA/LSU072
Probe ID: F505/33
Remarks:

LSU Civil Engineering Geotechnical Research

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In-Situ Cone Penetration Test Result

Job Description: Field Testing Program - 15 and Friction Cone Penetrometer
Job Location/Inc: Ins (Natural Ground) A/A/ LS1004
Probe P: F7.50E/3 17
Date/Time: 12-15-2004/ 030pm
Elevening/Unit: Unknown/ Unknown
Remarks: LSU Civil Engineering Geotechnical Research

Reve wills
In-Situ Cone Penetration Test Result

Depth (meters)

Job Description: Field Testing Program 15 cm2 Friction Cone Penetrometer
Job Location/No.: Iowa (Natural Grade) A/ L30009
Probe D: F750E/4 317
Remarks: LSU Civil Engineering Geotechnical Research

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) Friction (Kg/Sqcm) Ratio

Job Description: Field Testing Program D and E Friction Cone Penetrometer
Job Location/No.: Highland Road (Entebbe Road) Button Range/ L50001
Probe D: FSOE/508

Remarks:

LSU Civil Engineering Geotechnical Research

Revegits

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) Friction (Kg/Sqcm) Ratio

Depth (Meters)

Job Description: Field Testing Program II and Friction Cone Penetrometer
Job Location: Highway 90 (Enchanted) / Baton Rouge / 150022
Probe D: 500E/530
Remarks: LSU Civil Engineering Geotechnical Research

Date/Time: 11-16-1989 / 12:51pm
Elevation: Unknown / Unknown

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm)  Friction (Kg/Sqcm)  Ratio(s)

0  20  40  60  80  0  1  2  3  4  0  2  4  6  8  0

Depth (Meters)

Job Number: Field Testing Program 10-ax2 Friction Cone Penetrometer
Job Location/No.: Highland Road (Enchanted) / Prosecution / L320003
Probe D: POREA/ 528
Remarks:

AC/ALT

LSU Civil Engineering Geotechnical Research

Reveqits
In-Situ Cone Penetration Test Result

Job Description: Field Testing Program: 10 cm2 Friction Cone Penetration
Job Location/No.: Highland Road (Enhanced)/Datum Height/ LSUD05
Probe D: 100E/V 538
Date/Time: 11-16-2003/ 12:30 pm
Elephant/087: Unknown/ Unknown
Remarks:

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Revegits

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) | Friction (Kg/Sqcm) | Ratio

1 2 3 4 5

Depth (Meters)

Job Description: Field Testing Program - 15 cm2 Friction Cone Penetrometer
Job Location/No: Highland Road [Embankment]/Selma Range/LSU 008
Probe ID: F7505E/40
Data/Time: 11-16-2009/3:26pm
Elevation/ST: Unknown/Unknown
Remarks:

LSU Civil Engineering Geotechnical Research

Revegits

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm)          Friction (Kg/Sqcm)          Ratio(x)

Depth (Meters)

Job Description: Field Testing Program 10 and Friction Cone Penetrometer
Job Location/No.: McElroy Swamp (Embankment)/10/A/LSU001
Probe ID: F506/5 530
Date/Time: 11-17-2009/2:56am
Elevation/DIT: Unknown/Unknown
Remarks:

LSU Civil Engineering Geotechnical Research

Revegits
In-Situ Cone Penetration Test Result

Job Description: Field Testing Program 10 cm2 Friction Cone Penetrometer
Job Location/No.:ьевнмжнртнрт/и-п1т-п1/ L50003
Probe No.: F500/4.33

LSD Civil Engineering Geotechnical Research
Revegits

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In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm)  Friction (Kg/Sqcm)  Ratio(\(\tau\))

Depth (Meters)

Job Description: Field Testing Program 15 with Friction Cone Penetrometer
Job Location/No.: McIver Swamp/10/UA/LSU009
Probe ID: FJ509/307
Remarks:

Date/Time: 11/17/2000/14:30
Elevation/CTI: Unknown/Unknown

LSU Civil Engineering Geotechnical Research
Reveqts
In-Situ Cone Penetration Test Result

Tip Resistance (Kg/Sqcm) Friction (Kg/Sqcm) Ratio(f)

Depth (Meters)

Job Description: Field Testing Program 15 cm2 Friction Cone Penetrometer
Job Location/No.: Meckay Swamp/15/AA/LSUSD
Probe D: F75053/31

Remarks: LSU Civil Engineering Geotechnical Research

Date/Time: 11-17-1995/1200am
Elevation/DRF: Unknown/Unknown

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RESULT OF CONE PENETRATION TEST

C1

JOB NUMBER: 0362
CONE NUMBER:
LOCATION: BIG RIVER INDUSTRIES
GROUND SURFACE ELEV.: 0.0 FEET
DEPTH TO GROUNDWATER: 0.0 FEET
DATE: JAN 90
RESULT OF CONE PENETRATION TEST

C3

JOB NUMBER: 0362
CONE NUMBER: 1
LOCATION: BIO RIVER INDUSTRIES
DATE: JAN 90

GROUND SURFACE ELEV.: 0.0 FEET
DEPTH TO GROUNDWATER: 0.0 FEET
RESULT OF CONE PENETRATION TEST

JOB NUMBER: 0362
CONE NUMBER: 21
LOCATION: HIGHLAND ROAD
GROUND SURFACE ELEV.: 0.0 FEET
DEPTH TO GROUNDWATER: 0.0 FEET
DATE: JAN 80
### Result of Cone Penetration Test

<table>
<thead>
<tr>
<th>Penetration Feet</th>
<th>Point Resistance</th>
<th>Friction Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>300</td>
</tr>
<tr>
<td></td>
<td>400</td>
<td>400</td>
</tr>
</tbody>
</table>

- **Sleeve Friction (f)**
- **KSF**
- **FRICTION RATIO f/KSF**

**Result of Cone Penetration Test**

**Job Number:** 0362  
**Ground Surface Elevation:** 0.0 Feet  
**Depth to Groundwater:** 0.0 Feet  
**Location:** Highland Road  
**Date:** Jan 90
RESULT OF CONE PENETRATION TEST

JOB NUMBER: 0302
CONCRETE NUMBER: CONE
LOCATION: HIGHLAND ROAD
GROUND SURFACE ELEV.: 0.0 FEET
DEPTH TO GROUND WATER: 0.0 FEET
DATE: JAN 80
RESULT OF CONE PENETRATION TEST

J O B N U M B E R : 0 3 6 2
C O N E N U M B E R : 1 0
L O C A T I O N : I D W A

G R O U N D S U R F A C E E L E V . : 0 . 0 F E E T
D E P T H T O G R O U N D W A T E R : 0 . 0 F E E T
D A T E : J A N 9 0
RESULT OF Cone Penetration Test

- **Job Number**: 0362
- **Cone Number**: IDWA
- **Ground Surface Elevation**: 0.0 feet
- **Depth to Groundwater**: 0.0 feet
- **Location**: IDWA
- **Date**: JAN 90
RESULT OF CONE PENETRATION TEST

JOB NUMBER: 0362
CONE NUMBER: A1
LOCATION: 110/HIGHLAND ROAD

GROUND SURFACE ELEV.: 0.0 FEET
DEPTH TO GROUNDWATER: 0.0 FEET
DATE: JAN 80
RESULT OF CONE PENETRATION TEST

POINT RESISTANCE (PSF)

PENETRATION (FEET)

SLEEVE FRICTION (PSF)

Friction Ratio (f/s %)

JOC NUMBER: 0362

CONE NUMBER: 110

LOCATION: HIGHLAND ROAD

GROUND SURFACE ELEV.: 0.0 FEET

DEPTH TO GROUNDWATER: 0.0 FEET

DATE: JAN 90

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RESULT OF CONE PENETRATION TEST

JOB NUMBER: 0362
CONE NUMBER: A3
LOCATION: 110/HIGHLAND ROAD
DATE: JAN 90

GROUND SURFACE ELEV.: 0.0 FEET
DEPTH TO GROUNDWATER: 0.0 FEET
RESULT OF CONE PENETRATION TEST 02

JOB NUMBER: 0362  GROUND SURFACE ELEV.: 0.0 FEET
CONE NUMBER:  HAWLE ROY SWAMP  DEPTH TO GROUNDWATER: 0.0 FEET
LOCATION:    DATE: JAN 90
APPENDIX 2

FIELD TESTING PROGRAM: CALIBRATION DATA

307
### Cone # 0010

<table>
<thead>
<tr>
<th>Load (lbs)</th>
<th>Counts (Tip Sleeve)</th>
<th>Factor (Tip Sleeve)</th>
<th>Count Adjust (Tip Sleeve)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>31 -15</td>
<td>0.6300 0.4582</td>
<td>0 0</td>
</tr>
<tr>
<td>5.04</td>
<td>39 -4</td>
<td>0.5906 0.4564</td>
<td>8 11</td>
</tr>
<tr>
<td>10.04</td>
<td>48 7</td>
<td>0.5553 0.4418</td>
<td>17 22</td>
</tr>
<tr>
<td>15.02</td>
<td>58 19</td>
<td>0.5422 0.4458</td>
<td>37 45</td>
</tr>
<tr>
<td>20.06</td>
<td>68 30</td>
<td>0.5216 0.4355</td>
<td>86 103</td>
</tr>
<tr>
<td>44.86</td>
<td>117 88</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Average: 0.5681 0.4475

**Constant:** 0.5303 0.4384

**Initial:** 27 18

---

### Cone # 0010

![Graph showing change in count vs. load](image)

<table>
<thead>
<tr>
<th>Load (lbs)</th>
<th>Change in Count</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>30</td>
<td>30</td>
</tr>
<tr>
<td>40</td>
<td>40</td>
</tr>
</tbody>
</table>

**Graph: Loading Capacity Analysis**

**Legend:**
- Square: Tip
- Circle: Load, lbs
- Triangle: Sleeve

---

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### Cone # 0011

**10/19/89**

<table>
<thead>
<tr>
<th>LOAD (lbs)</th>
<th>COUNTS TIP SLEEVE</th>
<th>FACTOR TIP SLEEVE</th>
<th>COUNT ADJUST TIP SLEEVE</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>33 47</td>
<td></td>
<td>0 0</td>
</tr>
<tr>
<td>5.04</td>
<td>23 36</td>
<td>-0.5040 -0.4582</td>
<td>-10 -11</td>
</tr>
<tr>
<td>10.04</td>
<td>14 25</td>
<td>-0.5284 -0.4564</td>
<td>-19 -22</td>
</tr>
<tr>
<td>15.02</td>
<td>3 13</td>
<td>-0.5007 -0.4418</td>
<td>-30 -34</td>
</tr>
<tr>
<td>20.06</td>
<td>-7 1</td>
<td>-0.5015 -0.4361</td>
<td>-40 -46</td>
</tr>
<tr>
<td>44.86</td>
<td>-52 -53</td>
<td>-0.5278 -0.4486</td>
<td>-85 -100</td>
</tr>
</tbody>
</table>

**Average:** -0.5125 -0.4482

**Constant:** -0.5213 -0.4465

**Initial:** 36 51

---

**Cone # 0011**

**10/19/89**

![Graph of Cone # 0011 showing change in count against load, tip, and sleeve.](image)
**CALIBRATION DATA CONE 207**

Type F7.50E2W/V

**Calibration by norm**

<table>
<thead>
<tr>
<th>Base area</th>
<th>Zero</th>
<th>Cal. factor</th>
<th>Load limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>15.0 cm²</td>
<td>-13.0 mV</td>
<td>806</td>
<td>150 kN</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Range</th>
<th>Force</th>
<th>FSO</th>
<th>Cal. factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75 kN</td>
<td>806.0 mV</td>
<td>806</td>
</tr>
<tr>
<td>2</td>
<td>0 kN</td>
<td>0.0 mV</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0 kN</td>
<td>0.0 mV</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>0 kN</td>
<td>0.0 mV</td>
<td>0</td>
</tr>
</tbody>
</table>

**Sleeve loadcell (or C+K)**

<table>
<thead>
<tr>
<th>Sleeve area</th>
<th>Zero</th>
<th>Cal. factor</th>
<th>Load limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>200 cm²</td>
<td>53.5 mV</td>
<td>150 kN</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Range</th>
<th>Force</th>
<th>FSO</th>
<th>Cal. factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75 kN</td>
<td>809.0 mV</td>
<td>809</td>
</tr>
<tr>
<td>2</td>
<td>0 kN</td>
<td>0.0 mV</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>0 kN</td>
<td>0.0 mV</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>0 kN</td>
<td>0.0 mV</td>
<td>0</td>
</tr>
</tbody>
</table>

Pressure test at 2.5 MPa

<table>
<thead>
<tr>
<th>Zero</th>
<th>Cal. factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.0 mV</td>
<td>473</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Range</th>
<th>Max pressure</th>
<th>FSO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.5 MPa</td>
<td>473.0 mV</td>
</tr>
<tr>
<td>2</td>
<td>0.0 MPa</td>
<td>0.0 mV</td>
</tr>
<tr>
<td>3</td>
<td>0.0 MPa</td>
<td>0.0 mV</td>
</tr>
</tbody>
</table>

**Slope sensor**

<table>
<thead>
<tr>
<th>Number</th>
<th>Zero</th>
<th>Cal. factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>E50-11</td>
<td>43 mV</td>
<td>500</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Angle</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>20</th>
<th>25</th>
<th>30</th>
<th>35</th>
<th>degrees</th>
</tr>
</thead>
<tbody>
<tr>
<td>Output</td>
<td>12</td>
<td>100</td>
<td>215</td>
<td>350</td>
<td>560</td>
<td>880</td>
<td>115</td>
<td>mV</td>
</tr>
</tbody>
</table>

**Remarks**

CAL. FACTOR FRICTION ON 333

Calibrated by FOG

Approved by

Date 16-05-89
### Calibration Data

**Type**: E 5C KEL 1 V 538

#### Cone Load Cell

- **Base area**: 10 cm²
- **Load limit**: 100 kN
- **No load output**: -10.3 mV
- **Range 1**: 50 kN, FSO: 819 mV, Cal factor: 819, Rec. range: 4 V
- **Range 2**: 100 kN, FSO: 1638 mV, Cal factor: 819, Rec. range: 2 V
- **Range 3**: __________, FSO: __________ mV, Cal factor: __________, Rec. range: __________
- **Range 4**: __________, FSO: __________ mV, Cal factor: __________, Rec. range: __________

#### Cone + Sleeve Load Cell

- **Sleeve area**: 150 cm²
- **Load limit**: 100 kN
- **No load output**: -16.8 mV
- **Range 1**: 50 kN, FSO: 820 mV, Cal factor: 820, Rec. range: 4 V
- **Range 2**: 100 kN, FSO: 1640 mV, Cal factor: 820, Rec. range: 2 V
- **Range 3**: __________, FSO: __________ mV, Cal factor: __________, Rec. range: __________
- **Range 4**: __________, FSO: __________ mV, Cal factor: __________, Rec. range: __________

#### Pore Pressure Transducer

- **Type**: __________
- **Serial number**: __________
- **Output 0 bar**: __________ mV
- **Range 1**: __________ bar, FSO: __________ mV, Burst pressure: __________ bar
- **Range 2**: __________ bar, FSO: __________ mV, Cal factor: __________, Rec. range: __________
- **Range 3**: __________ bar, FSO: __________ mV, Cal factor: __________, Rec. range: __________

#### Slope Sensor

- **Serial number**: E 50 11
- **Output at vertical**: +10 mV
- **Range**: 30 deg, FSO: 505 mV, Cal factor: 505, Rec Range: 4 V
- **Output**: 0° 5° 10° 15° 20° 25° 30°
  - mV: 75 155 450 650 820 960

#### Remarks

- Cal for Elevation Channel on 335°.

- **Calibrated by**: HLD, Date: 15/1/88
- **Approved by**: HLD, Date: 18/1/88

---

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<table>
<thead>
<tr>
<th>PENETROMETER TIP DATA</th>
<th>TYPE NR.</th>
<th>538</th>
</tr>
</thead>
<tbody>
<tr>
<td>Output at zero load</td>
<td>10.8 mV</td>
<td></td>
</tr>
<tr>
<td>Output at max. load</td>
<td>810 mV</td>
<td></td>
</tr>
<tr>
<td>Input resistance</td>
<td>2 ohm</td>
<td></td>
</tr>
<tr>
<td>Output resistance</td>
<td>2 ohm</td>
<td></td>
</tr>
<tr>
<td>Bridge supply voltage</td>
<td>10 V</td>
<td></td>
</tr>
<tr>
<td>Max. bridge sup. volt.</td>
<td>15 V</td>
<td></td>
</tr>
<tr>
<td>Max. load</td>
<td>100 kN</td>
<td></td>
</tr>
<tr>
<td>Temperature coefficient</td>
<td>0.1 m/V/°C</td>
<td>0.3 m/V/°C</td>
</tr>
<tr>
<td>Effect of overpressure</td>
<td>0.72 m/V/bar</td>
<td>1.75 m/V/bar</td>
</tr>
<tr>
<td>Calibration uncertainty</td>
<td>&lt;0.5 %</td>
<td>&lt;0.5 %</td>
</tr>
</tbody>
</table>

Cone load effect on friction: 0.8 mN/500 kN
Cable length: 4 m
Cable tension: 50 kN
Cone base area: 10 cm²
Slope error number: 0.5
Temperature range: -10°C to 40°C
Checking date: 19/0/38
Name: E. H. Van H. Hannema

General remarks: OPEN SENSE LAY WITH SLOW SENSOR
### CALIBRATION DATA

<table>
<thead>
<tr>
<th>Type</th>
<th>F 7 5 C K E / V 3 1 7</th>
</tr>
</thead>
</table>

#### CONE LOADCELL

- **Base area**: 15 cm²
- **Load limit**: 150 kN
- **No load output**: 0.0 mV
- **Range 1**: 75 kN, FSO: 738 mV, Cal factor: 738, Rec. range: 4 V
- **Range 2**: 150 kN, FSO: 1556 mV, Cal factor: 738, Rec. range: 2 V
- **Range 3**: .... kN, FSO: .... mV, Cal factor: ...., Rec. range: ....
- **Range 4**: .... kN, FSO: .... mV, Cal factor: ...., Rec. range: ....

#### CONE + SLEEVE LOADCELL

- **Sleeve area**: 200 cm²
- **Load limit**: 150 kN
- **No load output**: 42.0 mV
- **Range 1**: 75 kN, FSO: 738 mV, Cal factor: 738, Rec. range: 4 V
- **Range 2**: 150 kN, FSO: 1556 mV, Cal factor: 738, Rec. range: 2 V
- **Range 3**: .... kN, FSO: .... mV, Cal factor: ...., Rec. range: ....
- **Range 4**: .... kN, FSO: .... mV, Cal factor: ...., Rec. range: ....

#### PORE PRESSURE TRANSDUCER

<table>
<thead>
<tr>
<th>Type</th>
<th>Serial number</th>
<th>Output at 0 bar</th>
<th>mV</th>
</tr>
</thead>
</table>
- **Range 1**: .... bar, FSO: .... mV, Burst pressure: .... bar
- **Range 2**: .... bar, FSO: .... mV, Cal factor: .... Rec. range: ....
- **Range 3**: .... bar, FSO: .... mV, Cal factor: .... Rec. range: ....

#### SLOPE SENSOR

<table>
<thead>
<tr>
<th>Serial number</th>
<th>F 5 0 - 1 1</th>
</tr>
</thead>
</table>
- **Output at vertical**: 420 mV
- **Range**: 30 degr.
- **FSO**: .... mV, Cal factor: ...., Rec Range: ....
- **Output at 0°, 5°, 10°, 15°, 20°, 25°, 30°**:
  - 0°: 90, 275, 480, 655, 815, 980 mV

#### REMARKS

- **CAL FACTOR TRACTION CHANNEL ON 3B3**
- **Calibrated by**: [Name]
- **Date**: 22/3/88
- **Approved by**: [Name]
- **Date**: **
<table>
<thead>
<tr>
<th>PENETROMETER TIP DATA</th>
<th>TYPE NR.: T 7.5 CAE/V</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VERKOECHT ANN P 150</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Output at zero load</td>
<td>0 mV</td>
</tr>
<tr>
<td>Output at max. load</td>
<td>758 mV</td>
</tr>
<tr>
<td>Input resistance</td>
<td>= 150 mH</td>
</tr>
<tr>
<td>Output resistance</td>
<td>= 150 mH</td>
</tr>
<tr>
<td>Bridge supply voltage</td>
<td>10 volt</td>
</tr>
<tr>
<td>Max. bridge sup. volt.</td>
<td>15 volt</td>
</tr>
<tr>
<td>Max. load</td>
<td>150 kN</td>
</tr>
<tr>
<td>Temperature coefficient</td>
<td>0.02 µW/°C</td>
</tr>
<tr>
<td>Effect of water pressure</td>
<td>0.12 µW/°C</td>
</tr>
<tr>
<td>Calibration accuracy</td>
<td>≤ 0.5 %</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Cone load effect on friction</td>
<td>0.0 %</td>
</tr>
<tr>
<td>Water pressure test</td>
<td>25 bar</td>
</tr>
<tr>
<td>Core loss area</td>
<td>15 cm²</td>
</tr>
<tr>
<td>Blade sensor number</td>
<td>250, 11</td>
</tr>
<tr>
<td>Compensation box</td>
<td>No</td>
</tr>
<tr>
<td>Drawing reference</td>
<td>E 82</td>
</tr>
<tr>
<td>General remarks</td>
<td>Cone with filter and blade sensor</td>
</tr>
</tbody>
</table>
### CALIBRATION DATA

**CONE LOADCELL**

- **Base area:** 45 cm²
- **Load limit:** 150 kN
- **No load output:** +15.0 mV

<table>
<thead>
<tr>
<th>Range</th>
<th>Load Limit</th>
<th>FSO</th>
<th>mV</th>
<th>Cal Factor</th>
<th>Rec. Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75 kN</td>
<td>765</td>
<td>mV</td>
<td></td>
<td>4 V</td>
</tr>
<tr>
<td>2</td>
<td>150 kN</td>
<td>1530</td>
<td>mV</td>
<td></td>
<td>2 V</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**CONE + SLEEVE LOADCELL**

- **Sleeve area:** 200 cm²
- **Load limit:** 150 kN
- **No load output:** +11.2 mV

<table>
<thead>
<tr>
<th>Range</th>
<th>Load Limit</th>
<th>FSO</th>
<th>mV</th>
<th>Cal Factor</th>
<th>Rec. Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>75 kN</td>
<td>773</td>
<td>mV</td>
<td></td>
<td>4 V</td>
</tr>
<tr>
<td>2</td>
<td>150 kN</td>
<td>1446</td>
<td>mV</td>
<td></td>
<td>2 V</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**PORE PRESSURE TRANSUDER**

- **Type:**
- **Serial number:**
- **Output @ bar:** mV

<table>
<thead>
<tr>
<th>Range</th>
<th>FSO</th>
<th>mV</th>
<th>Cal Factor</th>
<th>Rec. Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**SLOPE SENSOR**

- **Serial number:** E 50-11
- **Output at vertical:** 70 mV

<table>
<thead>
<tr>
<th>Range</th>
<th>FSO</th>
<th>mV</th>
<th>Cal Factor</th>
<th>Rec Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Output (°)</th>
<th>0°</th>
<th>6°</th>
<th>10°</th>
<th>15°</th>
<th>20°</th>
<th>25°</th>
<th>30°</th>
</tr>
</thead>
<tbody>
<tr>
<td>mV</td>
<td>0</td>
<td>65</td>
<td>255</td>
<td>465</td>
<td>665</td>
<td>645</td>
<td>975</td>
</tr>
</tbody>
</table>

**REMARKS**

- CAL FACTOR FRICTION CHANNEL: ON 3.53

Calibrated by: [Name] Date: 22/2/66
Approved by: [Name] Date: [Date]
### PENETROMETER TIP DATA

**Type Nr.: F76C6E/V 318**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Core load (mV)</th>
<th>Friction load (mV)</th>
<th>Verification Area P.I.R.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Output at zero load</td>
<td>215</td>
<td>21.2</td>
<td>UNI LOUISIANA</td>
</tr>
<tr>
<td>Output at max. load</td>
<td>765</td>
<td>77.3</td>
<td></td>
</tr>
<tr>
<td>Input resistance</td>
<td>20</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Output resistance</td>
<td>20</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Bridge supply voltage</td>
<td>10</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>Max. bridge sup. volt.</td>
<td>15</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>Max. load</td>
<td>150</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td>Temperature coefficient</td>
<td>0.72 ±/µV/°C</td>
<td>0.22 ±/µV/°C</td>
<td></td>
</tr>
<tr>
<td>Effect of waterpressure</td>
<td>0.015 ±mV/bar</td>
<td>0.016 ±mV/bar</td>
<td></td>
</tr>
<tr>
<td>Calibration accuracy</td>
<td>≤0.5%</td>
<td>≤0.5%</td>
<td></td>
</tr>
</tbody>
</table>

**Additional Information**

- Core load effect on friction: 0.2 mV 9% of cable length: G.E. mV
- Water pressure set: 0 barInsulation resistance: 50 ohm
- Core base area: 15 cm² Sleeve area: 200 cm²
- Slope sensor number: F50-11 Temperature range: -10 to 60 °C
- Compensation box: 0 Name: ERUDEZ MOLANDER
- Checking date: 12/3/98
- Drawing reference: F.92

**General Remarks:** CONE WITH ASPHIELER AND SLUFF SPHAERE
<table>
<thead>
<tr>
<th>CALIBRATION DATA</th>
<th>TYPE</th>
<th>F 15 CFE 2 W1-2 - 206</th>
</tr>
</thead>
</table>

### CONE LOADCELL
- **Base area:** 15 cm²
- **Load limit:** 150 kN
- **No load output:** -1.9 mV
- **Range 1:** 75 kN
  - **FSO:** 812 mV
  - **Cal factor:** 812
  - **Rec. range:** 1 V
- **Range 2:**
- **Range 3:**
- **Range 4:**

### CONE + SLEEVE LOADCELL
- **Sleeve area:** 200 cm²
- **Load limit:** 150 kN
- **No load output:** -0.3 mV
- **Range 1:** 75 kN
  - **FSO:** 808 mV
  - **Cal factor:** 808
  - **Rec. range:** 1 V
- **Range 2:**
- **Range 3:**
- **Range 4:**

### PORE PRESSURE TRANSODUCER **III** (SLEEVE)
- **Type:** 4045 A 50
- **Serial number:** 111 675
- **Output 0 bar:** 17 mV
- **Range:** 50 bar
  - **FSO:** mV
  - **Burst pressure:** 125 bar
- **Range 1:** 25 bar
  - **FSO:** 487 mV
  - **Cal factor:** 487
  - **Rec. range:** 1 V
- **Range 2:**
- **Range 3:**

### PORE PRESSURE TRANSODUCER **E** (FACE)
- **Type:** 4050 A 50
- **Serial number:** 111 676
- **Output 0 bar:** -17.6 mV
- **Range:** 50 bar
  - **FSO:** mV
  - **Burst pressure:** 125 bar
- **Range 1:** 25 bar
  - **FSO:** 489 mV
  - **Cal factor:** 489
  - **Rec. range:** 1 V
- **Range 2:**
- **Range 3:**

### SLOPE SENSOR
- **Serial number:** E 50 - 11
- **Output at:** vertical: 0 mV
- **Ranges:** 30 degrees
  - **FSO:** 546 mV
  - **Cal factor:** 830
- **Rec. range:**
- **Output:**
<table>
<thead>
<tr>
<th>0°</th>
<th>5°</th>
<th>10°</th>
<th>15°</th>
<th>20°</th>
<th>15°</th>
<th>30°</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>82</td>
<td>245</td>
<td>432</td>
<td>650</td>
<td>752</td>
<td>946</td>
</tr>
</tbody>
</table>

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## Reparatiekaart Conus Type

**Datum**: 01/07/87

### Conus Condities List voor Reparatie en Kalibratie

1. Isolatiewerstand minimaal $5 \times 10^6$ ohm: __________ ohm
2. Onderdelen krom
3. Onderdelen beschadigd of diameters te klein
4. Schaduwefect of eccentriciteit
5. Stekkerverbinding
6. Kleefmeelop % bij ton puntlast
7. Nulpunt conus **5.5 mV**; Bereik conus **776 mV**
8. Nulpunt kiel (of C+K) **364 mV**; Bereik kiel (of C+K) **816 mV**
9. Nulpunt drukopnemer; Bereik drukopnemer
10. Nulpunt hellingsmeter; Bereik hellingsmeter
11. Verbinding tussen kabel en conus, insnoering
12. Compensatie doors

### Bijzonderheden en oorzak defect/storing:

### Vervangen Onderdelen

<table>
<thead>
<tr>
<th>Aantal</th>
<th>Mantel</th>
<th>Reksnoekjes</th>
<th>Bedrading</th>
<th>Quaarding</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>O-ring</td>
</tr>
<tr>
<td></td>
<td>Punt</td>
<td></td>
<td></td>
<td>O-ring</td>
</tr>
<tr>
<td></td>
<td>Versterkerhuis</td>
<td>Aansluitkabel</td>
<td>Koppeling</td>
<td>O-ring</td>
</tr>
<tr>
<td></td>
<td>Aansluitstuk</td>
<td>Connector</td>
<td></td>
<td>O-ring</td>
</tr>
<tr>
<td></td>
<td>Aansluitstuk</td>
<td>Connector</td>
<td></td>
<td>O-ring</td>
</tr>
<tr>
<td></td>
<td>Hellingsmeter</td>
<td>Versterker</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Drukopnemerhouder</td>
<td>Drukopnemer</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Filtersteen</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Bijzonderheden:

### Conus Repareerd en Herijkt: Naam ________________ Datum ________________

<table>
<thead>
<tr>
<th>Nulpunt conus</th>
<th>Bereik conus</th>
<th>Nulpunt kiel (of C+K)</th>
<th>Bereik kiel (of C+K)</th>
<th>Nulpunt drukopnemer</th>
<th>Bereik drukopnemer</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>-11.9 mV</strong></td>
<td><strong>B12</strong></td>
<td><strong>-0.3 mV</strong></td>
<td><strong>B88</strong></td>
<td><strong>1.487 mV</strong></td>
<td><strong>485 mV</strong></td>
</tr>
</tbody>
</table>

### Eindcontrole

<table>
<thead>
<tr>
<th>Hellingmeters</th>
<th>Cal. fact. B30</th>
<th>Gradeen mV</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>0 mV</strong></td>
<td><strong>0</strong></td>
<td><strong>0</strong></td>
</tr>
</tbody>
</table>

**Datum**: ________________

**Naam**: ________________

### Bijzonderheden:

<table>
<thead>
<tr>
<th>Druktest conus</th>
<th>Bereik</th>
<th>Burst pres. bar</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>24.6 mV</strong></td>
<td><strong>10</strong></td>
<td><strong>245</strong></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Drukopnemer type</th>
<th>Bereik</th>
<th>Burst pres. bar</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>bar</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Druktest kiel (C+K)</th>
<th><strong>31.5 mV</strong></th>
</tr>
</thead>
</table>

**Datum**: ________________
APPENDIX 3

LABORATORY TESTING PROGRAM: CALIBRATION DATA
CALIBRATION OF 1.27 CM² CONE

CONIC PENETROMETER SLEEVE (LOADING CONE SLEEVE)

$R^2 = 0.9999$

Load (kg) = $0.90927 - 202.102 \times $ Voltage (volts)

VOLTAGE (VOLTS)

LOAD (KG)
CALIBRATION OF 1.27 CM² CONE PENETROMETER SLEEVE (LOADING CONE TIP)

\[
R_e = 5999 \times \frac{\text{Load (kg)}}{571508} - 0.1296 \times \text{Voltage (volts)}
\]
CALIBRATION OF 1.27 CM² Cone

Cone Penetrometer Sleeve

\[ P = 0.9999 \]
\[ \text{Load (kg)} = 0.9039 - 202.182 \times \text{Voltage (volts)} \]
\[ \text{Load (kg)} = 0.3715 - 212.98 \times \text{Voltage (volts)} \]

Voltage (Volts)

Load (kg)
CALIBRATION OF LVDT
(LVDT No. 01142)

\[ V_{\text{disp}} (\text{mm}) = 12.03273 + 1.85593 \times V_{\text{volt}} \]

\[ R^2 = 0.9999 \]

VOLTAGE (VOLTS)

VERTICAL DISPLACEMENT (MM)
CALIBRATION OF PRESSURE TRANSDUCER
CHAMBER OUTER CELL TRANSDUCER (0 – 30 PSI)

\[ R^2 = 0.9997 \]

\[ \text{PRESSURE (PSI)} = -6.51392 + 6.043262 \times \text{VOLTAGE (VOLTS)} \]
CALIBRATION OF PRESSURE TRANSDUCER
CHAMBER INNER CELL TRANSDUCER (0 – 30 PSI)

\[ R^2 = 0.9994 \]
\[ \text{PRESSURE (PSI)} = -5.2882 + 5.921839 \times \text{VOLTAGE (VOLTS)} \]
CALIBRATION OF PRESSURE TRANSDUCER
CHAMBER INNER CELL TRANSDUCER (0 - 100 PSI)

R² = .9999
PRESSURE (PSI) = -19.7333 + 19.97412 * VOLTAGE (VOLTS)
CALIBRATION OF PRESSURE TRANSDUCER
PISTON CELL TRANSDUCER (0 - 30 PSI)

\[ P = -6.44377 + 5.88886 \cdot V \]

\[ R^2 = 0.9993 \]

VOLTAGE (VOLTS) vs. PRESSURE (PSI) graph with data points and linear regression equation.
CALIBRATION OF PRESSURE TRANSDUCER

PISTON CELL TRANSDUCER (0 - 100 PSI)

\[ R^2 = 0.999 \]

\[ \text{Pressure (PSI)} = -29.3423 + 10.06399 \times \text{Voltage (Volts)} \]
CALIBRATION OF ELECTRO-PNEUMATIC TRANSDUCER
PISTON CELL TRANSDUCER (5.6 - 30 PSI)

\[ R^2 = 0.997 \]
\[ \text{PRESSURE (PSI)} = 5.39592 + 3.82226 \times \text{VOLTAGE (VOLTS)} \]
CALIBRATION OF ELECTRO-PNEUMATIC TRANSUDER

PISTON CELL TRANSUDER (3 - 120 PSI)

PRESSURE (PSI) = 2.444033 + 11.88/12 * VOLTAGE (VOLTS)

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CALIBRATION OF ELECTRO-PNEUMATIC TRANSUDER
INNER/OUTER CELL TRANSUDER (5.6 - 30 PSI)

\[ R_2 = 4.942 \]

PRESSURE (PSI) = \( 0.040778 \times \) VOLTAGE (VOLTS)

\( R_2 \) = 4.942

PRESSURE (PSI) = \( 0.040778 \times \) VOLTAGE (VOLTS)

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CALIBRATION OF ELECTRO-PNEUMATIC TRANSDUCER

INNER/OUTER CELL TRANSDUCER (3 - 120 PSI)

\[ R^2 = 0.9998 \]

\[ \text{PRESSURE (PSI)} = 3.272 \times 10^3 + 12.1803 \times \text{VOLTAGE (VOLTS)} \]

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APPENDIX 4

CALIBRATION CHAMBER TEST: EXPERIMENTAL PHASE
LSU/CALCHAS

1.27 Sqcm Cone Penetration Test: Ko Consolidation

Soil Sample: Mixture of 80% fine sand and 20% of kaolinite
Compaction Effort: Standard Proctor
Consolidation Stress (kPa): 68.9
Test Number: (50000)/Experimental Phase

Date/Time: 06-09-1990/11:15am
Sample Dry Density (g/cm3): 15.11
Sample Water Content (%): 112

LSU Civil Engineering Biotechnical Research Chamber
LSU/CALCHAS

1.27 Sqcm Cone Penetration Test: Penetration Phase

Cone Resistance (Kg/Sqcm) | Friction (Kg/Sqcm) | Ratio (s)
0  80  160  240  320 0  1  2  3  4  0  1  2  3  4  5

Soil Sample: Mixture of 80% of Fine Sand and 20% of Kaolinite
Compaction Effort: Standard Proctor
Boundary Condition: BC1
Test Number: LSU000/Experimental Phase

Date/Time: 06-09-1990/3:29pm
Sample Dry Density (Kg/m3): 18.14
Sample Water Content (c): 11.20

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Soil Sample: Mixture of 80% fine sand and 20% of kaolinite
Compaction Effort: Standard Proctor
Consolidation Stress (kPa): 68.9
Test Number: LSU000/Experimental Phase

Date/Time: 06-09-1980/16:41
Sample Dry Density (kN/m³): 19.14
Sample Water Content (θ): 11.2
APPENDIX 5

Computer Programs

(Please, see back pocket)
Dario Cardoso de Lima was born in Resplendor, Brazil on September 28, 1951. He completed his B.S. degree in Civil Engineering at the School of Civil Engineering of Sao Carlos, University of Sao Paulo (USP), Brazil in 1975. After his graduation he was affiliated with various Brazilian engineering corporations and had worked for two years of experience in the design and construction of roads. In 1977 he joined the Industrial Research Institute (IPAI) as a staff engineer, School of Civil Engineering of Sao Carlos, and also started his graduate studies in Geotechnical Engineering at the same institution. In 1978 he was awarded a scholarship from the IPAI to pursue a specialization course in Soil Mechanics at the New University of Lisbon, Lisbon, Portugal, where he was granted the degree of Specialist in Soil Mechanics in 1979. In 1980 Mr. Lima joined the Federal University of Vicosa as a faculty member in the Department of Civil Engineering. Currently he is an assistant professor at this Brazilian institution. He completed his M.S. program at the University of Sao Paulo in 1981. Mr. Lima commenced his graduate studies at the Louisiana State University in January 1986 and is presently a candidate for the degree of Doctor of Philosophy in Civil Engineering, specializing in Geotechnical Engineering.
DOCTORAL EXAMINATION AND DISSERTATION REPORT

Candidate: Dario Cardoso de LIMA

Major Field: Civil Engineering (GEOTECHNICAL)

Title of Dissertation: Development, Fabrication and Verification of the LSU In Situ Testing Calibration Chamber (LSU/CALCHAS)

Approved:

Mehmet T. Tumay
Major Professor and Chairman

Dean of the Graduate School

EXAMINING COMMITTEE:

Ara Arman

Ilan Juran

Gil Lee

Kwei Tang

Joseph N. Suhayda

Date of Examination:

July 13, 1990