DEVELOPMENT OF A SETTLING COLUMN AND ASSOCIATED PRIMARY CONSOLIDATION MONITORING SYSTEMS FOR USE IN THE GEOTECHNICAL CENTRIFUGE

INVESTIGATION OF GEOTECHNICAL-GEOPHYSICAL CORRELATIONS

SHAWN J. HURLEY

Development of a Settling Column and Associated Primary Consolidation Monitoring Systems for Use in the Geotechnical Centrifuge

Investigation of Geotechnical-Geophysical Correlations

by

Shawn J. Hurley, P.Eng.

A thesis submitted to the School of Graduate

Studies in partial fulfilment of the

requirements for the degree of

Master of Engineering

Faculty of Engineering and Applied Science

Memorial University of Newfoundland

May 1999

ABSTRACT

A research program to develop an instrumented settling column and obtain correlations between geophysical and geotechnical properties of soft soils as they undergo self-weight consolidation in the geotechnical centrifuge is described. The primary research objective was to develop the column and its measurement systems, conduct initial tests with speswhite kaolin clay and demonstrate that the data obtained is comparable to published results.

Soft soils are characterized by low shear strengths and low densities and it is virtually impossible to obtain representative samples for geotechnical testing. In order to study the geotechnical properties of soft soils, researchers have utilised settling column experiments and geophysical techniques to develop correlations between geotechnical and geophysical properties. However, 1 g column experiments are usually inefficient due to the amount of time required to conduct a single test (months to years) and low effective stresses which do not accurately represent field conditions.

The thesis focuses on two experiments during which in-flight measurements were made of compressional wave velocity, bulk density, and pore pressure. This work has demonstrated that the centrifuge can be used to enhance primary consolidation and significantly increase effective stress levels. The correlations between compressional wave velocity and bulk density for speswhite kaolin clay are in agreement with previous work. It is believed that this is the first documented case of a compressional wave being generated in the centrifuge through a consolidating soft soil.

ACKNOWLEDGEMENTS

I would like to thank my supervisors, Dr. Ryan Phillips and Dr. Ian McDermott, for their helpful guidance and advice and for the opportunity to work on this project. My colleagues at the C-CORE Centrifuge Centre also deserve a note of thanks. Mr. Tony King participated in a number of the centrifuge experiments and his help is greatly appreciated. Mr. Don Cameron assisted with package preparation and test conduct for all of the centrifuge experiments. Mr. Steven Smythe and Mr. Karl Tuff deserve a great deal of credit for preparation of the various electronic systems associated with the column.

I would also like to thank my parents for giving me the initial opportunity to pursue a post-secondary education.

Finally I would like to thank my wife, Michelle, for her endless patience, love and encouragement. Without her, this work would not have been possible.

CONTENTS

APPENDICES

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

B	Bulk modulus
C_w	Compressibility of fluid
C_s	Compressibility of solids
c_v	Coefficient of consolidation
d	Grain diameter
e	Void ratio
FF	Formation factor
g	Gravitational level
G	Shear modulus
G_s	Specific gravity
h	Height
H_{dr}	Length of drainage path
I_p	Plasticity index
k	Permeability
K_{onc}	Coefficient of lateral earth pressure during one-dimensional normal consolidation
p'	Mean effective stress
PPT	Pore pressure transducer
S	Salinity
T	Temperature
T_m	Model time
T_p	Prototype time
u	Excess pore water pressure
v_p	Compressional wave velocity
v_{pn}	Normalised compressional wave velocity
w_i	Initial water content
w_l	Liquid limit
w_p	Plastic limit
γ_s	Unit weight of soil solids
γ_f	Unit weight of pore fluid
η	Porosity
v	Velocity of soil grain
ρ	Density
ρ_b	Bulk density
ρ_s	Density of soil solids
ρ_f	Fluid density
ρ_w	Density of water
σ'	Effective stress
σ_{TOT}	Total stress

1.0 INTRODUCTION

This thesis describes a research program designed to develop an instrumented settling column which is used to develop correlations between geophysical and geotechnical properties of soft soils as they undergo self-weight consolidation in the geotechnical centrifuge. The primary research objective was to develop the column and associated measurement systems, conduct some initial tests with speswhite kaolin clay and demonstrate that the data obtained thus far is reasonably comparable to published results.

Geotechnical properties of soft soils have typically been studied using laboratory settling column experiments conducted under normal gravitational conditions. Even though these experiments provide valuable information, they are inefficient due to the large experimental times required (months to years) and the fact that the effective stresses developed due to self-weight consolidation are very small and do not closely replicate actual field conditions.

The centrifuge is used to enhance primary consolidation by increasing the self weight of the soil particles, which dramatically reduces the amount of time required for sedimentation, and significantly increasing the level of effective stress. The overall objective of this work is to improve the current state of knowledge of soft soil consolidation behaviour and to demonstrate the usefulness of the centrifuge for this type of research. This type of work has applications in the mining, waste disposal and dredging industries.

2.0 LITERATURE REVIEW

2.1 General

It is important to improve the current state of knowledge of sedimentation and consolidation of soft soils. Soft soils are characterised by their low densities and low strengths and it is virtually impossible to obtain a high quality sample of a soft soil for laboratory testing. With regards to industrial applications, it is important for geotechnical engineers to fully comprehend the unconsolidated behaviour of dredged materials in order to design efficient dredge spoil disposal sites (Lin and Lohnes, 1984). Thorough understanding of the settling and consolidation behaviour of soft soils, such as mine tailings, is necessary to design efficient disposal ponds for the oilsands industry for example. Another need for this research lies in the accurate determination of *in situ* parameters such as bulk density and voids ratio. The usual method of investigating marine soils is by taking cores or grab samples of the sediment using various types of sampling equipment. However, due to the low strengths of soft soils, it is very difficult and sometimes virtually impossible to collect representative samples. In any case, samples retrieved from the seabed are usually highly disturbed. A more useful approach has been to study soft soils in laboratory settling column experiments with the aim of developing correlations between those parameters that can be measured *in situ* to those that are required for engineering purposes.

2.2 Soft Soil Sedimentation & Consolidation

2.2.1 Sedimentation

In geotechnical engineering, Stokes Law' is commonly used to model the behaviour of soil grains falling through a column of water. Stokes law is given by:

$$\upsilon=\left(\frac{2}{9}\right)\times\frac{(\gamma_s-\gamma_f)}{\eta}\times\left(\frac{D}{2}\right)^2 \qquad [1]$$

where υ = velocity of fall of the sphere (soil grain), cm/s

γ_s = specific weight of the sphere, N/cm^3

γ_f = specific weight of the fluid, N/cm^3

η = absolute, or dynamic, viscosity of the fluid, g/(cm$\times$s)

D = diameter of the sphere, cm

Stokes' law describes the settling behaviour of a single spherical particle by equating the fluid drag force to the gravitational force on the particle at the terminal settling velocity. Settlement characteristics of soft cohesive soils are also affected by the medium through which sedimentation occurs. For example, when clay particles are deposited into seawater, ion exchange can occur and this can change the characteristics of the clay particles which, in turn, changes the settlement characteristics. Flocculation and aggregation of clay particles is also known to affect the settlement and consolidation of soft soils (Dyer, 1986). Clay particles are platelike, with a diameter less than 2 μm and a thickness which is approximately 1/5 th the diameter. Flocculation of clay particles results in clusters of clay particles plus enclosed water which are formed by chemical and electrostatic forces among clay particles and water. In a dispersed suspension, each clay particle settles at a velocity that is determined mainly by the grain size and this type of settling behaviour is approximated by Stokes' Law. Settling which occurs in a dispersed solution results in a segregation of particle sizes due to the different settling velocities. However in a flocculated suspension, an individual floc can be composed of a variety of particle sizes and as a result, large grains can settle at the same velocity as smaller grains. Flocculation and aggregation of particles therefore plays a significant role in the sedimentation and subsequent consolidation of a soil water suspension.

2.2.2 Soil Consolidation

A saturated clay soil is mainly composed of clay particles with a fluid, usually water, occupying the voids between the clay particles. The consolidation of a soil is defined by Been (1980) as water flowing out of the soil and the soil properties changing as a result of the pore water being expelled. Soil consolidation usually results from an external load being applied to the soil. For example when a new structure, such as a building, is placed on the foundation soil, effective stresses are increased within the soil matrix. At first, the total increase in load is carried by the soil pore water. As time progresses, the load is gradually transferred to the soil and the excess pore water pressure generated by the application of the external load is dissipated. As the excess pore water in the soil matrix is expelled, the soil layer will settle due to compression and a reorientation of the soil particles. The behaviour of the soil is governed by the effective stress:

$$\sigma' = \sigma_{TOT} - u \qquad [2]$$

where: σ' = effective stress

σ_{TOT} = total stress

u = pore water pressure

Terzaghi's (1925) classical spring analogy is commonly used to describe the process of soil consolidation, but does not consider the effect of self-weight which is important for soft soils and is limited to relatively small settlements (on the order of several centimetres).

The transitional stage from suspension to a soft soil is characterised by the development of a soil skeleton which is extremely compressible. As a result, the strains that are associated with this type of consolidation are very large for small applied stresses arising from the self weight of the material (Been and Sills, 1981).

The theory developed by Terzaghi (1925) cannot be used to analyse the consolidation of a soft soil which undergoes large strains. Terzaghian theory is based on certain simplifying assumptions which are not valid for large strain consolidation. For example, his theory assumes that the soil permeability and compressibility remain constant under a particular load increment (Gibson *et. al.*, 1967) and that an applied load increment will produce only small strains in the soil (Holtz and Kovacs, 1981). For the consolidation of soft soil the simplifying assumptions invoked by Terzaghi (1925) are invalid. Consolidation is brought about due to the self weight of the soil and the strains that result from soil loading can be extremely large.

2.2.3 Description of Self Weight Consolidation Theory For a Slurry

One theory used to describe the self weight consolidation of a soil slurry was developed from Gibson *et al.* (1967). In order to implement this theory, the following assumptions must be made:

(1) Compressibility of the soil particles and soil pore water is negligible.

(2) The soil is completely water saturated.

(3) The principle of effective stress is taken to be valid.

(4) Secondary compression is negligible.

(5) The flow of pore water through the soil is in accordance with Darcy's law.

(6) Behaviour is independent of strain/loading rates or intrinsic time effects.

(7) Unique compression-void ratio and permeability-void ratio relationships exist.

This theory was initially developed to explain the consolidation of a *thin* clay layer. As a result, this original theory developed by Gibson *et al.* (1967) did not take into consideration the effect of self weight of the soil. Lee and Sills (1981) modified the theory developed by Gibson *et al.* (1967) to account for the effect of *self-weight* on the consolidation characteristics of a settling dredged fill that was initially deposited with a uniform density and void ratio.

The equation which governs the void ratio behaviour for one dimensional consolidation was developed by Gibson *et al.* (1967) and is given as follows:

$$\frac{\delta e}{\delta t} = -\frac{\delta}{\delta z}\left[\frac{k}{\rho_f(1+e)}\frac{d\sigma'}{de}\frac{\delta e}{\delta z}\right] - (\rho_s - \rho_f)\frac{d}{de}\left(\frac{k}{\rho_f(1+e)}\right)\frac{\delta e}{\delta z} \qquad [3]$$

where z is the total solid thickness or material coordinate, σ' is the effective stress, k represents the permeability, ρ_s is the density of the soil particles and ρ_f is the density of the pore fluid (water). Mikasa (1963) proposed a similar theory for the self weight consolidation of a clay layer.

The permeability of the soil can be expected to change in accordance with Lee and Sills (1981):

$$\frac{k}{\rho_f} = k_o \times (1 + e) \qquad [4]$$

where k_o is the initial soil permeability. This equation has been experimentally verified by Been and Sills (1981).

Gibson *et al.* (1967) defined the coefficient of consolidation (C_f) to be,

$$C_f = \frac{k}{\rho_f (1+e)} \times \frac{d\sigma'}{de} \qquad [5]$$

If C_f is assumed to be constant, equation [3] is simplified to,

$$\frac{de}{dt} = C_f \times \frac{d^2 e}{dz^2} \qquad [6]$$

2.2.4 Stages of Soft Soil Formation

Been (1980) makes reference to three different stages of soft soil formation:

A. Suspension phase

B. Intermediate phase

C. Consolidation Phase - Settled soil

In the case of a soil-water suspension, effective stresses do not exist. The main difference between the suspension phase [A] and the consolidation phase [C] of soil formation lies in the existence of measurable effective stresses. A soft soil has formed once effective stress are generated. The different stages of soil formation were depicted by Been (1980) as follows:

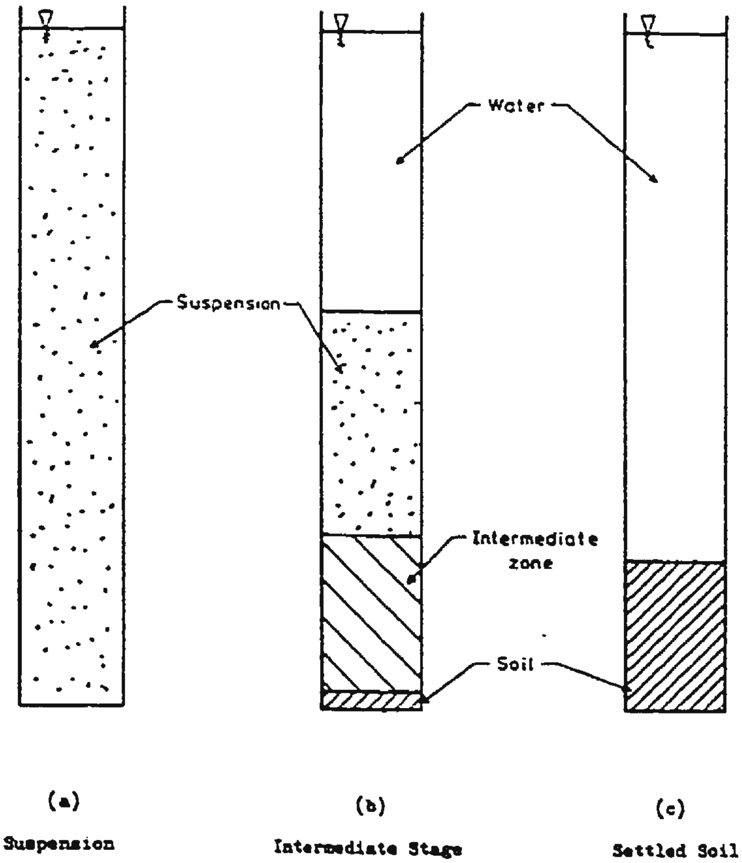


Figure 1 Stages of Soil Formation (After Been, 1980)

If a soil water suspension (Figure 1 (a)) is allowed to settle under gravity, it will eventually become a soil with measurable effective stresses. Shortly after settlement begins, flocs will start to form and settle. At the base of the settling soil column, the flocs are brought closer together. However, before a soil is truly formed, the soil particles must move closer together and flocs must break down (density must increase) and effective stresses must develop within the soil matrix. In order for this to occur certain chemical and electrostatic forces between particles and flocs which keep the particles apart while in suspension must be overcome. Been (1980) conjectured that an intermediate region existed where these forces between particles and flocs begin to break down and the soil grains are transformed from a random suspension to a more ordered soil matrix.

Yong(1984) studied the particle interaction of suspended solids in tailings discharge slurries and described a sedimentation/consolidation process similar to Been (1980) although his description utilised four distinct zones compared to Been's three. Yong's (1984) description could easily be simplified to include only three zones since the bottom two zones are characterised by the existence of measurable effective stresses.

Bloomquist and Townsend (1984) adopted a similar description of the sedimentation/consolidation behaviour of phosphatic clay. This three phase description of soft soil formation has also been utilised by Eckert *et al.* (1996) to study the sedimentation and consolidation properties of fine tails produced by the hot water extraction of oil from the Athabasca tar sands. McDermott (1992) and Elder (1985) also recognized the three stage system of soft soil formation in their sedimentation/consolidation studies of marine clays.

2.2.5 Critical Concentration / Critical Void Ratio

There is evidence in the literature to suggest that the sedimentation / consolidation behaviour of a sediment is strongly influenced by the initial concentration or initial voids ratio. If a sediment is deposited with an initial concentration below some critical value (or with a voids ratio higher than some critical value), the sediment will first undergo sedimentation and will eventually begin to consolidate once the sedimentation phase is complete. However, if the initial concentration is higher than the critical value (or if the voids ratio is lower than critical), the consolidation phase will begin immediately after slurry deposition (i.e. effective stress will develop immediately). In other words, only one phase, the consolidation phase, will exist.

The existence of this critical concentration has been documented by several researchers. Lin and Lohnes (1984) were interested in the sedimentation and consolidation properties of dredge spoil slurries. The silty sediment tested had the following properties:

grain size distribution, 37% clay - 63% silt

natural water content = 78.9%

in-situ unit weight = 8.64 kN/m^3

liquid limit, w_l = 59.4%

plasticity index = 24.7

Once the soil was mixed into a uniform slurry with a predetermined specific concentration, it was pumped into the settling column and air was pumped into the mixture from the bottom to prevent any settlement from occurring until the column had been completely filled. After the column was filled, the air was stopped and settling was started. The settlement was monitored very closely and when a sharp interface between the sediment laden water and the overlying water was visible, its height was recorded. After recording the height of the interface, the slurry height was recorded at regular intervals and interface height verses time curves were plotted. The researchers used this procedure to study several slurries that were mixed to different initial concentrations.

The tests that were conducted revealed the existence of a critical concentration for this test series. When the initial slurry concentration was higher than the critical concentration, the water sediment interface was observed to form immediately after starting the test. The critical concentration was reported to be dependant upon the nature of the material and the settling environment.

One potential problem with the research carried out by Lin and Lohnes (1984) was that they agitated the slurry with air to prevent premature settling. It has been shown (McDermott, 1992) that sediments containing a certain amount of entrapped air can have different consolidation properties than saturated sediments.

Elder (1985) also observed the existence of a critical void ratio for his tests on natural estuarine soils. If the initial slurry void ratio of this soil was less than 12 at the time of deposition, no suspension phase was observed and the soil immediately began to consolidate. The surface bulk density increased slowly with time. However, for an initial void ratio greater than 12, a suspension phase was observed and the surface void ratio was seen to initially increase. Gradually the soil began to consolidate and the void ratio decreased.

McDermott (1992) noted that the behaviour of his natural sediment in sea water was similar to the behaviour described by Elder (1985) in fresh water and he also noted that critical void ratio was a function of the soil type. For sediment slurries that were deposited above a critical void ratio, there existed a suspension phase during which the void ratio gradually increased with time. This was said to be due to the coarser soil particles settling out of suspension and falling towards the column base. After reaching a peak, the surface void ratio was observed to decrease rapidly until the suspension surface reached the consolidating layer underneath. The void ratio at which this type of behaviour was observed was reported to be between 14.5 to 15 for the type of natural sediment studied. For soil slurries that were deposited below a critical void ratio, there was no suspension phase and the surface void ratio simply decreased slowly with time. This observation led McDermott (1992) to conclude that consolidation behaviour of a rapidly deposited soil slurry is greatly influenced by its initial input density. Similar behaviour has also been observed by Sills and Thomas (1984) during tests on samples of an estuarine silty clay.

Michaels and Bolger (1962) investigated the settling rates and sediment volumes of flocculated kaolin suspensions as functions of kaolin concentration. However, they do not make strong conclusions with regards to the existence of a critical concentration for kaolin. They simply state that the observed settling rate was a function of the floc volume concentration.

The existence of a critical concentration (or void ratio) has added importance for studies of soft soil sedimentation / consolidation using the geotechnical centrifuge. Centrifuge theory is based on the assumption that the sediment behaves similarly in an accelerated gravitational field as it does under normal gravitational conditions. You and Znidarcic (1994) conducted centrifuge modelling experiments as well as 1 g tests at the University of Colorado to study the initial stage of consolidation of Speswhite kaolin clay at high initial void ratios. They note that for dilute clay suspensions (low densities and high initial void ratios) the soil that was created from self-weight sedimentation / consolidation experiments had a different structure and different geotechnical properties than the prototype. These authors demonstrate that the void ratio at the soil surface, after consolidation was completed (zero effective stress void ratio), depended on the initial sediment water content (or density / void ratio). This was a confirmation of Elder's (1985) results for natural sediments. They also demonstrate that the zero effective stress void ratio is gravity level dependent. For example, under 1 g conditions the zero effective stress void ratio was equal to the mixing void ratio (initial void ratio) up to values of about 8. However, at 40 g sedimentation took place with a mixing void ratio (initial void ratio) as low as 4 (i.e. water content = 152 %).

You and Znidarcic (1994) believe that the different soil properties from 1 g and centrifuge experiments was caused by the high initial water contents which led to sedimentation and particle segregation. It would have been beneficial to conduct a series of grain size analyses on samples from the 1 g and centrifuge experiments and compare the results. However, this work was not carried out.

Scott *et al.* (1986) also refers to the existence of a critical initial water content for large scale 1 g tests conducted on samples of oilsands tailings.

2.3 Soft Soil Laboratory Studies - Apparatus and Measurement Systems

2.3.1 Introduction

Settling columns have been developed and instrumented to study the self weight sedimentation and consolidation of soft soils. Basic instrumentation has included techniques to measure settlement and pore pressure. More sophisticated columns have also been equipped with systems to measure bulk density during consolidation as well as electronic pore pressure measurement systems.

2.3.2 Types of Settling Columns

Various types of settling columns have been used to study the sedimentation and consolidation behaviour of soft soils both under normal 1 g conditions and in the geotechnical centrifuge. These columns have ranged from small cylinders used in 0.6 m bench top centrifuges (McDermott and King (1998)) to 10 m high , 1 m diameter transparent columns used by Scott *et al.* (1986) to study sedimentation/consolidation properties of oilsands tailings.

The majority of laboratory 1 g experiments have been conducted using columns manufactured from plexiglass (Been and Sills (1981), Sills and Thomas (1984), Lin and Lohnes (1984), Elder (1985), Scott *et al.* (1986), McDermott (1992)). This type of material is transparent and settlement can be easily measured using a scale attached to the outside of the column. Liu *et al.* (1994) described a series of standpipes used to conduct tests on samples of oilsands tailings. It is assumed that these standpipes were also manufactured from plexiglass although the shorter standpipes described by the authors are assumed to be similar to the glass jars used for hydrometer grain size analyses. Typical maximum column heights and diameters reported in the literature are 2 m and 200 mm respectively. Many of the 1 g column experiments were conducted with single drainage from the top of the sample only. In other words, most 1 g columns were manufactured with an impervious base so that drainage could not take place from the bottom of the column. This contributed to the significant amount of time required to perform a 1 g column experiment, which has been reported to range from days to even years. Figure 2 is a diagram of McDermott's (1992) instrumented settling column.

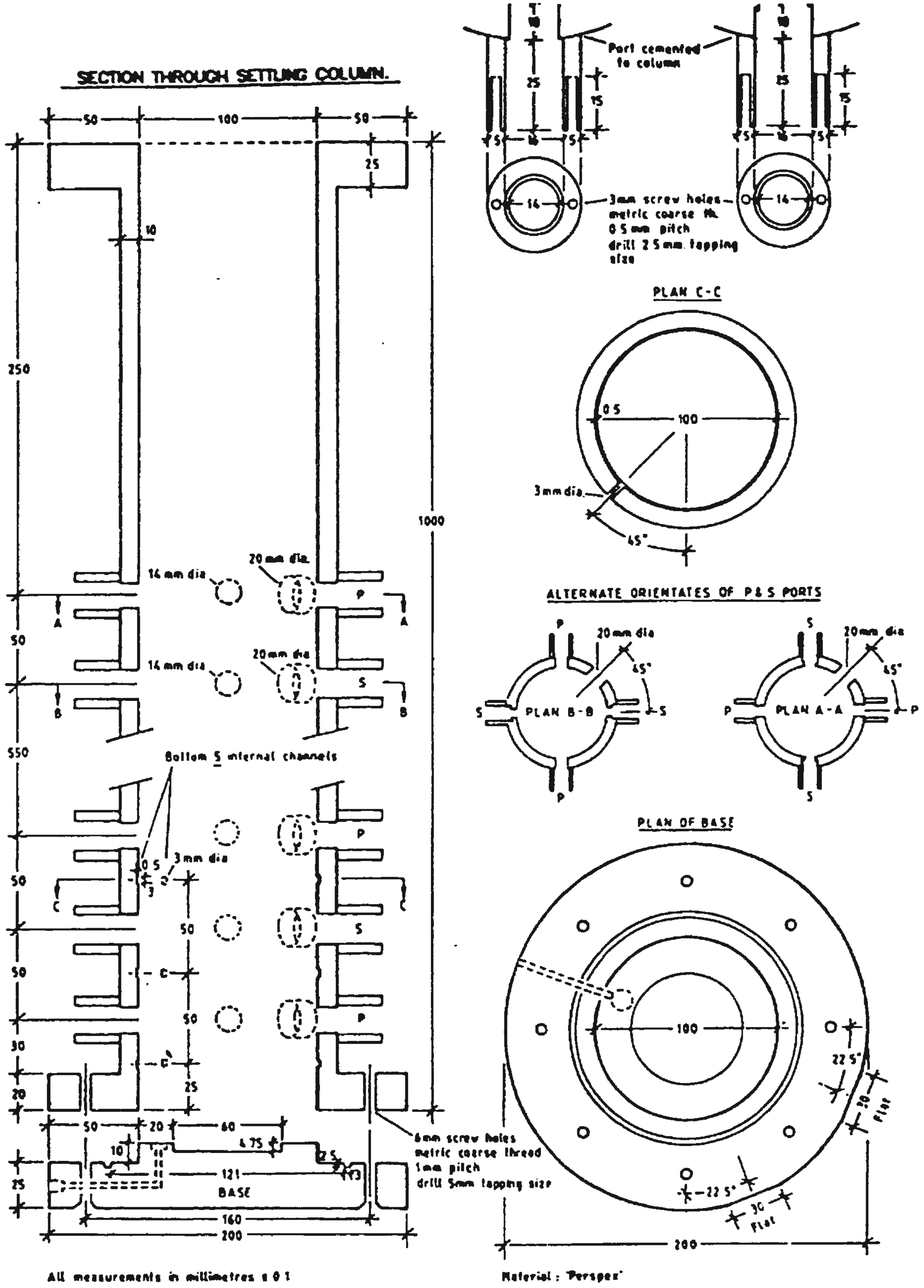


Figure 2 McDermott's (1992) Instrumented Settling Column

Miyake *et al.* (1988) used acrylic cylinders to conduct 1 g as well as centrifuge consolidation studies on samples of dredged marine clays. Some of their columns were also equipped with porous stones to permit drainage from the base. The maximum height of the 1 g samples was 1 m while the centrifuge samples ranged in height from 6 to 60 cm, however the 60 cm long samples were only accelerated to 10 g. An interesting feature incorporated into both types of columns (centrifuge and 1 g) was a piston that allowed the sample to be extruded after consolidation was completed. This allowed an accurate water content profile to be obtained after each test. Other Japanese researchers such as Kitazume *et al.* (1993) have also used transparent graduated cylinders to study the settlement behaviour of soils in the geotechnical centrifuge.

The geotechnical centrifuge at the University of Boulder has been used to conduct soft soil sedimentation / consolidation studies (You and Znidarcic (1994), Illangasekare *et al.* (1991)). Transparent acrylic cylinders were again used with sample heights limited to a maximum of about 300 mm. These types of containers were also used at the University of Florida to study the sedimentation / consolidation behaviour of phosphatic clays (Bloomquist and Townsend (1984)).

2.3.3 Column Measurement Systems

2.3.3.1 Pore Pressure

One of the most important measurements in any column experiment is excess pore pressure. The excess pore pressure is actually obtained by measuring total pore pressure and then subtracting hydrostatic pressure. The majority of pore pressure measurements have been accomplished using electronic pressure transducers (McDermott, (1992), Elder (1985), Been and Sills (1981), Tan *et al.* (1988), Sills and Thomas (1984), Scott *et al.* (1986).). The transducers were mounted in housings attached to the column walls and separated from the soil by porous filters. However, manometers or standpipes have also been used to measure excess pore pressure (Liu *et al.* (1994)). These are thin transparent vertical tubes attached through the column sidewall which allow the vertical elevation of the water surface to be read manually. Due to the low effective stresses generated during 1 g self-weight consolidation tests, generally < 10-15 kPa depending on the soil type and column height, the accuracy of the manometer technique is questionable. Williams (1988) also utilised the standpipe technique for his 1 g studies on the consolidation properties of coal mine tailings. Been (1980) initially also used a series of standpipes. However, this method later proved unsatisfactory because they resulted in a flow of water into the soil as the pressure decreased.

Centrifuge modelling of soft soil sedimentation / consolidation necessitates the use of miniature electronic pore pressure transducers due to the effect of self weight and scaling considerations. The small size of these pore pressure transducers (approximately 6 mm diameter and 12 mm long) is advantageous when the transducer is placed in a soil layer to measure pore pressure dissipation during consolidation (Williams, 1988). Pore pressure transducers used in the geotechnical centrifuge must also have a capacity far greater than transducers used under 1 g conditions due to the increase in the level of effective stress. This is a major advantage of centrifuge sedimentation/consolidation experiments over 1 g experiments. The level of effective stress generated in the centrifuge is much higher and more closely replicates effective stresses generated under field conditions in deep deposits.

2.3.3.2 Settlement

Settlement measurements have usually been made with a scale attached to the outside of a transparent column. This technique has also been used in the centrifuge and combined with a remote video camera to monitor surface settlement (Illangasekare *et al.* (1991), You and Znidarcic (1994), Miyake *et al.* (1988), Williams (1988)). Stone *et al.* (1993) used a linear variable differential transformer (LVDT) to monitor settlement during centrifuge tests to investigate the consolidation behaviour of gold mine tailings at the University of Western Australia. LVDT's have also been used by Kitazume *et al.* (1993) to monitor the settlement during centrifuge testing of sand/clay samples.

2.3.3.3 Density Measurements

Bulk density measurements for 1 g column experiments have been conducted using a variety of techniques. Scott *et al.* (1986) used side wall taps which could be opened to extract a sample for water content or density measurement. McDermott (1992) also measured density by inserting a hypodermic syringe through a sampling port in the column wall and extracting a sample of soil slurry. By measuring the volume and mass of the extracted sample, bulk density could be easily calculated. This method was also used by Lin and Lohnes (1984). Disadvantages of the sample extraction method include soil disturbance during sampling, a necessity to correct settlement measurements (Lin and Lohnes, 1984) and the fact that these types of measurements are possible only at higher water contents. Only the soil reaches some degree of consolidation, it will no longer flow from a sampling port.

A non-destructive x-ray technique (Been and Sills (1981), Sills and Thomas (1984), Elder (1985), McDermott (1992)) has also been used to measure bulk density. Density measurements were accomplished by traversing an x-ray apparatus along the column length and projecting a collimated beam of x-rays through the settling column. The amount of x-ray radiation passing through the column was detected using a sodium iodide crystal and photo multiplier assembly and an exponential relationship between density and count rate was obtained. An example of a density profile obtained using the x-ray technique is given in Figure 3.

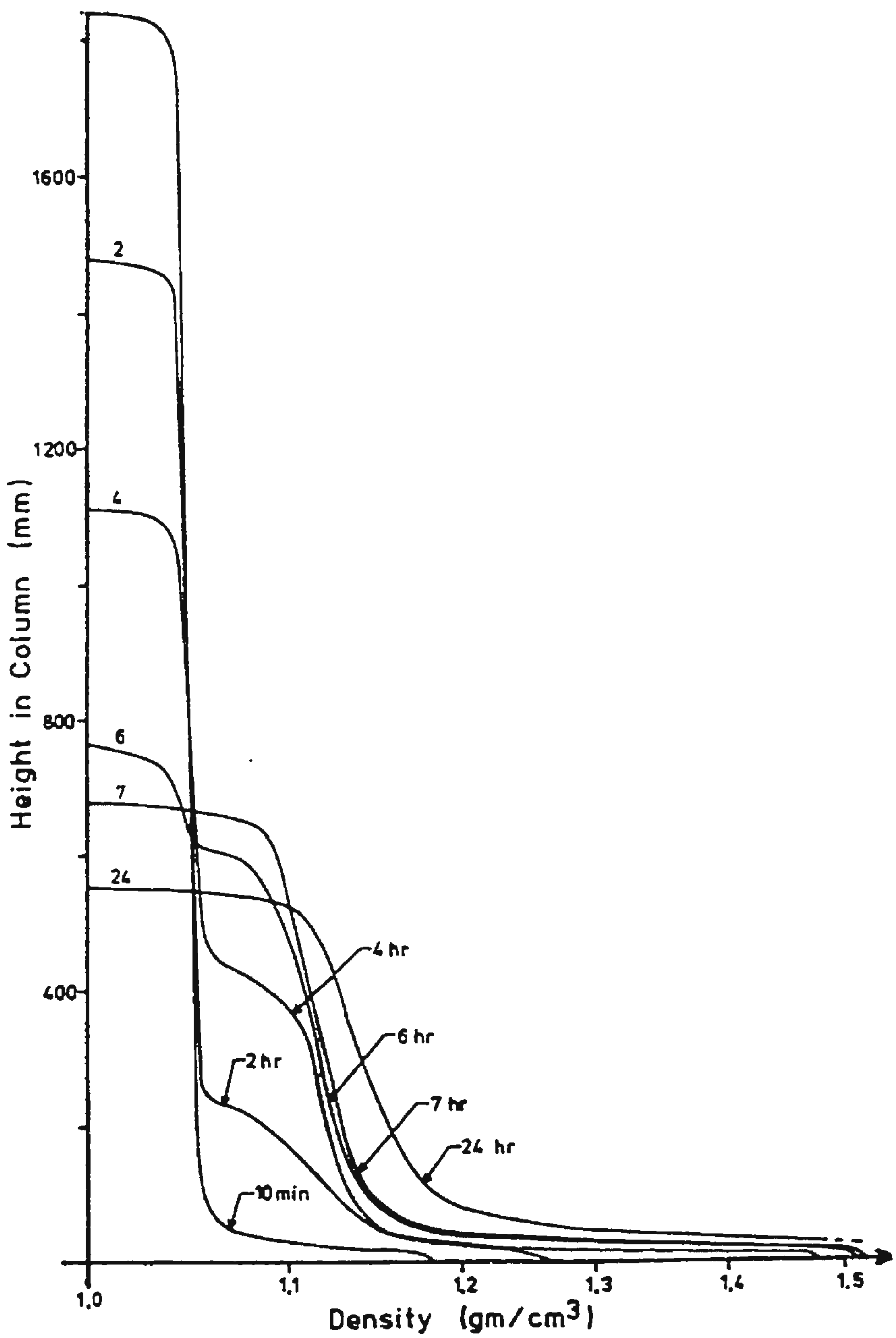


Figure 3 Density Profile (After Been and Sills, 1981)

Tan *et al.* (1988) developed a large strain consolidation cell to study the compressibility and permeability relationships for very soft clay. They reviewed the density measurements of Been and Sills (1981) and concluded that the readings were inaccurate due to the low effective stress levels and the small changes in bulk density. The consolidation cell developed by Tan *et al.* (1988) could be used to apply a constant surcharge to a soil sample thereby increasing effective stress levels. Instead of an x-ray density measurement system similar to Been and Sills (1981), density measurements were accomplished using a Gamma ray attenuation technique (detector and a cesium-137 Gamma ray source). The authors do not explain why they did not use the x-ray technique with their consolidation cell. A disadvantage of the gamma ray system is that a radioactive source will gradually decay with time. Another disadvantage, also applicable to the x-ray system, is that these types of measurement systems can have serious health effects if not operated with the necessary degree os care and control.

Besides using the sampling technique and an x-ray system to measure bulk density, McDermott (1992) also utilised an electrical resistivity technique to measure bulk density.

2.3.3.4 Description of Seismic Waves - Elastic Theory

Seismic waves are considered to consist of elastic strain energy (Kearey and Brooks, 1991) and propagate outward from a seismic source. Seismic sources that are used to conduct seismic surveys typically generate extremely short wavetrains, which are known as pulses. These pulses can contain a wide range of frequencies. The strains that are associated with a seismic pulse are very small ($<10^{-6}$, Stoll, 1989) and are assumed to be elastic. As a result, the velocity with which a seismic pulse is transmitted through a particular medium is determined by consideration of the elastic moduli and the density of the medium.

Compressional waves, which are sometimes called longitudinal, primary or P-waves, propagate through a material by compressional and dilational uniaxial strains in the direction of wave propagation (Kearey and Brooks, 1991). Shear waves, which are often referred to as transverse, secondary or S-waves, propagate by a pure shear strain in a direction perpendicular to the direction of wave travel. This research program is focussed mainly on the propagation of waves of the first type, namely compressional waves, as they propagate through a fluid saturated sediment at various degrees of consolidation. Compressional wave velocity is used regularly to characterise marine sediments (Boyce (1980), McCann (1968)).

2.3.3.5 Compressional Wave Measurements In Soft Soils

The equation for the velocity of compressional waves, V_p, propagating through an infinite, homogeneous, isotropic and elastic medium comes from the solution of the wave equation:

$$V_p = \sqrt{\frac{B+\frac{4}{3}G}{\rho}} \qquad [7]$$

where, B = bulk modulus (resistance to change in volume)

G = shear modulus (resistance to change in shape)

ρ = bulk density

An early development of equation 7, for a zero shear modulus, to explain how a compressional wave was transmitted through a suspension was made using the Wood's equation:

$$V_P = \sqrt{\frac{1}{(\eta C_w + (1-\eta)C_s)\ (\eta\rho_w + (1-\eta)\rho_s)}} \qquad [8]$$

where C_w (= $1/B_w$) and C_s (= $1/B_s$) are the fluid and solid compressibilities, ρ_w and ρ_s are the fluid and solid densities and η represents porosity. The Wood's equation is applicable for soils with no rigidity (i.e. suspensions) and represents a lower limit for the velocity of compressional waves in a fluid saturated soft soil (Hamilton, 1971).

As a soil framework starts to develop, the soil rigidity increases and therefore compressional wave velocity also increases to a value higher than that predicted by the Wood's equation. Urick and Hampton (from Ogushwitz (1985) and McDermott (1992)) performed laboratory experiments to measure compressional wave velocity in kaolinite suspensions ranging in porosity from 98% to 65%. It was discovered that a distinct minimum existed in the compressional wave speed (V_p) vs porosity (η) plots between 75% and 80% porosity. Ogushwitz (1985) stated that the Biot theory (1956) provided a good fit to the data of Urick (1947) and Hampton (1967) and that the porosity at which the minimum P-wave velocity occurs could be predicted using the Biot theory (1956). The Biot (1956) theory can be used to predict the propagation of seismo-acoustic waves through saturated porous mediums. The importance of the Biot (1956) theory is that for suspensions it reduces to Wood's equation.

However, as a soil develops the theory also takes into account the contribution from the shear modulus and the bulk modulus.

McDermott (1992) measured the velocity of 500 kHz compressional waves transmitted through natural marine sediments. McDermott's (1992) compressional wave transducer design consisted of a piezoelectric ceramic disc enclosed within an acrylic receptacle. A thin (2-3 mm) epoxy resin coating was applied to the front face of the transducer which was installed in the column such that the epoxy resin facing was in direct contact with the sediment. Wires were soldered to the front and rear faces of the piezoelectric disc, passed through the acrylic receptacle and were connected to a B-N-C connector at the rear of the transducer. Figure 4 is a diagram of McDermott's (1992) compressional wave transducer.

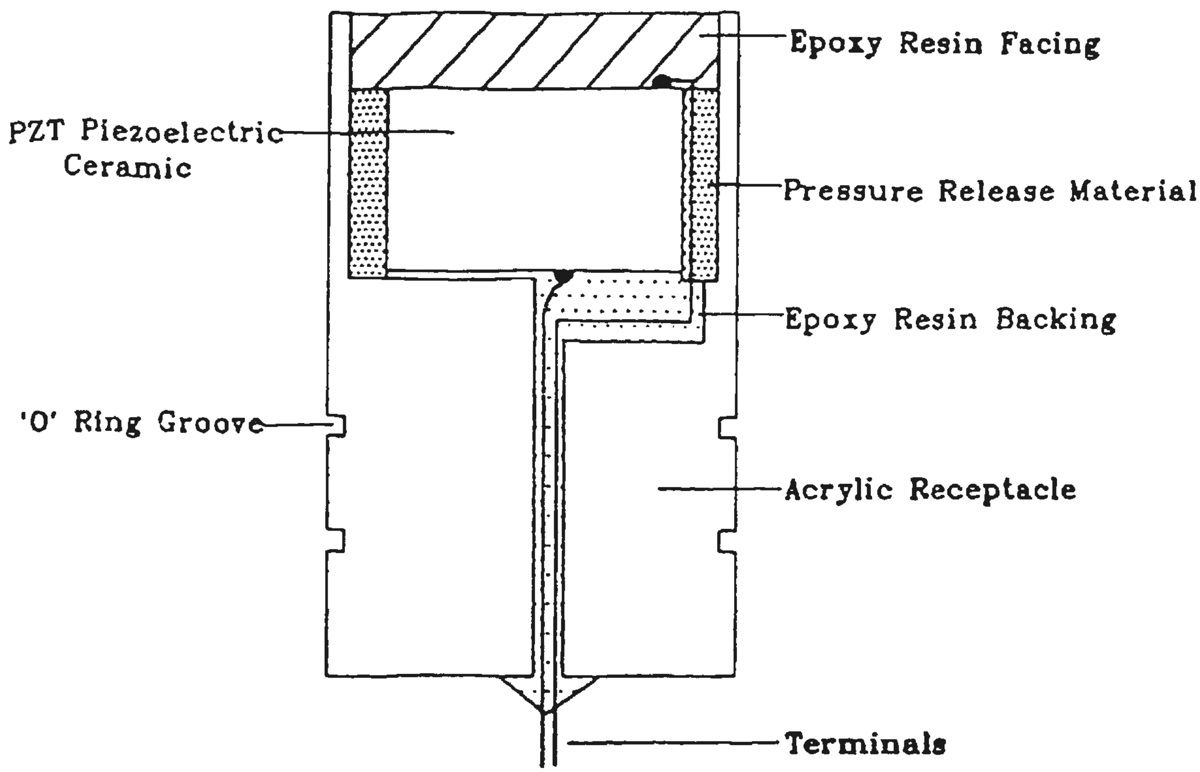


Figure 4 Compressional Wave Transducer (After McDermott, 1992)

2.4 Summary and Main Conclusions Drawn From Previous Research

The main conclusion that can be drawn from a review of previous research into soft soil sedimentation/consolidation behaviour is that an instrumented settling column is a useful tool for this type of work. A transparent column allows one to measure settlement very easily. The best method for measuring excess pore pressure is to instrument the column with a series of electronic pore pressure transducers at strategic locations. Density measurements are can be made at 1 g using an x-ray or gamma ray attenuation technique. However, these measurements should be checked by taking physical samples of the consolidating soil. Another significant conclusion is that even though 1 g columns are very useful, they are also inefficient due to the low effective stress levels and the amount of time required to conduct a single experiment, irrespective of the maximum column height. For example, Scott *et al.* (1986) refers to a 10 m high, 1 m diameter column containing oilsands tailings that had been consolidating for over 1.5 years!

Some researchers have attempted to enhance the consolidation process by applying a surcharge, developing a new type of oedometer cell or using the geotechnical centrifuge to increase the level of effective stress. With respect to centrifuge testing of soft soil sedimentation/consolidation, it is very important to establish the critical void ratio. This is important to ensure that the soil formed during self-weight consolidation in the centrifuge has the same geotechnical properties as the same soil formed under 1 g conditions. Most soft soil centrifuge experiments have been conducted in columns with heights less than 0.65 m and at relatively low gravity levels. Centrifuge measurements have been limited and consisted of settlement and pore pressure only. There is no documented evidence of density or compressional wave velocity measurements in the geotechnical centrifuge. In order to design a column capable of acceleration levels up to 100 g, careful consideration must be given to the choice of column material due to the increased stresses.

McDermott's (1992) design of compressional wave transducers proved reliable and this design can be used as the basis for the design of compressional wave transducers for the centrifuge column. As also emphasized by McDermott (1992), it is very important to establish a mixing procedure for the soil slurry so that the amount of entrained air is minimized. Air entrainment is known to inhibit primary consolidation of soft soils and has been shown to attenuate compressional wave signals. There is also no documented evidence of compressional wave measurements in the centrifuge although the Japanese have conducted centrifuge shear wave velocity measurements in sands. Table 1 summarises the key details of some relevant soft soil experiments while Table 2 summarises some geotechnical properties of speswhite kaolin clay.

Table 1 Summary of Some Relevant Soft Soil Experiments

Researchers	Apparatus Description	Soil Description	Measurements Conducted	Comments
Sills and Thomas (1981)	Acrylic, h=2 m, I.D. = 101.6 mm	Silty marine clay, w_l = 64%, w_p = 32%	Density, pore pressure, total stress, settlement	ρ_{bl} = 1080 kg/m^3, 1 g test
Tan *et al.* (1988)	Consolidation cell, h = 500 mm, I.D. = 153 mm	Marine clay, w_l = 84%, w_p = 28%, % Kaolinite = 20	Settlement, density, total stress, pore pressure	ρ_{bl} = 1229 kg/m^3 (e_i ~ 6), 1 g test with ~ 20 kPa surcharge
Liu *et al.* (1994)	Acrylic, h = 2 m, 500 mm, I.D. = 300 mm	oilsands tailings, 42-50% solids	Settlement, pore pressure	1g test, e_i ~ 2.3
Lin and Lohnes (1984)	Plexiglass, h = 1.83 m, I.D. = 140 mm	Dredge spoil slurry, w_l = 59.4%, w_p = 34.7 %, 37% clay & 63% silt	Settlement, density	ρ_{bl} ~ 75-200 g/l, 1 g test
Michaels and Bolger (1962)	Glass tubes, h (max) = 120 cm, I.D. = 5 cm	Kaolin	Settlement	1 g test with varying pH levels
Been and Sills (1981)	Acrylic, h = 2 m, I.D. = 102 mm	Uniformly graded silt with 30% clay	Settlement, pore pressure, density, total stress	1 g test, ρ_{bl} ~ 1020 - 1220 kg/m^3
Urick (1947) & Hampton (1967) from Ogushwitz (1985)	Unknown	Kaolin	Compressional wave velocity, density	1 g test
McDermott (1992)	Acrylic, h = 1m, I.D. = 99 mm	Natural marine sediments, w_l = 40-80%, w_p = 14-34 %	Settlement, density, pore pressure, compressional wave velocity, electrical resistivity, shear wave velocity	1g test, ρ_{bl} = 1130 - 1248 kg/m^3
Elder (1985)	Acrylic, h = 200 mm to 4 m, I.D. = 102 mm	Natural estuarine sediment, w_l ~ 65 %, w_p ~ 30 %	Settlement, density, pore pressure, undrained shear strength	1g test, ρ_{bl} = 1020 - 1250 kg/m^3, surcharge applied in some experiments

Scott *et al.* (1986)	Acrylic, h = 10 m, I.D. = 1 m	Oilsands tailings	Settlement, pore pressure, density	1 g test (initial solids content ~ 31%)
Illangasekare *et al.* (1991)	Plexiglass, h = 310 cm, I.D. = 83 mm	Sand/Silt	Settlement	1 g test
Illangasekare *et al.* (1991)	Plexiglass, h = 31 cm, I.D. = 21.5 cm	Sand/Silt	Settlement	20 g tests
Takada and Mikasa (1986)	Acrylic, h ~ 30 cm, I.D. ~ 5 cm	Marine clays, w_l = 100-120%, w_p = 30-40%	Settlement	100-150 g, w_i= 120-450%
You and Znidarcic (1994)	Transparent column, h ~ 120 mm (inferred)	Speswhite china clay	Settlement, pore pressure	10, 20 & 40 g, e_i ~ 4-15
Martinez (1987)	Acrylic, h = 12 cm, I.D. = 14 cm	Phosphatic clay slurry	Settlement, pore pressure	60 & 80 g tests
Bloomquist and Townsend (1984)	Plexiglass, h = 15.3 cm, I.D. = 14 cm	Phosphatic clay slurry, w_l = 176 %, w_p = 57 %	Settlement	40, 60 & 80 g tests, e_i = 6 - 16
Schiffman *et al.* (1984)	Unknown	Georgia kaolin, w_l = 44 %	Settlement	g level not stated, w_i = 110%
Stone *et al.* (1993)	Rectangular strongbox	Gold mine tailings deposited in layers	Settlement, pore pressure	100 g, w_i ~ 160%
Martinez *et al.* (1987)	Large deformation consolidometer	Phosphatic clay	Settlement, pore pressure, applied stress	e_i ~ 23
Williams (1988)	Rectangular strongbox	40 mm Gault clay over 200 mm kaolin	Settlement, pore pressure	100g, w_i (Gault clay) ~ 92%, w_i (kaolin) ~ 120%
Berre and Iversen (1972)	Oedometer h = 15 cm, I.D. = 8 cm	Norwegian marine clay w_l ~ 57%, w_p ~31 %	Settlement, pore pressure, vertical stress, side friction	Samples tested at natural water content (~ 60%)
Kitazume *et al.* (1993)	Perspex, h = 64 cm, I.D. = 11 cm	Marine clays, w_l = 83%, w_p = 39%	Settlement, pore pressure	25 g tests, w_i = 160- 30%

Table 2 Geotechnical Properties of Speswhite Kaolin Clay

Liquid Limit, w_l	59.0 (McDermott and King, 1998) 69.0 (Al Tabba, 1987) 53.1 (Liu, 1990)
Plastic Limit, w_p (%)	32.0 (McDermott and King, 1998) 38.0 (Al Tabba, 1987) 31.9 (Liu, 1990)
Plasticity Index, I_p	27.0 (McDermott and King, 1998) 31.0 (Al Tabba, 1987) 21.2 (Liu, 1990)
Specific Gravity, G_s	2.63
Mean Grain Size, D_{50} (μm)	0.5
Clay Fraction (< 2 μm) (%)	76.0 (McDermott and King, 1998) 80.0 (Al Tabba, 1987) 75.0 (Liu, 1990)
Coefficient of Consolidation, C_v (mm^2/s)	0.10 (Al Tabba, 1987) 0.79 (Manson, 1980 from Al Tabba, 1987) 0.50 (Clegg, 1981 from Al Tabba, 1987) 0.60 (Elmes, 1986 from Al Tabba, 1987)
Permeability, k (m/s)	7.1×10^{-8} (McDermott and King, 1998) (w_i = 117 %) 1.3×10^{-8} (McDermott and King, 1998) (w_i = 70 %) 6×10^{-6} (w=95%) (Al Tabba, 1987) 0.5×10^{-6} (w= 19%) (Al Tabba, 1987)
Permeability / Void Ratio Relationship (k vs. e)	$k = 0.5\ e^{3.25} \times 10^{-6}$ (mm/s) (Al Tabba, 1987)
Void Ratio / Effective Stress Relationship (e vs.σ')	e = 2.13-0.187 ln p' (Al Tabba, 1987) e = 2.62 - 0.25 ln p' (Phillips, 1986) e = 2.16 - 0.17 ln σ (McDermott and King, 1998)
Coefficient of Lateral Earth Pressure During One-Dimensional Normal Consolidation (K_{onc})	0.64 (Al-Tabba, 1987)

3.0 RESEARCH FACILITIES

3.1 The C-CORE Centrifuge Centre

The C-CORE Centrifuge Centre is a research facility located on the campus of Memorial University of Newfoundland between the Captain Robert A. Bartlett building and the S.J. Carew Building. The centrifuge centre was established through funding provided by the Canada/Newfoundland Offshore Development Fund, the Technology Outreach Program of Industry, Science and Technology Canada and the Natural Sciences and Engineering Research Council Canada.

The centre is essentially a two story building, with offices and a soils laboratory on the upper level and a model preparation area on the lower level, and a containment structure which houses an Acutronic 680-2 geotechnical centrifuge. The upper level of the three level containment structure provides a stiff ceiling for the main centrifuge chamber to resist the aerodynamic excitation imposed by the centrifuge in rotation. The upper level also houses the electrical slipring capsule and associated interfaces. The intermediate level is the main centrifuge chamber which is accessible by forklift from the main building. The centrifuge chamber is 13.5 m in diameter and 4.2 m high. The 300 mm thick reinforced concrete wall is aerodynamically clean inside and retains a rockfill safety berm outside. The lower level is underground and contains the centrifuge drive unit and the refrigeration unit. The model preparation area consists of a machine shop, a sand raining room, an electronics lab, a cold room and an X-ray facility (Paulin, (1998)). The plan of the C-CORE Centrifuge Centre is shown in Figure 5.

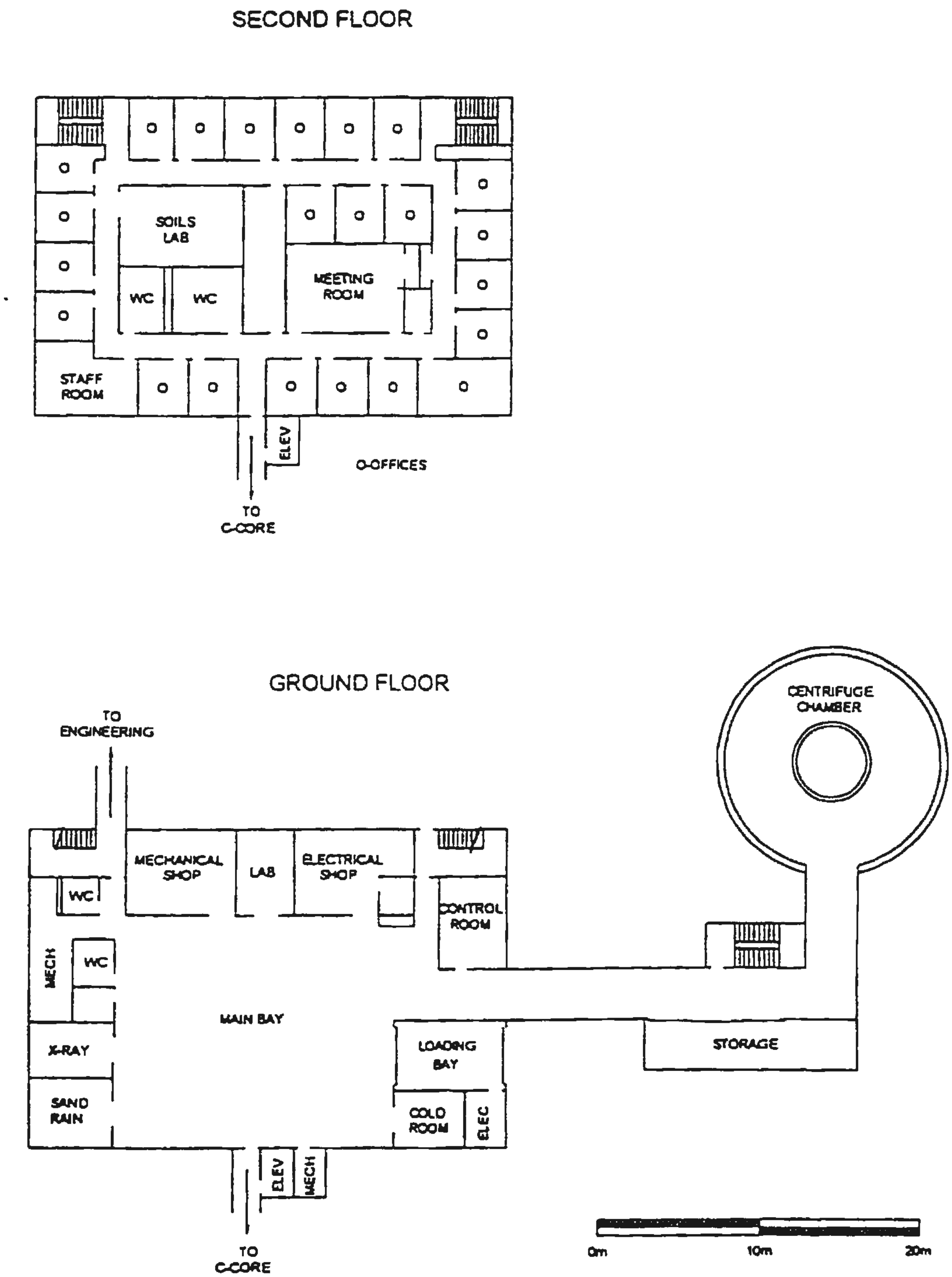


Figure 5 The C-CORE Centrifuge Centre (After Paulin, 1998)

3.2 The Acutronic 680-2 Centrifuge

The Acutronic 680-2 centrifuge, shown in Figure 6, is capable of testing models to 200 g and has a radius of 5.5 m to the surface of the swinging platform. The test package centroid is typically at a nominal working radius of 5 m. At the maximum centrifuge rotational speed of 189 rpm, the acceleration of the package is approximately 200g. The C-CORE centrifuge has a maximum payload capacity of 100 g x 2.2 tonnes = 220g-tonnes at the 5 m working radius. This capacity reduces to 130g-tonnes at 200 g due to the increased self-weight of the platform. The capacity and the specifications of the Acutronic 680-2 centrifuge are given in Figure 7. The maximum payload size is 1.1m high by 1.4m long and 1.1m wide (Paulin, 1998).

Figure 6 C-CORE Acutronic 680-2 Geotechnical Centrifuge

Centrifuge Specifications

Machine:	Acutronic 680-2
Platform Radius	5.5m
Platform Width:	1.1m
Platform Depth:	1.4m
Container Height:	1.1m
Maximum Usable Height:	1.5m
Acceleration Range:	10 - 200g
Acceleration Accuracy:	0.5%
Maximum Operating Imbalance:	100kN

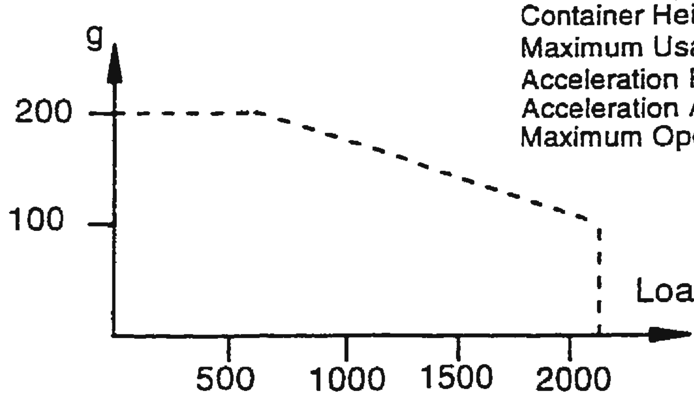


Figure 7 C-CORE Centrifuge Specifications

The centrifuge arm consists of two parallel steel tubes held apart by a central drive box and spacers as shown in Figure 8. The swinging platform is suspended on pivots from the ends of the load carrying beams and is covered by an aerodynamic shroud to reduce drag. The platform and the payload are balanced by a 20.2 tonne mass counterweight. The position of this counterweight is adjusted by driving a series of gearwheels along screwthreads on the outside of the parallel steel tubes using an electric motor.

The centrifuge arm rotates on a set of tapered roller bearings inside the central drive box and is mounted on a stationary shaft. This shaft is attached to the concrete base through a four branch star support suspended on four springs. Each of the four springs is strain-gauged to sense imbalance within the centrifuge arm to within 10kN.

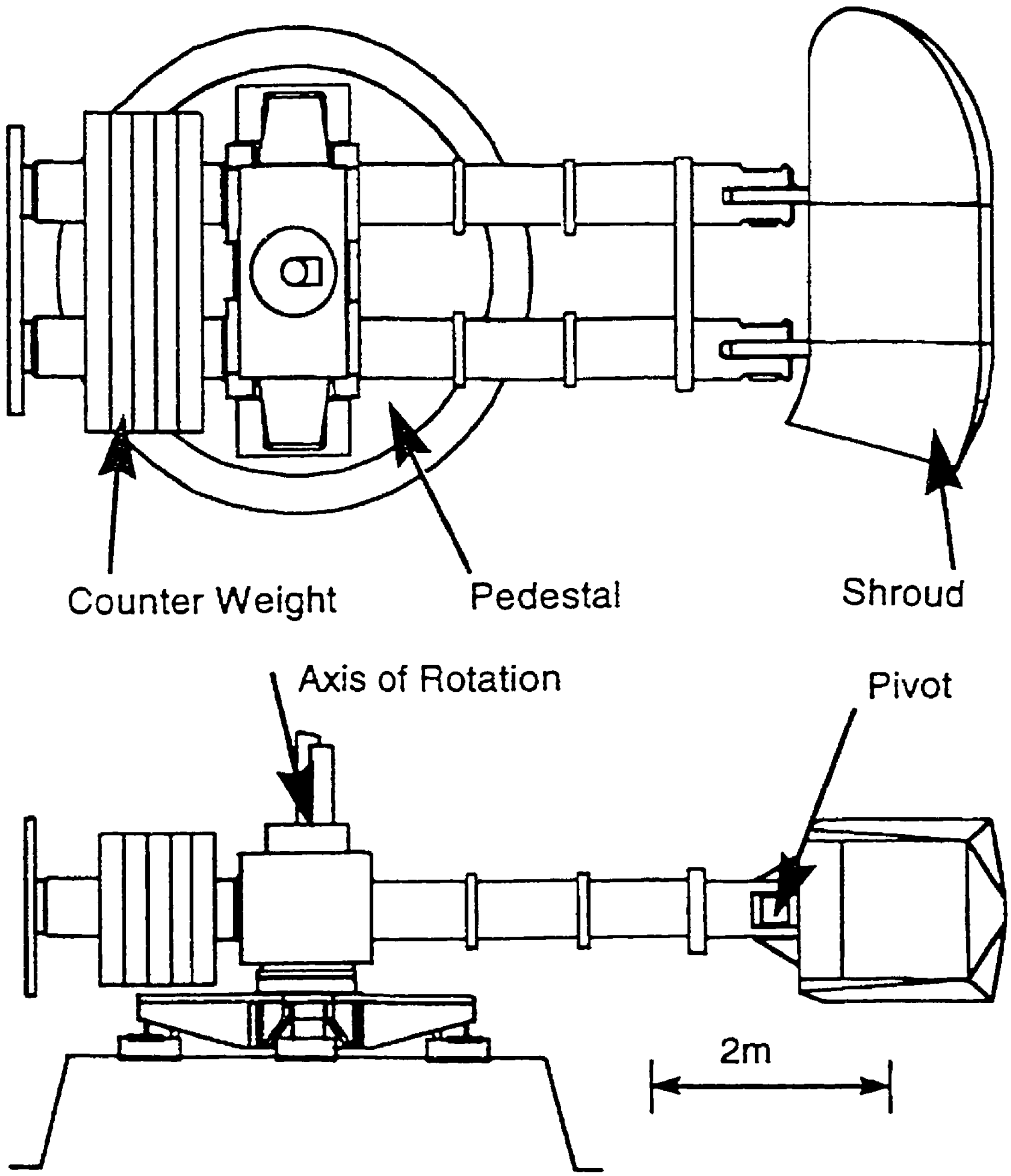


Figure 8 Acutronic 680-2 Centrifuge

The centrifuge drive unit comprises a 450kW AC variable speed motor and a 9:1 gear reducer. The variable speed motor is energized through two 250kW invertors connected in parallel. Precision couplings and a hollow vertical drive shaft connect the hollow output shaft of the gear reducer to the central drive box.

4.0 DEVELOPMENT OF AN INSTRUMENTED SETTLING COLUMN FOR CENTRIFUGE RESEARCH

4.1 Column Design Considerations

The settling column which was developed for this research program was manufactured from Schedule 80 PVC pipe. The column is one 1 m high, has a nominal internal diameter of 241 mm and a nominal wall thickness of 15 mm. Even though a review of the literature revealed that previous columns had been constructed using plexiglass, this material was not chosen to construct the centrifuge column for several reasons. A one metre high plexiglass column with the wall thickness required to withstand the large stresses generated in the centrifuge would be very expensive. There would also be machining considerations with a thick walled plexiglass column for pore pressure and compressional wave transducer installation. This necessitated the use of relatively inexpensive PVC pipe rather than a transparent plexiglass pipe. The high pressures that would be generated in the centrifuge under an acceleration level up to 100 g (> 1000 kPa) made it necessary to reinforce the column to ensure its structural integrity during testing using four 6 mm thick steel rings. These steel rings are placed around the column exterior and reinforce approximately the lower half of the settling column.

During testing the actual PVC pipe sits in a circular steel base plate, which has an approximate diameter and thickness of 370 mm and 25 mm respectively, and is covered by a second identical steel plate. O-ring seals are incorporated into the base and lid to prevent leakage during testing. The settling column apparatus is held together using four steel tierods which are threaded into the base plate and pass through clearance holes in the lid. Prior to testing in the centrifuge, the lid is securely bolted down and the entire column apparatus secured inside one of the C-CORE strongboxes. Appendix A contains column design calculations while detailed drawings of the various column components can be found in Appendix B.

4.2 Pore Pressure Measurement System

4.2.1 General

Been (1980) stated that, for soft soils, the main difference between settling and consolidation was in existence of effective stresses. During the settling stage (stage A), effective stresses are non-existent. A soil is formed when effective stresses are developed due to the self-weight of the soil particles. Consolidation occurs as excess pore pressures are dissipated and the soil particles move closer together. It was therefore very important that the settling column have the capability to measure excess pore pressure dissipation during centrifuge testing.

4.2.2 Apparatus

Measurement of excess pore pressure is accomplished using "Druck" type miniature pore pressure transducers (PPT's). These transducers have a diameter and length of 6 mm and 12 mm respectively and are designed specifically for centrifuge testing.

In order to incorporate the PPT's in the column, seven 12 mm diameter holes were drilled and tapped through the column wall at the required PPT locations. The male portion of a Conax pass-through stainless steel coupling was screwed into the column wall at each hole location. The internal diameter of the coupling itself was 6 mm. A PPT was inserted into the coupling so that the face of the transducer was flush with the inside surface of the column. A 6 mm diameter rubber bushing was then placed inside the coupling around the PPT cable and the female component of the coupling was attached. When the female part of the coupling was tightened, pressure was applied to the rubber bushing thereby creating a watertight seal. The PPT cable passed through the rear of the coupling and was attached to a signal conditioning box on the centrifuge package. Figure 9 is a photograph showing a pore pressure transducer and the components of a stainless steel coupling. Figure 10 is a photograph which shows how the entire apparatus appears after installation in the column.

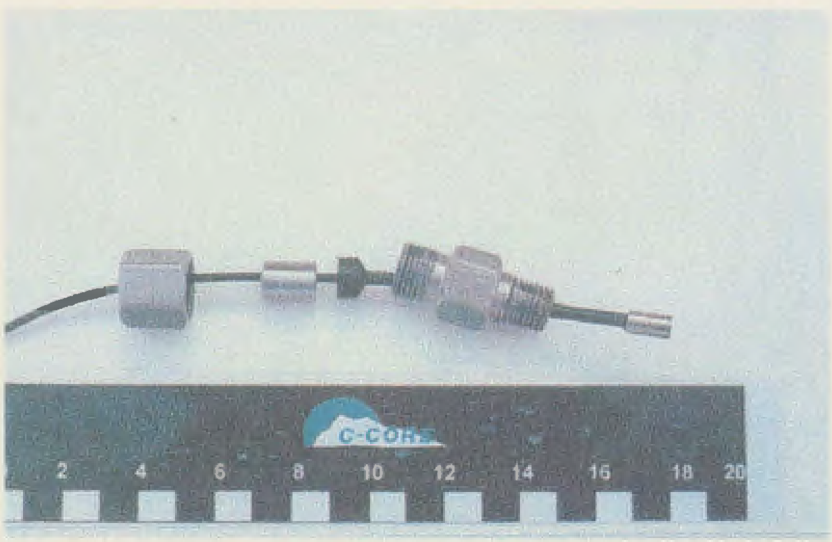


Figure 9 Pore Pressure Transducer and Coupling

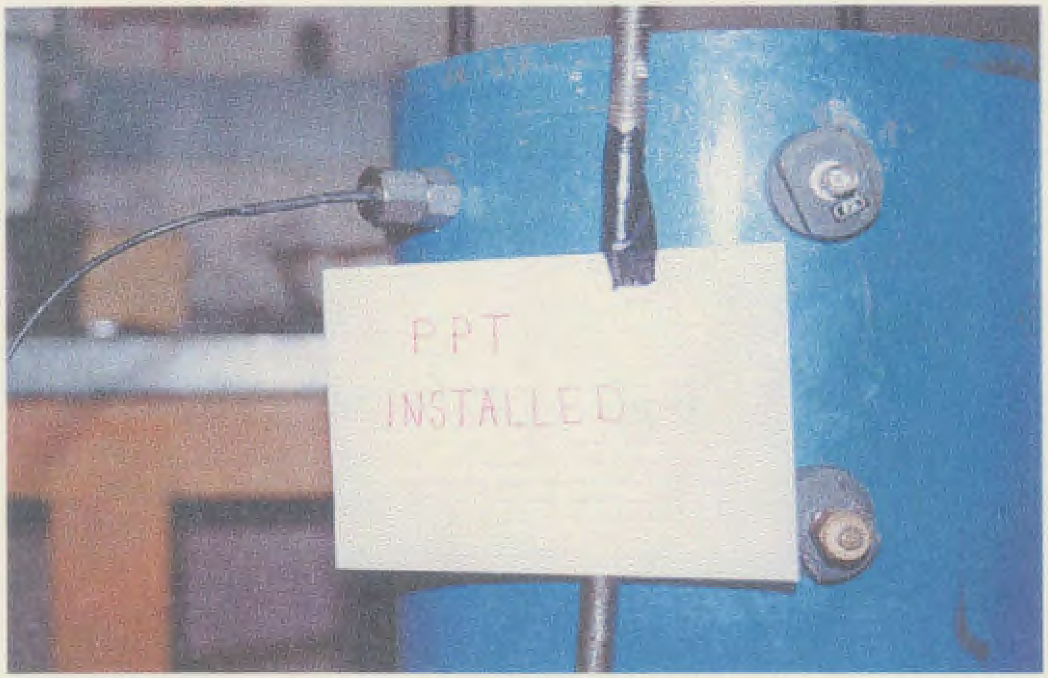


Figure 10 PPT Installed in Column

4.3 Compressional Wave Measurement System

4.3.1 General

The research objective was the development of a settling column that could be used to develop correlations between geophysical and geotechnical properties of soft soils as they undergo self weight consolidation in a geotechnical centrifuge. The research focussed on the development of robust compressional wave transducers and the incorporation of these transducers in the column.

4.3.2 Apparatus

Each compressional wave transducer was constructed in-house and comprised a piezoelectric ceramic disc, rubber backing material, a PVC housing and a BNC connector. The inside of the PVC housing was drilled to the required diameter (slightly larger than the disc outside diameter) and a circular piece of sound absorbing backing material, approximately 10 mm thick, was placed inside. The backing material had a small (~ 3 mm) clearance hole through which wires were able to pass. The piezoelectric ceramic disc, which had a diameter of 12.7 mm, a thickness of 8 mm and a resonant frequency of 250 kHz, was then placed inside the housing and wires which were soldered to the front and rear faces passed through the backing material and out through the rear of the housing. The rubber backing material prevented direct contact between the housing and the disc and served to absorb sound waves emitted from the rear of the disc and prevent interference from reflections off the PVC housing. The wires were soldered to a BNC connector which was then attached to the rear of the PVC housing using four small screws. The area between the sides of the piezoelectric disc and the interior surface walls of the PVC housing was filled with a pressure release material. The surface of the transducer was then coated with a 3 mm thick epoxy resin. When completely assembled, the compressional wave transducer had a diameter of 35 mm, a length of 44 mm (excluding the BNC connector) and weighed approximately 250g. Figure 11 is a schematic of the compressional wave transducer apparatus while Figure 12 and 13 are photographs of the transducer components and the assembled transducer respectively. Impedance plots for the transducers are contained in Appendix C.

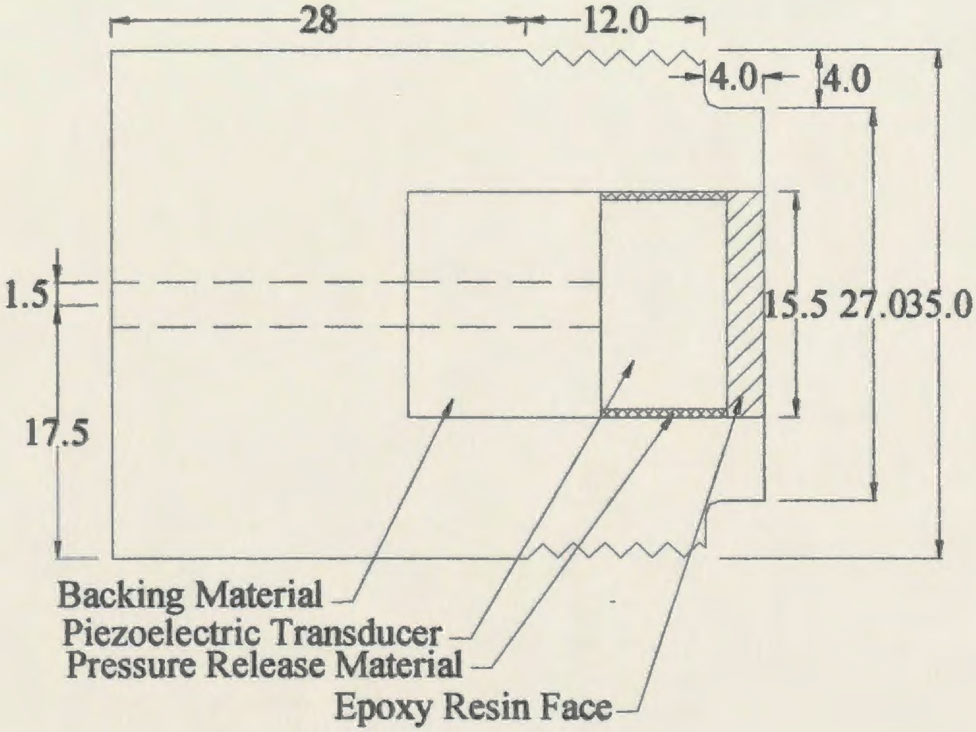


Figure 11 Schematic of Compressional Wave Transducer

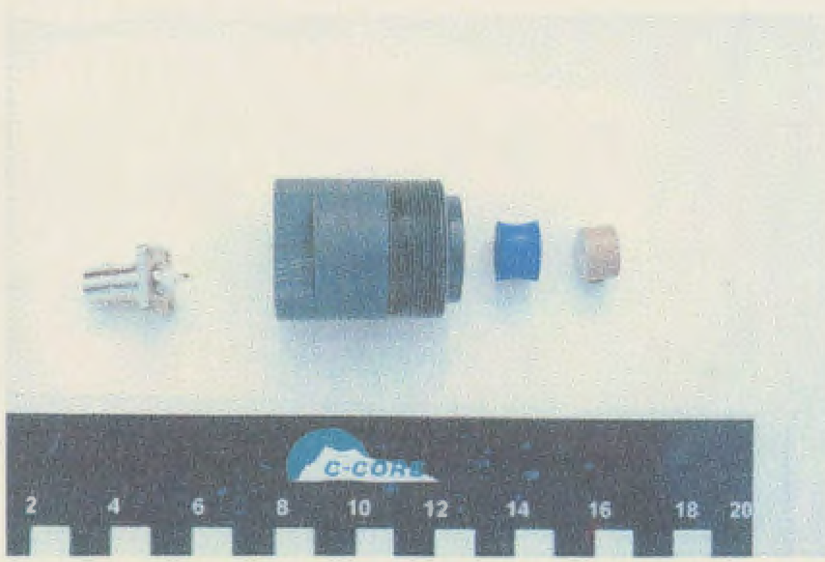


Figure 12 Components of Compressional Wave Transducer

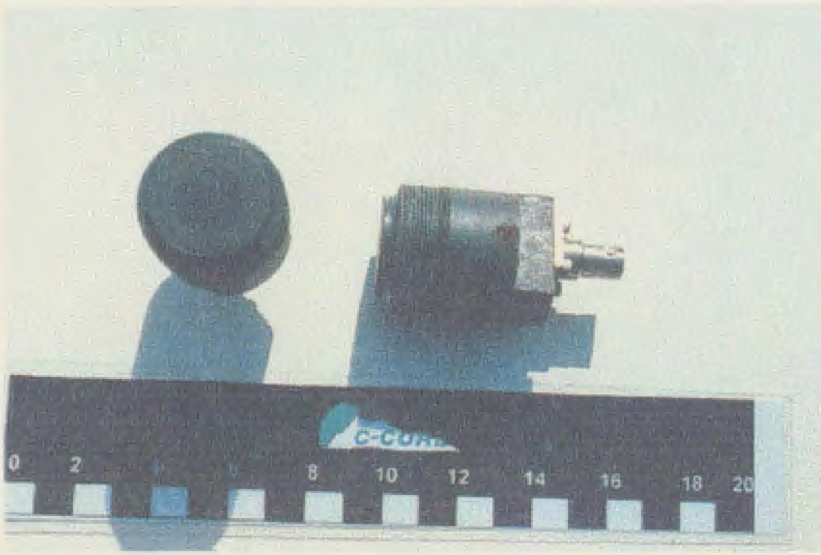


Figure 13 Compressional Wave Transducer (Plan and Profile Views)

The compressional wave transducers were installed in the column by drilling and tapping holes through the column wall at the required locations. These holes have a reduced cross sectional area and only the larger diameter section of the hole is tapped. A compressional wave transducer is screwed into the hole and an O-ring seal prevents leakage during centrifuge testing.

The set of two 250 kHz compressional wave transducers was located 50 mm from the base of the column and oriented 180° apart. Figure 14 is a schematic of the system used for P-wave generation and velocity measurement during testing. One P-wave transducer was excited using the falling edge of a 20 V pulse. The excitation pulse was sent to the transmitting P-wave transducer via the centrifuge slip rings. On energizing, the P-wave transmitter caused a small amplitude compressional wave to be generated in the soil which was detected by the receiving transducer. The compressional wave signal was amplified, sent back through the centrifuge slip rings via a co-axial cable and viewed using a digital Tektronix Oscilloscope (No. TDS-420). A data acquisition program was used to view the compressional waves on a computer monitor and to store the signals on a PC.

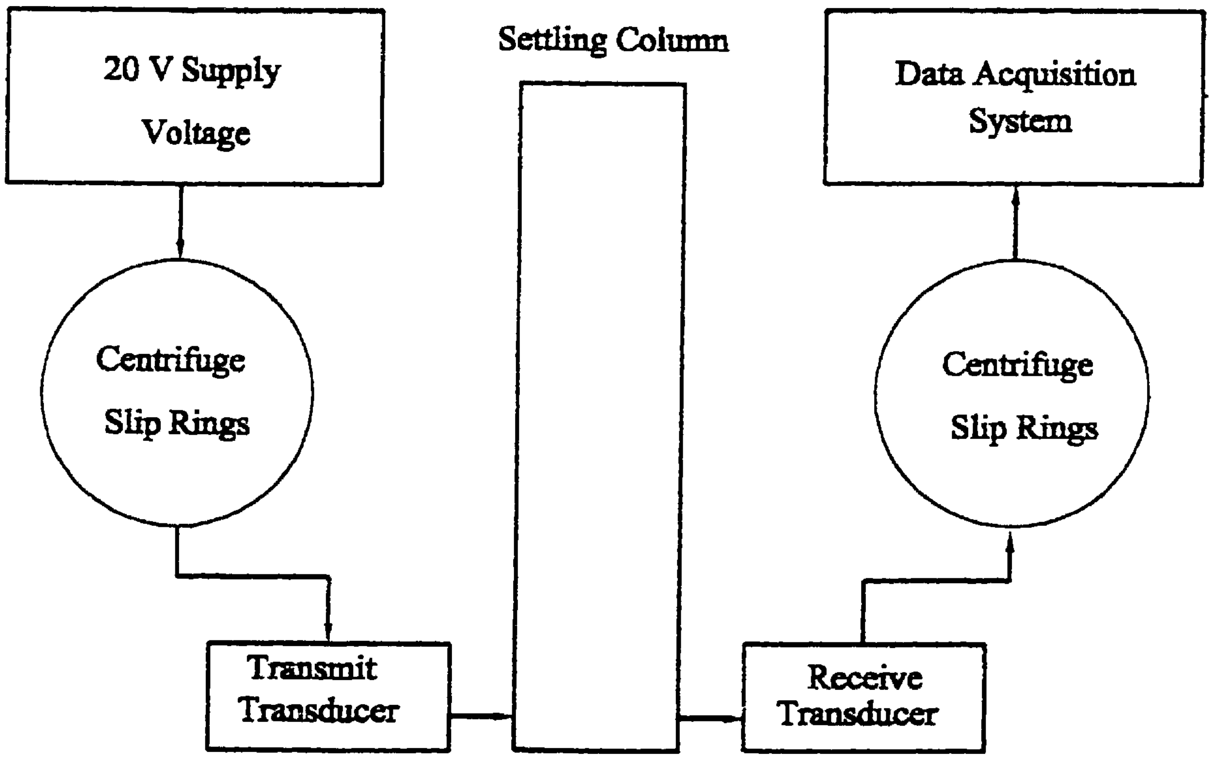


Figure 14 Compressional Wave Velocity Measurement System

4.4 Density Measurement System - Electrical Resistivity Technique

4.4.1 General

The research included the development of a system that could be used to measure the density of the consolidating soil during centrifuge testing. A correlation could then be derived between soil density and compressional wave velocity.

The first system developed for this purpose, an electrical resistivity technique, was only partially successful. Therefore, a second system based on a gamma ray attenuation technique was developed. The following sections describe both systems in detail.

4.4.2 Principle of Measurement

When an electric current passes through a porous medium most of the conduction occurs through the pore fluid. Resistivity is defined, after Jackson (1975a), as the electrical resistance of a cube of material with the current flowing normally to one face. The resistivity of a porous medium is dependent on the percentage of the cube that is pore space and the way in which these pore spaces are interconnected. Bulk density, porosity and permeability may therefore be correlated to electrical resistivity. The measurement of the electrical resistivity of the porous medium (soil) should eliminate the pore fluid resistivity (Jackson, 1975a). Therefore, the electrical resistivity is usually converted to a parameter referred to as a Formation Factor (FF) where:

$$FF = \frac{\textit{resistivity of the porous medium}}{\textit{resistivity of the pore fluid}} \qquad [9]$$

The electrical resistivity of the pore fluid should be measured at the same temperature and pressure as the soil electrical resistivity.

The empirical relationship between Formation Factor (FF) and soil porosity is well known and has the general form:

$$FF = A \times \eta^{-\beta} \qquad [10]$$

The parameters A and β are constants and η is the soil porosity. For marine sediments, most researchers have found $A \approx 1$. McDermott (1992) reports the following relationship between Formation Factor and porosity for kaolinite:

$$FF = 1 \times \eta^{-1.8} \qquad [11]$$

Jackson (1975b) reports Formation Factors between 1 and 2 for cohesive sediments.

4.4.3 Apparatus

The electrical resistivity measurement system comprised an electrical resistivity probe and an ABEM Terrameter. The electrical resistivity probe consisted of a four-electrode array which was mounted inside one of the PVC mounts and installed in the column in a manner identical to the compressional wave transducer installation. After installation, the four gold plated electrodes protruded into the soil a total distance of 4 mm. The electrodes were arranged in the Wenner configuration. That is, all of the electrodes were in-line and spaced at an equal distance of 5 mm apart.

An ABEM Terrameter was used to obtain the resistivity of the soil by supplying an alternating current to the two outer electrodes and measuring the potential difference between the two inner electrodes. The ABEM terrameter was secured inside the centrifuge cabinet for testing and could be set up to measure resistivity at a variety of frequencies. These readings were then sent back to the centrifuge slip-ring room and saved to disk and also displayed on a computer monitor in the centrifuge control room.

4.5 Density Measurement System - Gamma Ray Attenuation Technique

4.5.1 General

Due to disadvantages associated with the electrical resistivity technique, which will be explained later, it was decided to adopt a different approach for the measurement of density of consolidating soft soils in the centrifuge. The chosen method consisted of generating gamma ray radiation on one side of the column and then measuring the amount of gamma ray attenuation through the soil column using a radiation detector on the opposite side.

4.5.2 Principle of Measurement

The degree to which a collimated beam of gamma rays is attenuated when passing through a given medium is a function of the thickness (t), density of the medium (ρ) and mass attenuation coefficient (μ) of the medium. The attenuation is detected as the change in the count rate which is observed in the presence (k) and the absence (k_t) of the attenuating mass. These variables are related by the following equation:

$$k = k_t \exp(\mu\ t\ \rho) \qquad [12]$$

4.5.3 Apparatus

The gamma ray attenuation system was comprised of a Geiger-Muller (GM) tube detector, a Cesium-137 gamma ray source and a LUDLUM ratemeter. A Geiger-Muller tube detector is a gas filled detector designed for maximum gas amplification effect. The principles of a GM counter are illustrated in Figure 15.

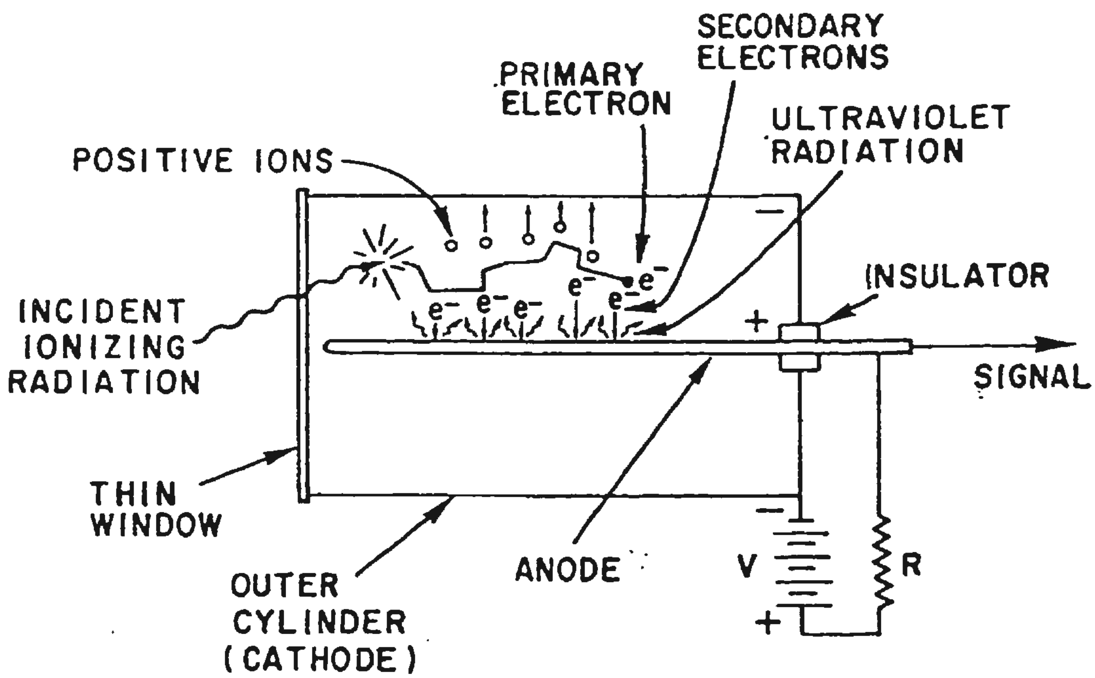


Figure 15 Operation of a Geiger Mueller Tube Counter (After Sorenson and Phelps, 1987)

The centre wire (anode) is maintained at a high positive voltage relative to the outer cylindrical electrode (cathode). The outer electrode may be a metal cylinder or a metallic film sprayed on the inside of a glass or plastic tube. The cylinder is sealed and filled with a gas, typically argon plus a quenching gas.

When ionization occurs in a GM tube counter, electrons are accelerated toward the centre wire. When electrons strike the centre wire, they do so with such energy that ultraviolet (UV) photons are emitted. Some of the UV photons travel to and liberate additional electrons from the outer wall of the chamber. These electrons are accelerated toward the centre wire, where they cause more UV radiation to be emitted, and so on. In this manner, an "avalanche ionization" is propagated throughout the gas volume and along the entire length of the centre wire.

As the avalanche progresses, the electrons, being relatively light, are quickly collected, but the heavy slow moving positive ions are not. Eventually a "hose" of slow-moving positive charges is formed around the centre wire. The avalanche then terminates because electrons in this region find themselves in a heavy cloud of positive ions and are captured by them before they reach the centre wire.

The avalanche ionization in a GM tube releases a large and essentially constant quantity of electrical charge, regardless of the voltage applied to the tube and the energy of the ionizing radiation event. The large electrical signal can be easily detected with electronic circuits.

Thus a GM counter can be used to detect and count individual ionizing radiation events (Sorenson and Phelps, 1987).

The GM tube used in the experiments was an LND Type 7121 gamma ray cylindrical radiation detector. The GM tube had an effective length of 38 mm, an overall length of 54 mm and a diameter of 14.4 mm and was positioned near the bottom of the column at the same elevation as the gamma ray source. The apparatus used to count the number of individual ionizing events during a particular length of time was a LUDLUM Model 2200 Scaler ratemeter. The ratemeter was set up in the centrifuge control room during testing.

To make in-flight density measurements, a voltage of 500V DC was supplied to the GM tube from a Hewlett Packard power supply. The gamma ray radiation passing through the consolidating soft soil resulted in ionizing events inside the GM tube. This produced an electrical signal which was amplified and then sent back to the control room through a 75Ω centrifuge slip ring. The LUDLUM ratemeter was used to count the number of ionizing events in a given time interval (usually 1 minute). The count rate was then manually recorded. The GM tube, gamma ray source and LUDLUM ratemeter are shown in Figures 16, 17 and 18 respectively.

Figure 16 LND Type 7121 Geiger Mueller Tube

Figure 17 LUDLUM Model 2200 Scaler Ratemeter

Figure 18 Gamma Ray Source in Lead Containment Vessel

5.0 CENTRIFUGE TESTING TO DEVELOP IN-FLIGHT DENSITY MEASUREMENT TECHNIQUE

Prior to the development of the gamma ray density measurement system, an electrical resistivity technique was developed to measure the in-flight density of consolidating soft soils.

5.1 Electrical Resistivity Calibration Test - Test SCOL07

5.1.1 General

In order to predict bulk density (or water content) at varying degrees of consolidation using electrical resistivity measurements in the centrifuge, it was first necessary to determine the relationship between soil density and electrical resistivity (or Formation Factor) for a Speswhite kaolin clay. This was done by performing a calibration of the electrical resistivity density measurement system. A preliminary calibration was carried out during settling column test SCOL07 on Feb. 28/1996.

5.1.2 Procedure

A soft kaolin slurry was mixed under a vacuum of approximately 65 kPa overnight to an average initial water content of 527.8% (ρ_b = 1097 kg/m^3). The slurry was placed in the column in a manner which minimized air entrainment. It was important to minimize the amount of entrained air because it has been shown (McDermott (1992)) that the presence of gas in a soft soil attenuates compressional wave signals. Even though compressional wave signals were not being obtained during this test, it was good practise to always minimize the amount of air in the slurry. The amount of entrained air in the slurry was minimized by siphoning the slurry from the mixer into several 16 l buckets which were then slowly transported to the centrifuge chamber. Each bucket was equipped with a hose and spigot. The buckets were elevated above the column and the hose was placed inside the column near the bottom. The spigot was then opened and the slurry permitted to flow from the bucket into the column. The volume of slurry required to fill the column was approximately 45l. The actual centrifuge test commenced about 30 minutes after initial slurry deposition. Figure 19 presents the experimental setup for this test.

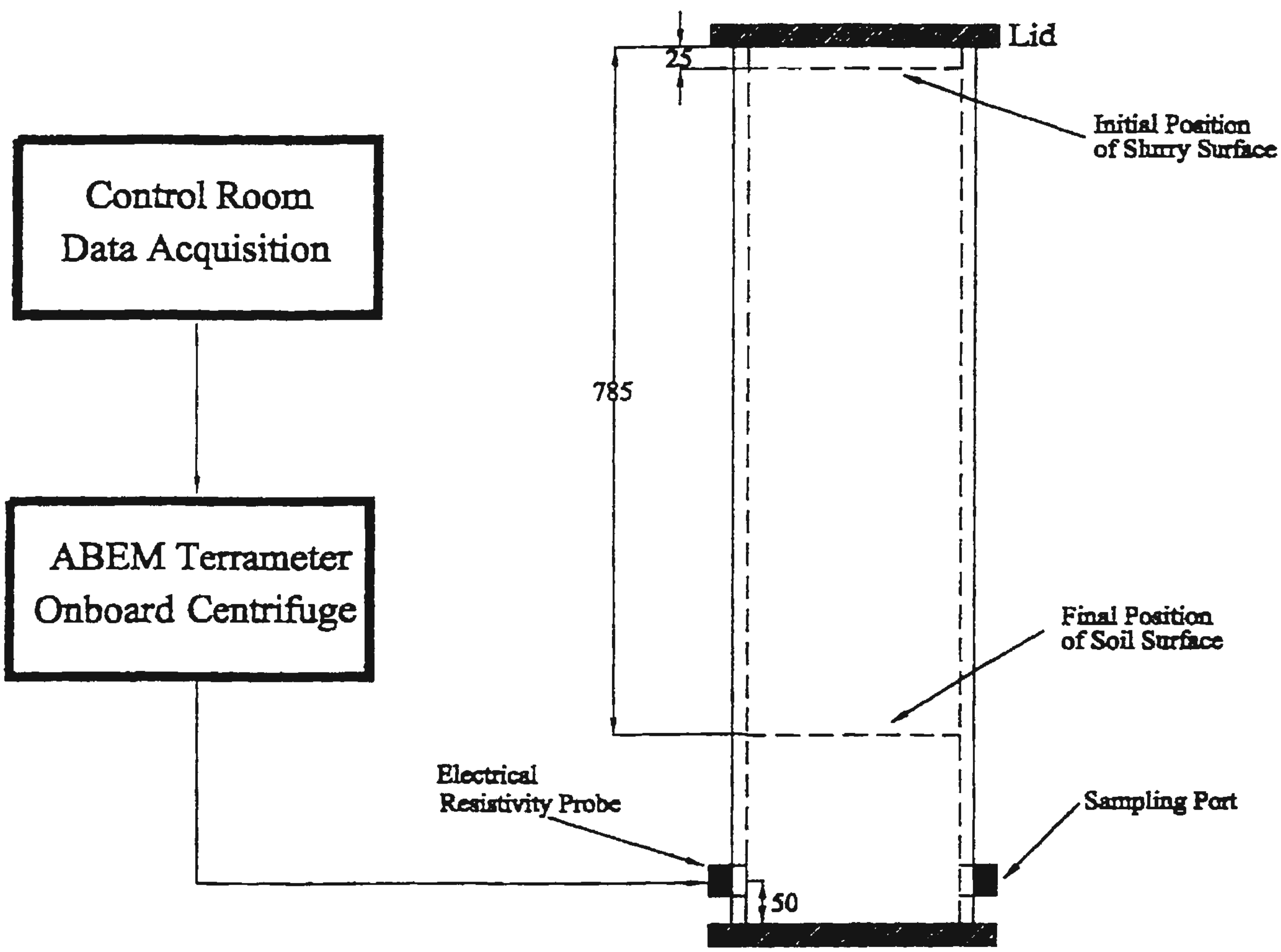


Figure 19 Experimental Setup - Centrifuge Test SCOL07

In order to obtain corresponding electrical resistivity and bulk density measurements for the calibration, it was necessary to consolidate the soil and periodically stop the centrifuge to extract soil samples for density/water content measurement. Once the centrifuge had stopped, samples of the soil were extracted using a syringe inserted through sampling ports placed at the same elevation as the resistivity probe (50 mm from the bottom of the column). The density of the material in the syringe was immediately calculated and the sample placed into a water content container and then into an oven for measurement of water content. A total of six measurements were recorded; one prior to testing, four measurements during consolidation after stopping the centrifuge and one measurement after completion of the test.

5.1.3 Test Results

The results are given in Table 3. Formation factors were calculated by dividing the measured soil electrical resistivity by the resistivity of the pore water. The pore water electrical resistivity was measured after the test at 1 g and was found to have an average value of 801.8 Ω. Electrical resistivity measurements 2-5 given in Table 3 are an average of the three resistivity readings taken immediately after stopping the centrifuge and again after sampling. The first measurement (#1) was recorded prior to spinning and the final measurement (#6) was recorded after terminating the test.

Table 3 Electrical Resistivity Calibration Test Results

Meas. #	Water Content w (%)	Bulk Density (from w) ρ_b (kg/m^3)	Bulk Density (from syringe) ρ_b (kg/m^3)	Soil Electrical Resistivity (Ω)	Formation Factor
1	527.8	1109.5	1097.0	885	1.10
2	477.8	1120.2	1209.0	1050	1.31
3	330.4	1168.2	1261.2	1210	1.51
4	110.0	1418.7	1439.2	1253	1.56
5	91.9	1477.0	1514.8	1280	1.60
6	79.7	1526.5	1532.0	1335	1.66

The calibration plot of bulk density vs. formation factor is shown in Figure 20 while Figure 21 presents soil porosity (calculated from water content measurements) vs. formation factor.

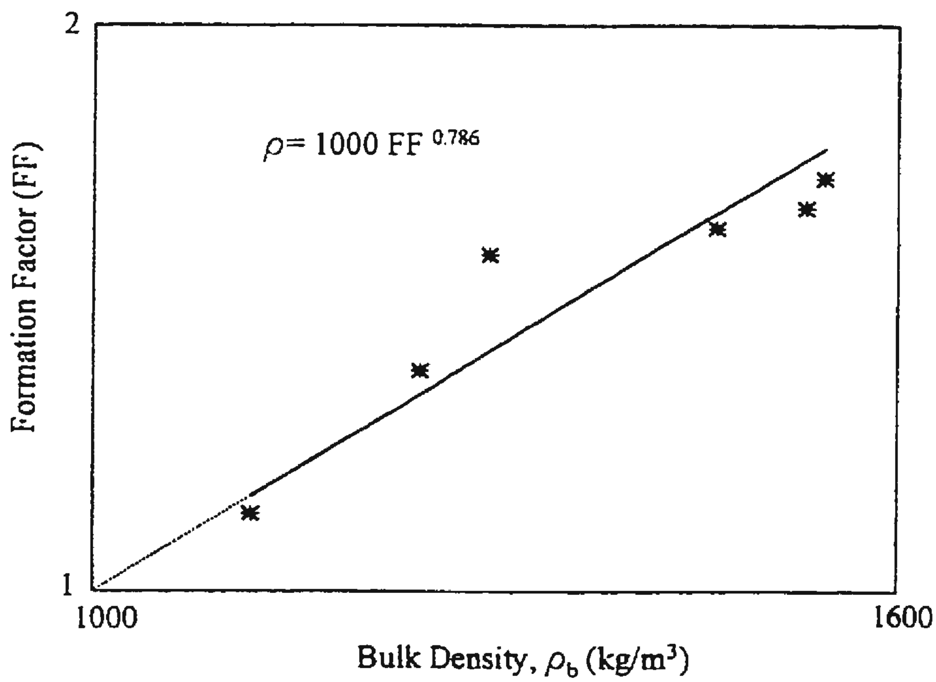


Figure 20 Calibration Plot of Bulk Density vs. Formation Factor for a Kaolin Slurry

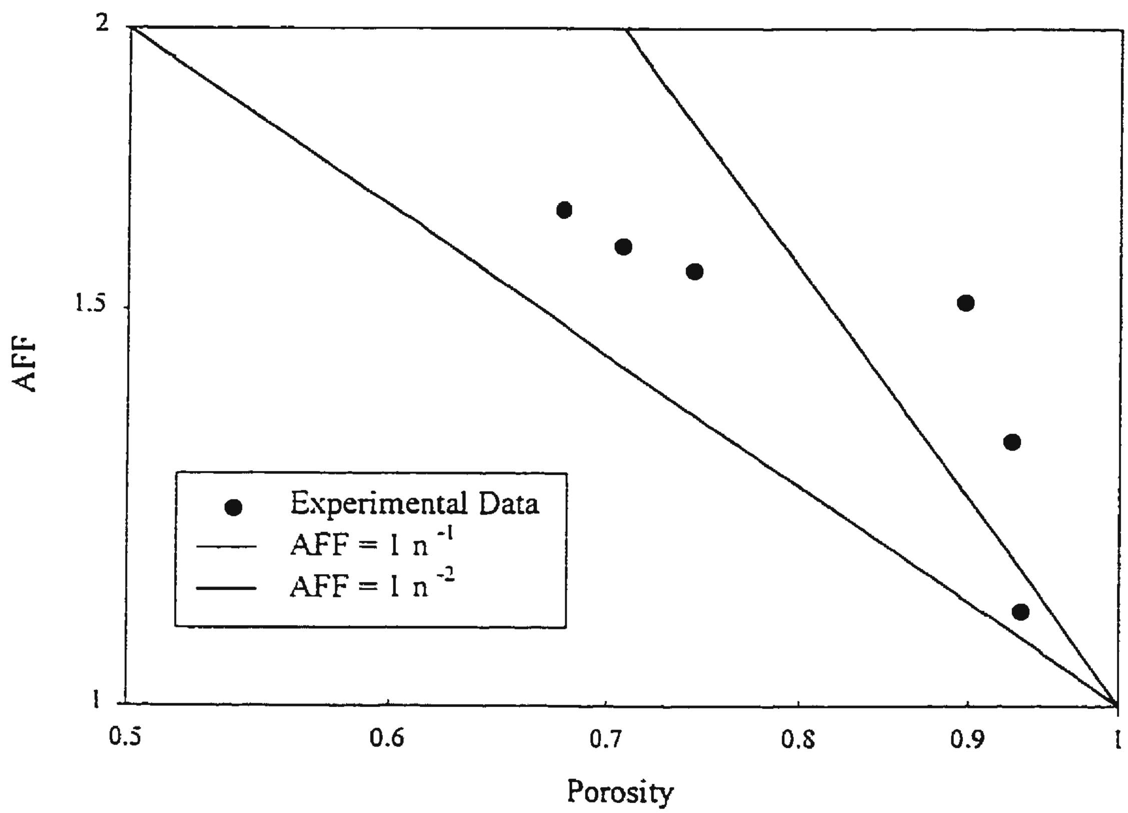


Figure 21 Porosity vs. Formation Factor

5.1.4 Discussion

The formation factors for this test ranged from a maximum of 1.66 to a minimum value of 1.10. These formation factor values are consistent with measurements made by Jackson *et al.* (1978) and McDermott (1992). There is a reasonable fit to the bulk density vs. formation factor data but if the results are plotted in terms of soil porosity vs. formation factor (Figure 21), the results are not consistent with published 1 g results (Jackson *et al.* (1978) and McDermott (1992)). Previous 1 g measurements indicate that if the log of formation factor is plotted as a function of soil porosity, a linear relationship should result. Upper and lower bounds to the data can be represented as shown on Figure 21. The centrifuge porosity

/formation factor data does not seem to follow a linear relationship and some measurements (approximately 30%) actually lie outside the upper bound.

Even though the formation factor values were consistent with previous measurements in cohesive sediments, there was significant variation observed in the electrical resistivity measurements. This variation can possibly be attributed to a gravity effect on the ABEM Terrameter during testing. The ABEM Terrameter was located in a specially designed mounting frame near the axis of rotation of the centrifuge and was subjected to a gravity level of about 20g during the centrifuge test. The variation in resistivity measurements may have also been caused by a change in soil temperature during the test. However, it was not possible to confirm this since the temperature measurement system had not yet been developed. Also, it was thought that the nature of the clay particles themselves (surface charges) may have made it difficult to accurately measure electrical resistivity.

Even though the data from this test was satisfactory in terms of formation factor vs. bulk density, it was decided that a different technique was required to measure bulk density in-flight for the following reasons:

1. The questionable porosity / formation factor results.
2. Significant variation in electrical resistivity measurements during centrifuge testing.
3. Subsequent testing (SCOL09-10) revealed that resistivity measurements were not repeatable.

5.2 Gamma Ray Density System Initial Calibration Test - Test Gamma2

5.2.1 General

In the development of the gamma ray system for in-flight density measurement, it was necessary to proof test and calibrate the system. A normal column test was usually conducted in the "large" 1 m high column. However, in order to test the operation of the gamma ray source and detector under the high gravity levels in the centrifuge, the first test was conducted using a smaller experimental column. This smaller column was easier to work with and minimized personnel exposure time to the gamma ray source.

5.2.2 Procedure

The column used for this test was a steel container, approximately 400 mm in height with an internal diameter of 300 mm. Two pieces of ABS pipe, each with a nominal diameter of 38 mm, a nominal wall thickness of 6 mm, and an approximate height of 425 mm were procured. The cesium source was enclosed within a lead containment vessel with dimensions of 25 mm diameter and 50 mm in height. A small (~3 mm diameter) epoxy filled port in the lead containment vessel allowed a collimated beam of gamma rays to be emitted. The top of the lead container was threaded so that a piece of threaded rod could be attached and used to handle the source. The source was lowered into the ABS pipe until it was located as close to the bottom of the tub as possible. The detector was attached to another length of threaded rod and lowered inside the second ABS pipe section to the same elevation as the source. The source and detector were located on opposite sides of the tub (i.e. 180° apart). A plastic clamping apparatus was used to secure the ABS pipe sections to the round tub. The entire apparatus was placed inside a C-CORE strongbox for centrifuge testing. Figure 22 presents the experimental setup for this calibration test.

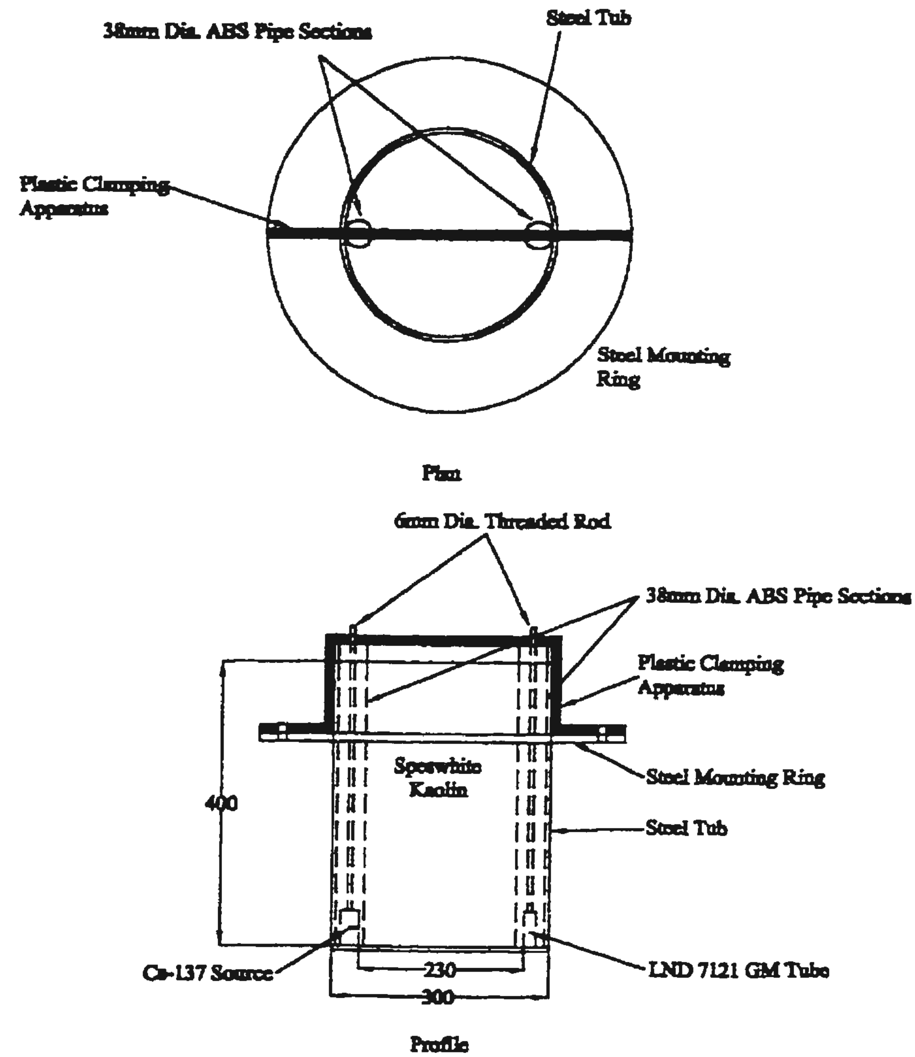


Figure 22 Experimental Setup - Centrifuge Test Gamma2

The test procedure consisted of first taking gamma ray attenuation measurements through air only. Measurements were taken at 1 g and at various g levels to ensure that the source and detector operated satisfactorily at accelerated gravity levels. Water was then added to the tub and the entire process repeated. After these baseline readings were recorded in air and water, the first kaolin slurry was placed inside the round tub and consolidated under 100

g. Counts were recorded after initial slurry deposition, upon achieving 100 g and approximately every five minutes until the end of the test. The following day, a second kaolin slurry with a different initial water content was placed inside the round tub and the measurements repeated as the clay consolidated in the centrifuge under 100 g.

5.2.3 Test Results

Table 4 presents results of the preliminary test of the gamma ray density measurement system.

Table 4 Gamma Ray Density System - Centrifuge Test Results

Medium	Gravity Level	Average # of Counts per min. (c/min.)	Standard Deviation (c/min.)	Std. Error in Mean (c/min.)	Comments
Air	1	12293	157	26.2	36 Readings
Air	10	12413	87	35.5	6 Readings
Air	20	12318	73	42.5	3 Readings
Air	40	12419	106	61.2	3 Readings
Air	60	12313	133	77.1	3 Readings
Air	80	12753	33.5	19.3	3 Readings
Air	100	12755	13.2	7.64	3 Readings
No Source	1	10	1.53	0.88	3 Readings
No Source	10	11	1.15	0.67	3 Readings
No Source	20	11	1.73	1.00	3 Readings
No Source	40	7	3.61	2.08	3 Readings
No Source	60	9	2.08	1.20	3 Readings
No Source	80	10	3.00	1.73	3 Readings

No Source	100	12	2.48	0.94	7 Readings
Water	1	8597	91.6	37.4	6 Readings
Water	10	8506	98.3	44.0	5 Readings
Water	20	8527	102	22.8	20 Readings
Water	40	8517	44.7	20.0	5 Readings
Water	60	8558	112	49.9	5 Readings
Water	80	8644	108	48.1	5 Readings
Water	100	8666	76.4	24.2	10 Readings
Kaolin #1	1	7469	63.7	20.1	10 Readings $w_i(\%) = 350$
Kaolin #1	100	7824	35.2	17.6	Avg. of first 4 readings
Kaolin #1	100	6018	51.1	25.6	Avg. of last 4 readings $w_f(\%) = 82$
Kaolin #2	1	6699	74.1	15.1	24 Readings $w_i(\%) = 145.6$
Kaolin #2	100	6800	63.3	31.7	Avg. of first 4 readings
Kaolin #2	100	5817	127	63.3	Avg. of last 4 readings $w_f(\%) = 73.1$
Kaolin #2	1	5702	94.1	29.8	10 Readings

To derive the calibration between density and count rate, the initial and final water contents for both kaolin slurries were converted to bulk density values. The corresponding average 100 g count rate was then determined for each density (water content) measurement. For example, the average 100 g count corresponding to the initial water content of the first soil (350%) was taken as the average of the first four count rates obtained after reaching test speed. These values were then normalized using the average 100g count through water. A relationship was then determined between bulk density and the natural log of the normalized count. The results are presented in Figure 23.

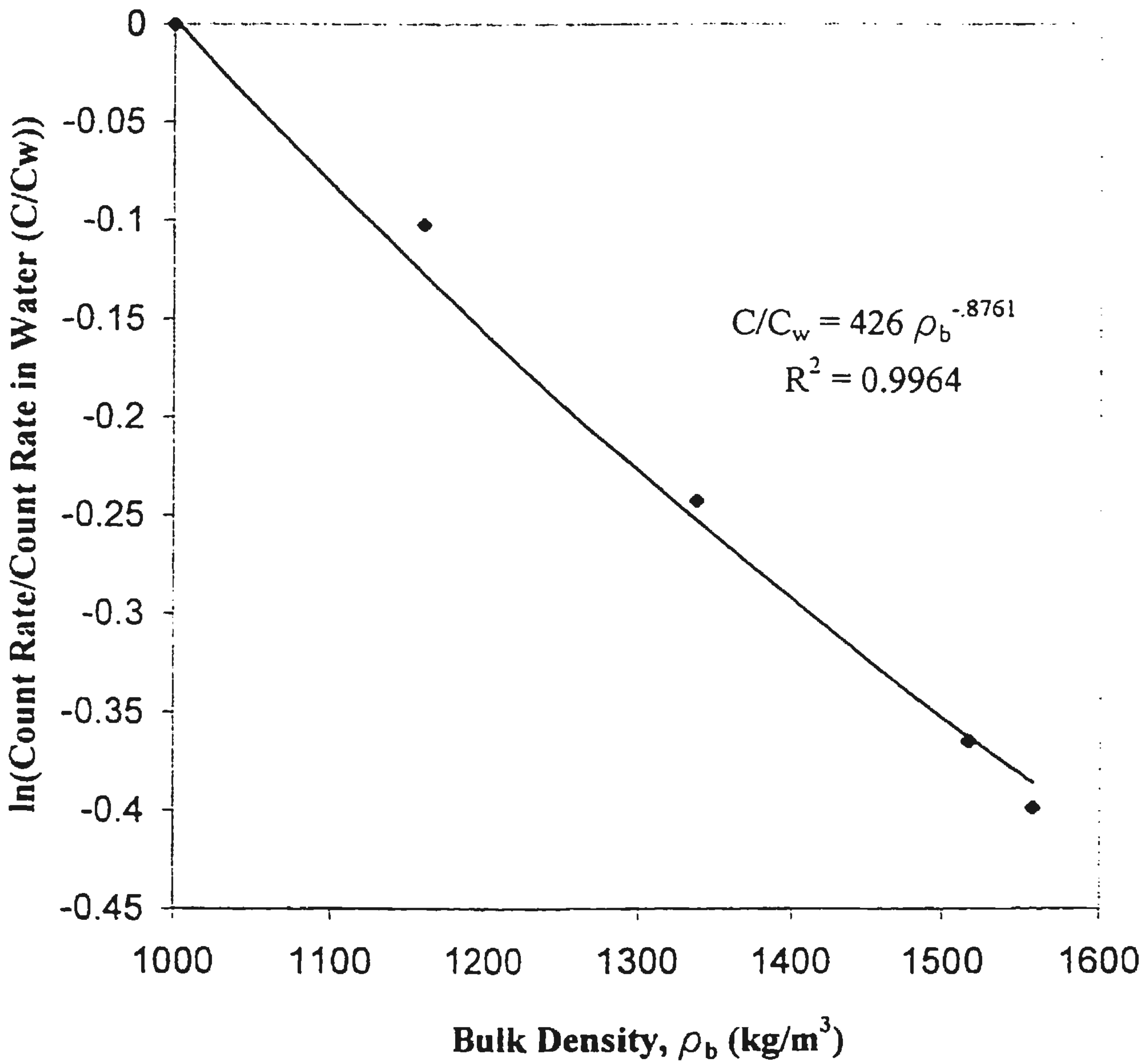


Figure 23 Bulk Density vs. ln (Normalized Count Rate) for Kaolin Clay

5.2.4 Discussion

One problem with this test was the one minute count duration used for count rate measurement, especially at the beginning of the test. Sedimentation/consolidation is more rapid initially and therefore a one minute count duration can result in a considerable change in soil properties between readings. This problem is compounded when one considers the fact that an average reading was taken over the first few minutes. Therefore, the average initial count determined after first reaching test speed is considered less reliable than the count determined towards the end of the test, when consolidation is occurring much more slowly. Even though the LUDLUM ratemeter could be set up for a variety of count durations, it was felt that too short of a duration would reduce the resolution of the system and a count duration that was too high would lead to excessive times to obtain a reading. In the future, it may be beneficial to more closely examine the effect of count duration on the calibration and use a lower count duration at the start of the test, when consolidation is occurring much more quickly.

An examination of the data in Table 4 for both air and water indicates that the 100 g count rate is slightly higher than the 1 g count rate. There was about a 1% increase in count rate in air and a 4% increase in water. This indicates a possible minor gravity effect on the source and/or detector. It is also possible that this variation in count rate was caused by a possible slight shift in the orientation between the source and GM detector.

6.0 SETTLING COLUMN TESTS TO DEVELOP GEOTECHNICAL-GEOPHYSICAL CORRELATIONS

In order to develop the column and the various systems which are used to measure excess pore pressure, compressional wave velocity and bulk density, a number of separate experiments had to be conducted as outlined in Table 5. Experiments SCOL01 - SCOL05 were conducted to develop the pore pressure, electrical resistivity and compressional wave velocity measurement systems. During test SCOL06, a soft soil (kaolin) was deposited at a low initial density (high water content/void ratio) and allowed to undergo sedimentation and self-weight consolidation under 100 g while pore pressure, electrical resistivity and compressional wave velocity were monitored. Experiment SCOL07 was a calibration test of the electrical resistivity system while SCOL08-10 were proof /system development tests. Centrifuge test SCOL11 was a calibration/systems test of the gamma ray density measurement system while test SCOL12 was another complete soft soil test, with the initial suspension density similar to the SCOL06 initial density, using the gamma ray system for density measurement. Excess pore pressure and compressional wave velocity were also measured during SCOL12. Tests SCOL06 and SCOL12 are the focus of the thesis since the initial densities were similar and pore pressure, compressional wave velocity and sediment density were measured continuously during each test.

Table 5 Key Soft Soil Centrifuge Experiments

Test	Date	G Level	Comments
SCOL01	Jan. 5/95	125	Containment Proof Test - Column filled with water, no holes through wall
SCOL02	Jan. 30/95	125	Containment Proof Test - Column with two compressional wave transducer holes through wall, filled with water
SCOL03	June 27/95	125	Proof Test of on arm data acquisition system, 4 PPT's installed, column filled with 60%-40% kaolin silt
SCOL04	Oct. 18/95	100	Test of P-wave transducers before and after spinning, Kaolin ~ ρ_{bi} = 1110 kg/m^3
SCOL05	Feb. 7/96	125	Proof Test of Terrameter Frame, Measured Elec. Resistivity and P-wave Velocity in flight, column filled with water
SCOL06	Feb. 12/96	100	Excess pore pressure dissipation, P-waves and Elec res., column filled with kaolin ~ ρ_{bi} = 1110 kg/m^3
SCOL07	Feb. 28/96	100	Calibration test of elec. res. system, ρ_{bi} = 1097 kg/m^3
SCOL08	April 9/96	125	Column proof test with all P-wave transducer holes incorporated, filled with water
SCOL09	April 10/96	100	P-wave, S-wave, Elec. Res. in soft kaolin slurry
SCOL10	June 19/96	100	Multiplexer system check, in-flight meas. of pore pressure dissipation, P-wave velocity and electrical resistivity
Gamma2	Nov. 19/97	100	Small tub test to determine if Gamma rays could be detected at 100 g through kaolin, tub height = 400 mm, sample thickness = 300 mm
SCOL11	April 7/98	125	Proof test of gamma ray apparatus, column filled with water
SCOL12	April 8/98	100	P-wave, Gamma ray and excess pore pressure dissipation, ρ_{bi} = 1110 kg/m^3

6.1 Test SCOL06

6.1.1 General

The first measurement of P-wave speed through a soft soil consolidating under 100 g in the C-CORE centrifuge occurred on February 12, 1996. At this time, the electrical resistivity system was being used to determine in flight density during consolidation. This test was designated as SCOL06 (Settling Column Test #6).

6.1.2 Procedure

The kaolin slurry was initially mixed under a vacuum of approximately 60 kPa overnight to an initial water content of 536.7% (ρ_b = 1108 kg/m^3). After mixing, the slurry was transported to the centrifuge chamber and deposited into the column in a manner which minimized air entrainment. The initial slurry height in the column was 915 mm. Time zero (t=0) was taken as the time the slurry was placed in the column. Centrifuge rotation commenced at t = 32 minutes and test speed (100 g) was reached at t = 38 minutes. Once at test speed, compressional wave velocities were recorded at regular intervals and the electrical resistivity was continuously monitored. The test was terminated after t = 385 minutes.

6.1.3 Test SCOL06 Results

The compressional wave speeds, as expected, were observed to slightly decrease at first and then gradually increase. This is similar to behaviour observed by other researchers, such as McDermott (1992), Urick (1947) and Hampton (1967) and predicted by the Wood's equation. Table 6 lists the various compressional wave speeds and the times at which they were measured along with the corresponding soil electrical resistivity. Due to experimental limitations, the electrical resistivity was not measured at exactly the same time as compressional wave speed. Therefore, the values in Table 6 are the electrical resistivity readings recorded at the times closest to the compressional wave measurement time. The final slurry height was measured as 175 mm. This translates to total sedimentation/consolidation settlement of 740 mm. Water contents of 152%, 79% and 72% were measured at the surface, middle and bottom of the consolidated layer respectively. Figure 24 shows the dissipation of excess pore pressure during consolidation for SCOL06.

Table 6 Experimental Results - Test SCOL06

Model Consolidation Time (minutes)	**P-Wave Speed (m/s)**	**Soil Electrical Resistivity (Ω)**	**Formation Factor (FF)**	**Bulk Density (kg/m³)**
0 (1 g reading)	1471.4	397	1.16	1120.5
33	1473.1	428.5	1.25	1189.8
43	1443.7	505	1.47	1353.8
55	1443.7	553.5	1.61	1455.0
63	1459.1	551	1.60	1449.8
75	1474.9	539.5	1.57	1426.0
86	1457.4	540	1.57	1427.0
110	1460.9	542.5	1.58	1432.2
120	1466.1	544	1.58	1435.3
141	1466.1	547	1.59	1441.5

169	1476.6	552	1.61	1451.9
185	1473.1	553.5	1.61	1455.0
224	1481.9	557	1.62	1462.2
267	1481.9	558	1.62	1464.3
292	1489.1	558	1.62	1464.3
347	1489.1	564	1.64	1476.61
385	1494.5	576	1.68	1501.3

In Table 6, the formation factors were calculated by dividing the measured soil electrical resistivity by the resistivity of the pore water, which was measured to be 343.5 Ω. Bulk density values were determined using the electrical resistivity calibration equation (Figure 20).

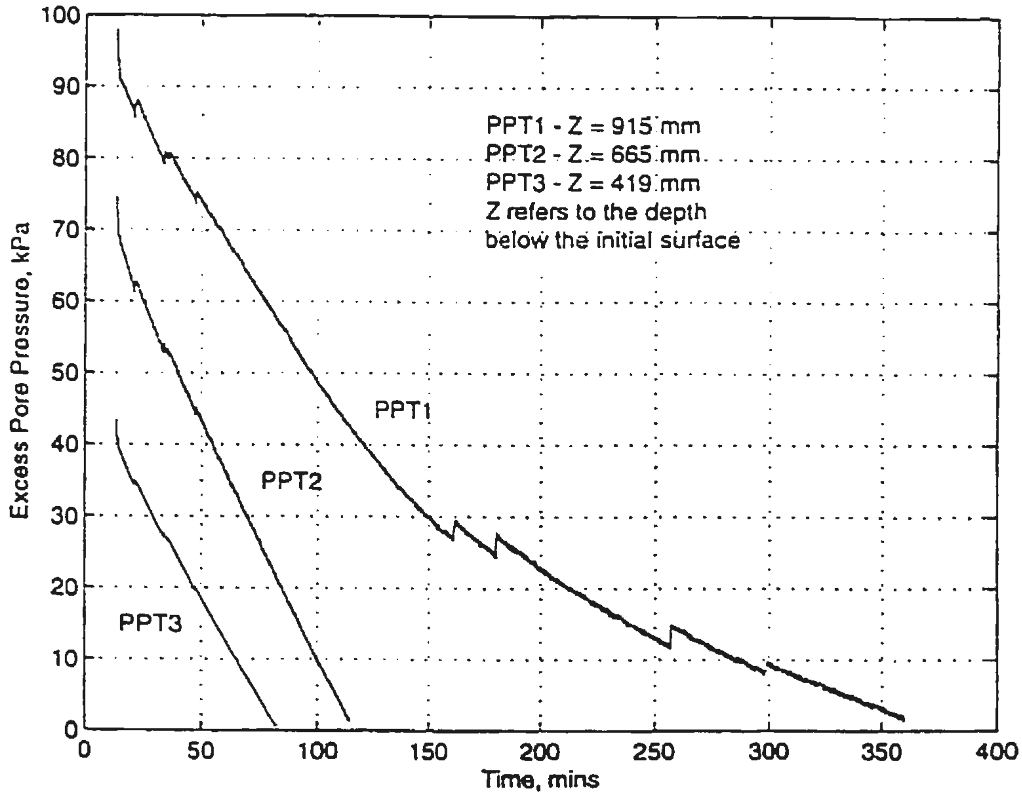


Figure 24 Excess Pore Pressure Dissipation - SCOL06
(Z refers to the depth below the initial soil surface)

For PPT's 2 and 3, the excess pore pressures shown in Figure 24 were derived by subtracting the observed hydrostatic pressures from the total pore water pressure. For PPT 1, the excess pore pressure was derived by subtracting the total pressure at about t = 360 minutes (i.e. the pressure at the end of the test). In other words, it was assumed that at the end of the test the soil was at least 90% consolidated. Table 7 compares the observed and calculated initial and hydrostatic pressures.

Table 7 Comparison of Observed vs. Calculated Pressures

PPT #	Calculated Initial Pressure (kPa)	Observed Initial Pressure (kPa)	Calculated Hydrostatic Pressure (kPa)	Observed Hydrostatic Pressure (kPa)
1	995	988	898	892
2	723	690	652	617
3	455	415	411	373

Figure 25 shows a diagram of a typical P-wave generated at 100 g through the consolidating kaolin slurry in the C-CORE geotechnical centrifuge. For this test, each in flight P-wave signal was obtained using a 150 times signal averaging procedure.

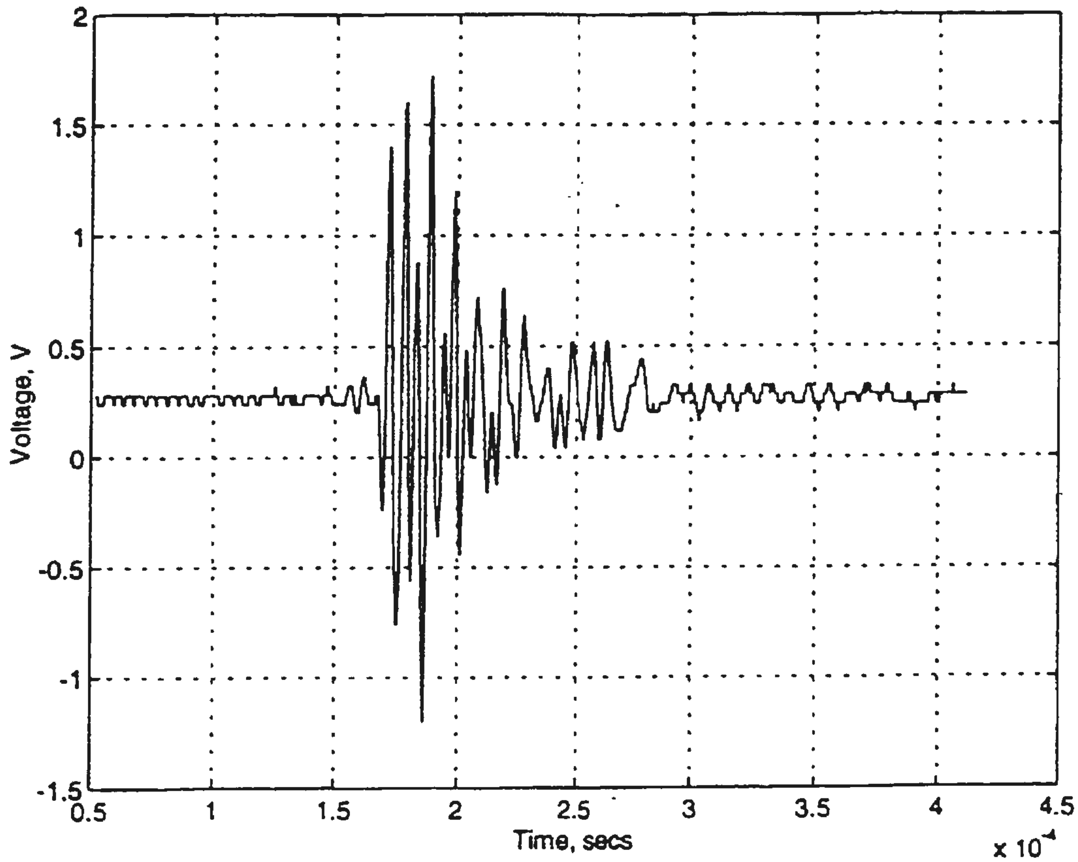


Figure 25 Compressional Wave Signal Through Consolidating Kaolin Clay

6.1.4 Discussion

The electrical resistivity measurement system was affected by the increased gravity level. This resulted in a decrease of 26 Ω in measured resistance values at 100 g as compared to those measured at 1 g. Since it was one of the objectives of this experiment to compare P-wave velocity/bulk density data obtained in the centrifuge with that of other authors obtained at 1 g, all resistance measurements made in the centrifuge at 100 g were corrected by a value of + 26 Ω. After correcting resistance measurements, the calculated accuracy in bulk density measurement was ± 2.5 kg/m^3.

The relationship between bulk density of the consolidating soft soil and compressional wave velocity is shown in Figure 26. The measured compressional wave velocity in the sediment has been normalized with the P-wave velocity in freshwater at 20° C, which was calculated to be 1481.6 m/s using the Clay and Medwin (1977) equation. The equation of the best fit line through the SCOL06 data is given by the following quadratic relationship:

$$V_{pn} = (3 \times 10^{-7})\,\rho_b^{\,2} - (8 \times 10^{-4})\,\rho_b + 1.451 \qquad [13]$$

Figure 27 presents a comparison of the centrifuge data with some 1g settling column data from Urick and Hampton (from Ogushwitz (1985)). These data were obtained at 1g through kaolinite suspensions.

There is a limited number of data points in the initial portion of the curve due to the fact that this was the first complete test and the experimental technique had not yet been fully developed. The soil consolidated much faster that had been initially expected.

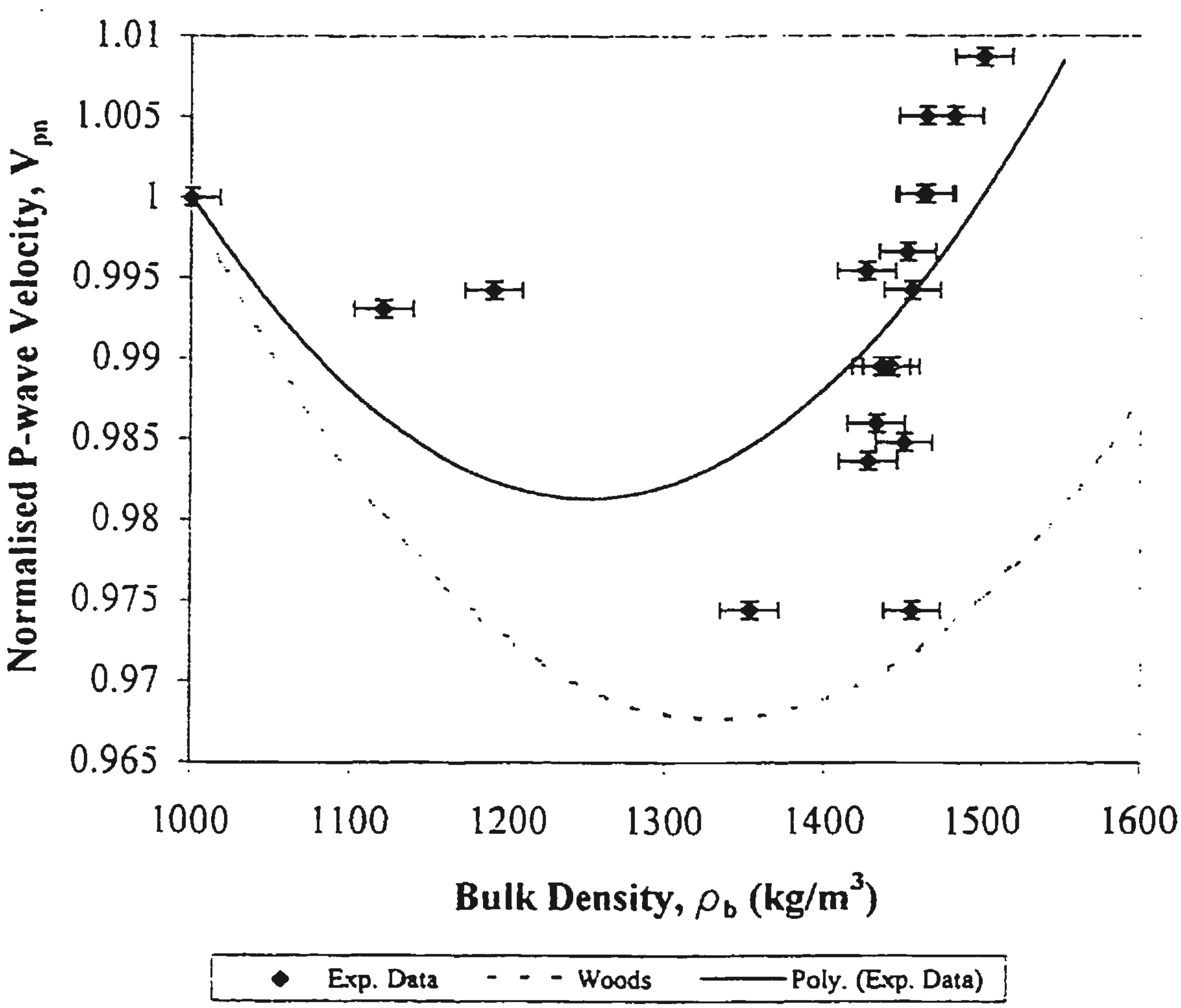


Figure 26 Bulk Density vs. Normalised Compressional Wave Velocity - SCOL06

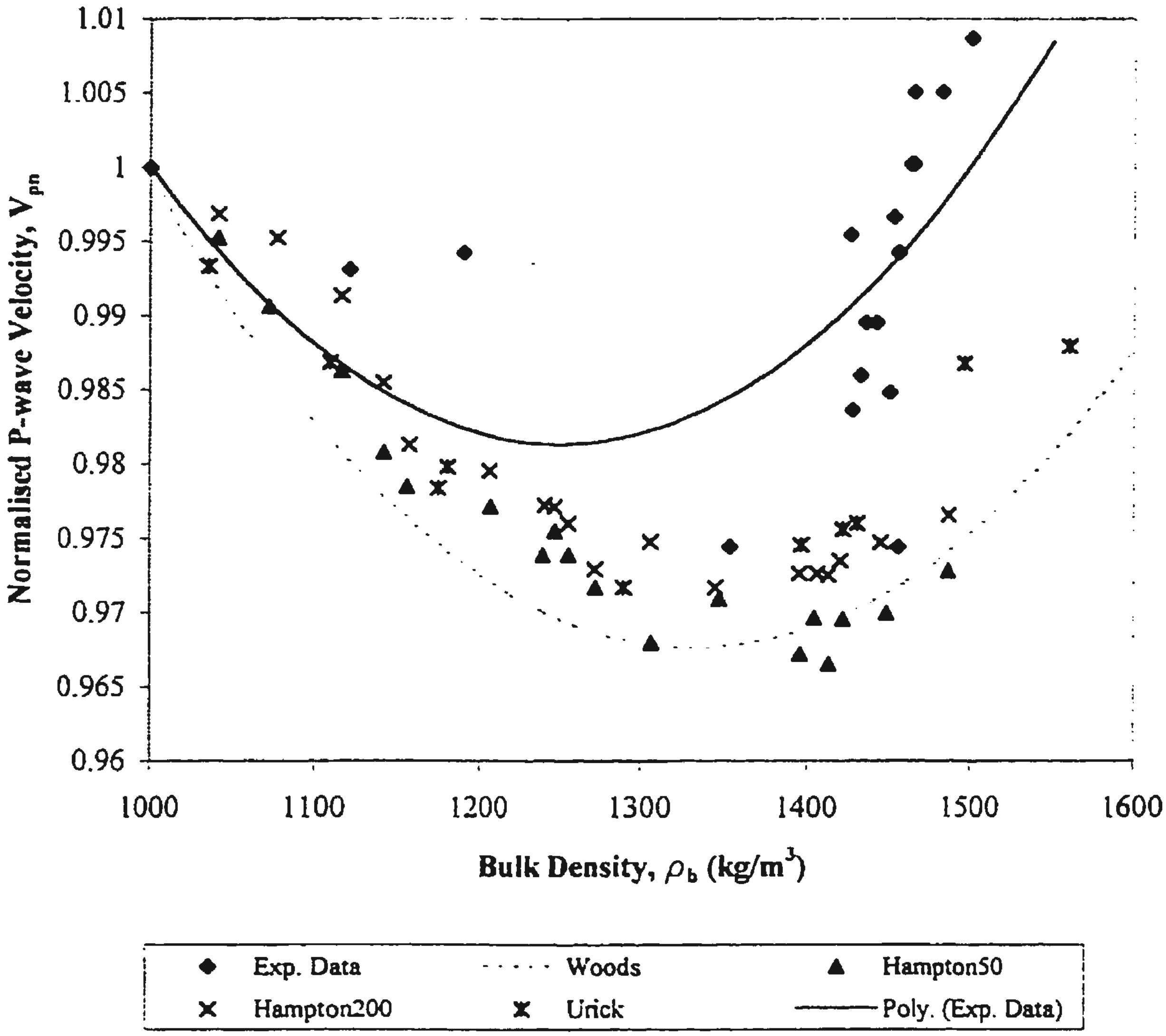


Figure 27 Comparison Between Centrifuge and 1g Column Data

The 1 g kaolinite data demonstrate that a distinct minimum exists in the normalized compressional wave velocity vs. bulk density plot for kaolin clay between $\rho = 1300$ kg/m^3 and $\rho = 1400$ kg/m^3. A comparison of the data from Urick and Hampton (from Ogushwitz (1985)) and the data from the centrifuge test reveals that the minimum measured normalized P-wave velocity occurs at approximately the same value of bulk density. Figures 26 and 27 reveal another interesting point about the stages of soil formation. At low values of bulk density ($< \sim 1200$ kg/m^3) the data are reasonably close to Wood's equation for a soil suspension. However, at higher densities, the data deviates from Wood's equation due to the effect of the soil shear modulus (G). This was expected and indicates that the soil slurry is no longer a suspension but is a consolidating soil with a stiff soil fabric. Also, the value of normalized compressional wave velocity obtained by either Hampton (1967) or Urick (1947) did not exceed unity indicating that the P-wave velocity in the sediment did not increase beyond the value of that in water. However, the normalised compressional wave velocities obtained in the centrifuge increased slightly beyond 1 towards the end of the test. Hampton (1967) conducted experiments at both 50 kHz and 200 kHz while Urick (1947) conducted his experiments at 1 MHz. Compressional wave velocities from the centrifuge test were obtained using 250 kHz P-wave transducers. The centrifuge data may be expected to fall between Urick's (1947) 1 MHz data and Hampton's (1967) 200 kHz data. This was not the case. Compressional wave velocities obtained in the centrifuge were higher than Urick's (1947) 1 MHz data. This may be due the fact that shear modulus depends strongly on effective stress levels (Stoll, 1989). As a result, the shear modulus of the soil in the centrifuge experiment can be expected to be higher than the shear modulus of the same soil consolidated under 1 g conditions. An increase in shear modulus would result in a higher compressional wave velocity.

There is a lack of density / compressional wave velocity data between 1000-1350 kg/m^3. This is due primarily to the fact that the acceleration level was 100 g, sedimentation occurred very quickly and the soil was formed sooner than expected.

Wood's equation represents a lower bound to the normalised compressional wave velocity/bulk density data. However, some of Hampton's 50 kHz data as well as some of the centrifuge data were very close to or below the line represented by Wood's equation. This is due to some scatter in the experimental data and the fact that there are different values reported in the literature for the compressibility of kaolin soil grains (C_s). Wood's equation will predict a slightly different lower bound depending on which value of grain compressibility is used. The value of compressibility used for purposes of this research was 1.0×10^{-12} cm^2/dyne (Shumway, 1958).

The final water content at the bottom of the column was measured to be about 72% after the test which corresponds to a final void ratio (e) of 1.89, assuming 100% saturation. A comparison can be made between the final void ratio (calculated from the final water content at the bottom of the column) to the predicted final void ratio using Al Tabba's (1987) relationship (Table 2) which is based on one dimensional consolidation tests on speswhite kaolin clay in a typical oedometer cell. Since the bulk density at the end of the test was about 1500 kg/m^3 (calculated value), the vertical effective stress was about 84 kPa. Assuming K_{onc} = 0.64 (Table 2), the mean effective stress (p') was calculated to be about 64 kPa. Based on this value of mean effective stress at the end of the centrifuge test, the predicted final void ratio was 1.35 which is about 29% lower than the void ratio corresponding to the measured final water content. Based on this result, it is possible that the soil did not reach 100% (or even 90%) primary consolidation after about 6 hours of sedimentation / consolidation at 100 g in the centrifuge.

The pore pressure transducer outputs indicated a different type of behaviour for PPT 1 than for PPT's 2 and 3. For PPT's 2 and 3 a linear decrease in pressure was observed, possibly indicating sedimentation, while for PPT 1 the data seemed to indicate that consolidation was occurring at the bottom of the soil column. To investigate this further, the soil bulk density was back-calculated using the known PPT positions (2 and 3) and the time at which hydrostatic pressure was reached. For PPT 3, hydrostatic pressure was achieved at about t = 82 minutes. At this time, the excess pore pressure at PPT 2 was about 20 kPa and the height of soil above PPT 2 was assumed to be equivalent to the separation distance between PPT 2 and 3. The calculated soil bulk density was about 1083 kg/m^3. A similar calculation conducted using the time at which PPT 2 reached hydrostatic conditions (114 minutes), the equivalent excess pore pressure at PPT 1 (40 kPa) and the separation distance between PPT 1 and 2 (250 mm) resulted in an average density of 1163 kg/m^3. Both of these calculated densities are close to the initial density of 1108 kg/m^3 indicating that sedimentation only occurred in the area of PPT 2 and 3.

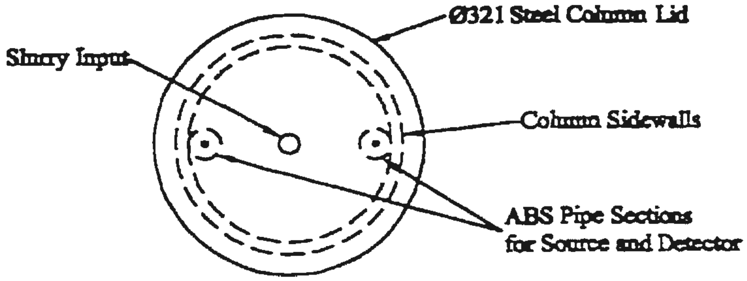
6.2 Test SCOL12

6.2.1 General

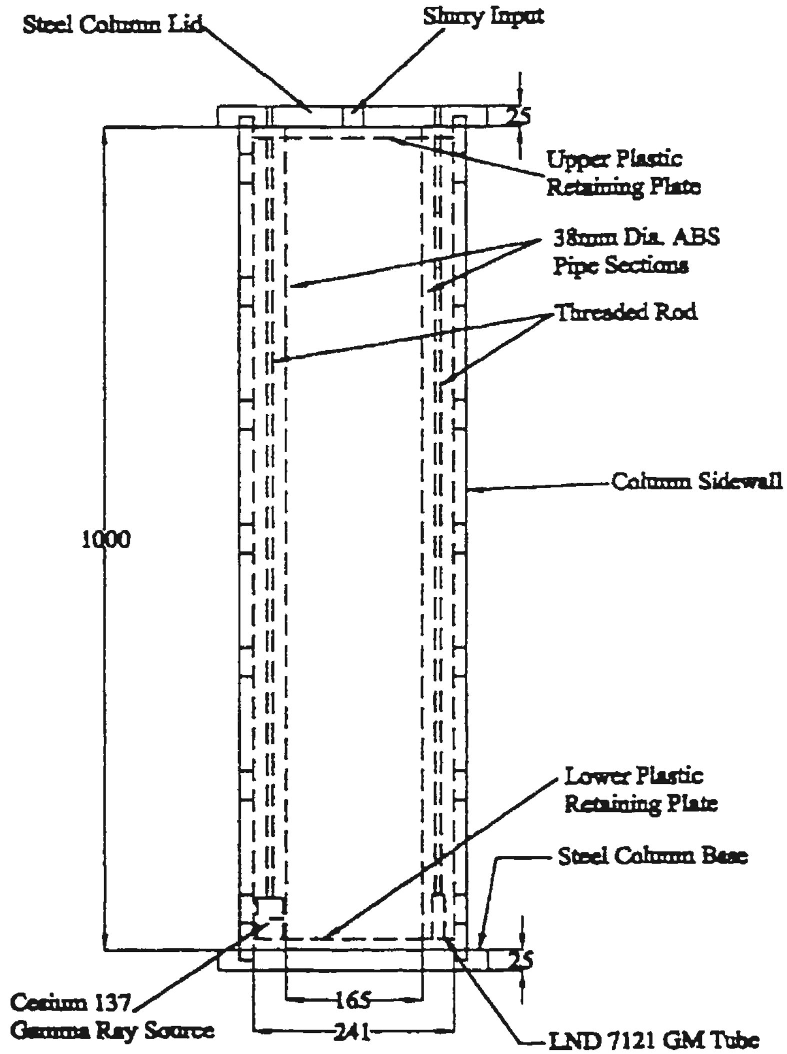
Test SCOL12 was designed to measure compressional wave velocity, excess pore pressure dissipation, soil temperature and bulk density using a gamma ray attenuation technique. This test actually consisted of two different phases. Phase 1 was a water test which was conducted at 100g to obtain baseline measurements. Phase 2 was the actual consolidation test.

6.2.2 Procedure - SCOL12 Phase 1

A similar technique was adopted to install the Cesium 137 source and detector in the 1m high column as that used for the small tub calibration test. A 13 mm thick plastic base plate was first placed on the inside bottom surface of the column. One end of each 980 mm long ABS pipe section was sealed with an end cap and the pipe sections were then placed inside the column 180° apart. The end caps used to seal both pipe sections were equipped with a small stud that passed through the plastic base plate and thereby helped to keep the pipe sections in position inside the column. A second plastic retaining plate, with an outer diameter equal to the inside diameter of the column, was placed over the top of each pipe section to ensure that the pipes remained straight. Figure 28 is a schematic showing the various column components for this test.



Plan



Profile

Figure 28 Column Components for Test SCOL12

The first phase of this test involved determining 100g count rates and compressional wave velocities in the column through water to obtain baseline readings. After installation of the pipe sections and associated retaining plates, the column was filled with water to a height of about 877 mm and the steel column lid, which was equipped with two 25 mm diameter holes, was attached and secured with the four steel tie rods. The gamma ray source and detector were then installed 180° apart in the column using the same technique as that outlined for the small tub test. The gamma ray instrumentation was installed such that measurements were taken at the same elevation as the P-wave measurements but oriented 90° apart. The column was also instrumented with six pore pressure transducers and one temperature probe for this test. A schematic of the experimental setup is shown in Figure 29. The experimental setup is also shown in the photographs of Figure 30. The test was then started and 100 g was achieved at the level of the compressional wave/gamma ray instrumentation after about 10 minutes.

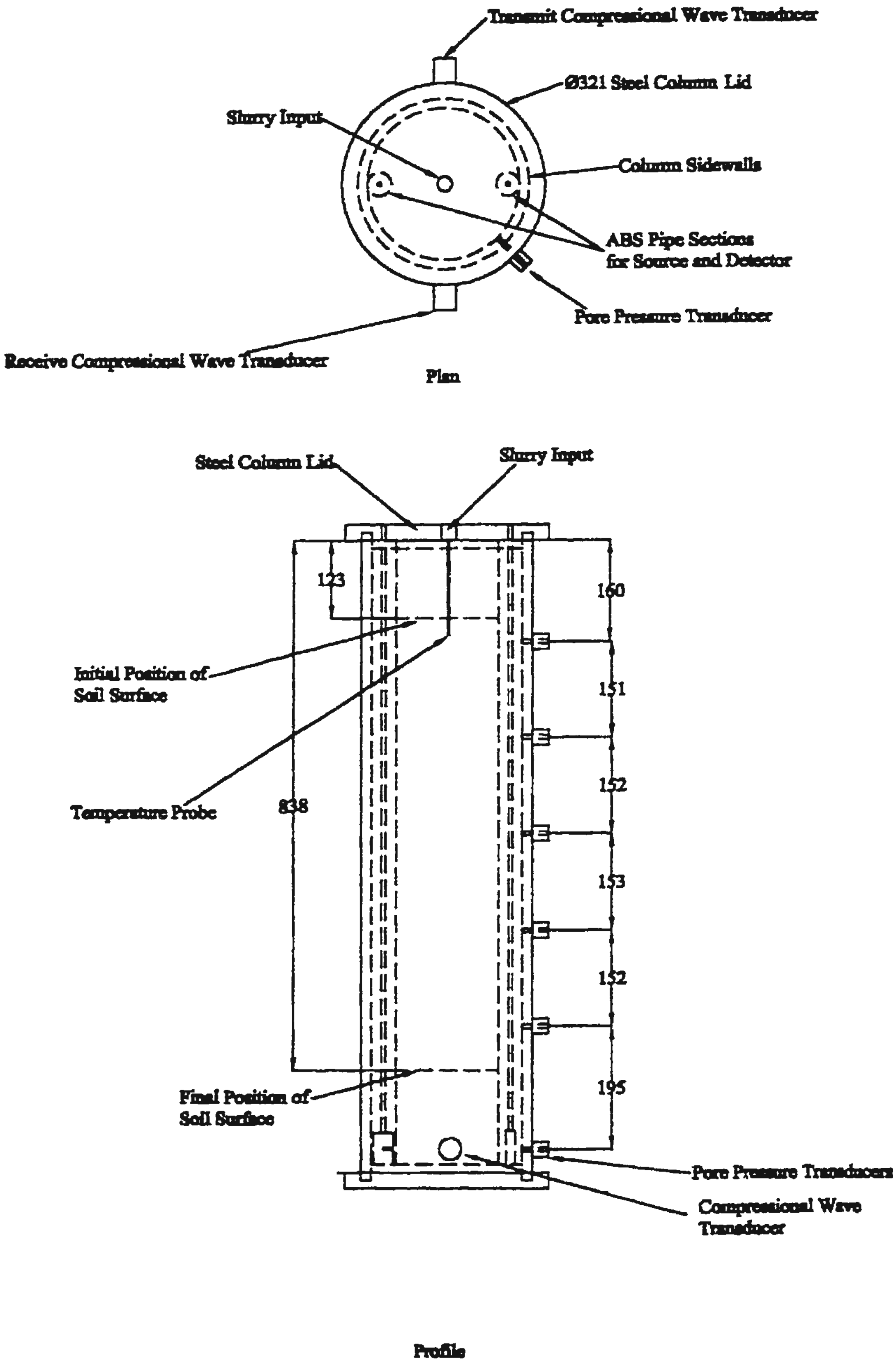


Figure 29 Schematic of Experimental Setup - SCOL12

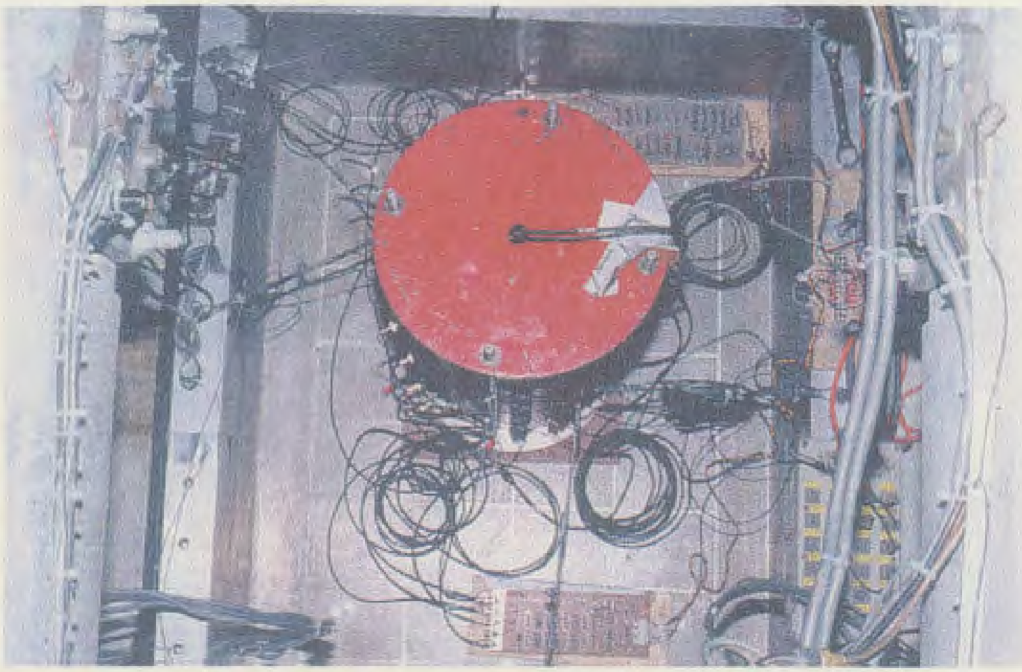


Figure 30a Experimental Setup - SCOL 12 (Plan)

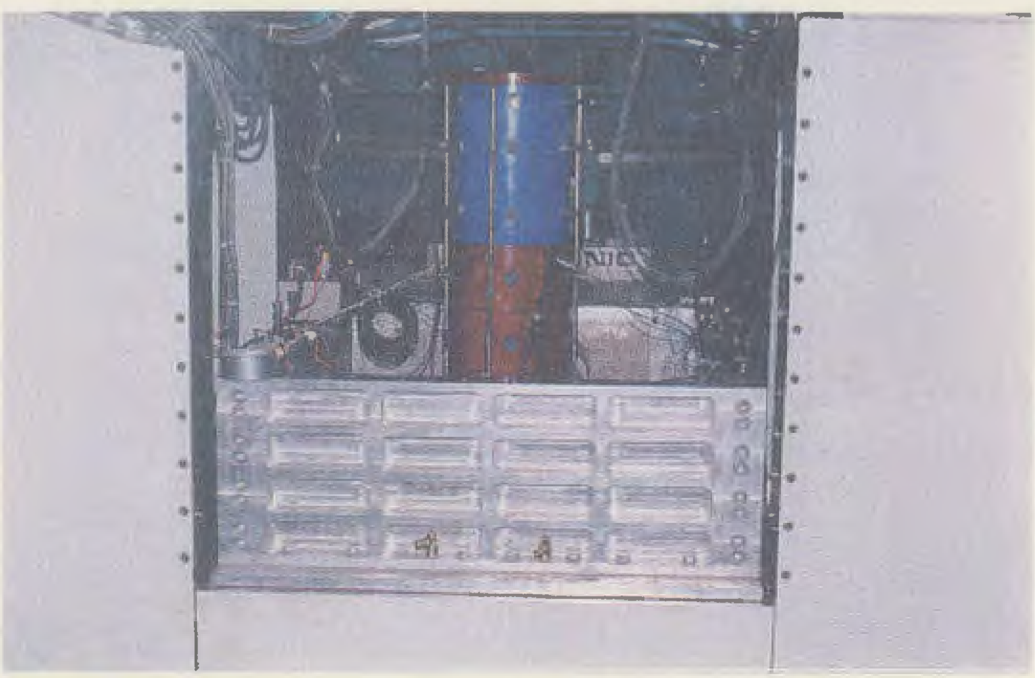


Figure 30b Experimental Setup - SCOL12 (Profile)

6.2.3 Results - SCOL12 Phase 1

Table 8 and 9 present count rates and compressional wave velocity data respectively, for the first phase of the experiment.

Table 8 Count Rates Through Water - SCOl12

Elapsed Time (minutes)	Counts/minute	Comments
0	15064	1g / 1 min duration
1	15241	1g / 1 min duration
2	15158	1g / 1 min duration
5	15153	10g / 1 min Duration
6	15172	10g / 1 min Duration
7	14960	10g / 1 min Duration
14	14580	100g / 1 min Duration
15	14457	100g / 1 min Duration
16	14569	100g / 1 min Duration
17	14433	100g / 1 min Duration
18	14518	100g / 1 min Duration
20	14615	100g / 1 min Duration
22	14501	100g / 1 min Duration
23	14641	100g / 1 min Duration
25	14668	100g / 1 min Duration
26	14580	100g / 1 min Duration
30	14524	100g / 1 min Duration
33	14699	100g / 1 min Duration
35	14387	100g / 1 min Duration

38	14453	100g / 1 min Duration
39	14563	100g / 1 min Duration
50	15132	1g / 1 min Duration
51	15026	1g / 1 min Duration
53	15095	1g / 1 min Duration

Table 9 Compressional Wave Data Through Water - SCOL12

Elapsed Time (minutes)	P-Wave Arrival Time (μs)	P-Wave Velocity (m/s)	Acceleration Level (g)
0	176.0	1371.0	1
5	174.8	1380.4	10
14	175.1	1378.1	100
19	175.2	1377.3	100
24	175.2	1377.3	100
29	175.1	1378.1	100
41	175.3	1376.5	100

Note: The expected velocity through water was about 1481 m/s. The values in Table 9 are not corrected for temperature, pressure or system delays.

6.2.4 Discussion - SCOL12 Phase 1

The average 1g count rate through water prior to testing was 15154. The average 100g count rate was 14546 while after spinning the average 1g count rate was 15084 which is slightly lower than the pre-test count rate. The average 100g count rate was about 4% lower than the average 1g count. The lower count rate at 100g can possibly be attributed to a gravity effect on the detector and/or source.

The average **uncorrected** compressional wave velocity was measured to be 1377.0 m/s. This velocity was determined by dividing the separation distance of the P-wave transducers inside the column (241.3 mm) by the arrival time of the P-wave signal.

6.2.5 Procedure - SCOL12 Phase 2

Following completion of the first phase, the water was removed from the column and replaced with a kaolin slurry which had been mixed overnight to an average initial water content of about 580%, which is equivalent to a bulk density of about 1100 kg/m^3. The initial height of the slurry in the column was 877 mm. Time zero (t = 0) was taken as the time of slurry deposition into the column. After slurry deposition, baseline readings of count rates and compressional wave velocity were taken at 1g and then the column was accelerated to 100g. Once at test speed, measurements were taken on a regular basis to determine the compressional wave velocity and gamma ray attenuation during consolidation. The excess pore pressure dissipation and soil temperature was continuously monitored using the centrifuge on-arm data acquisition system. The test was terminated after t = 302 minutes. However, at t = 165 minutes, a problem developed with the P-wave electronics and it was not possible to generate P-waves after this time.

6.2.6 Results - SCOL12 Phase 2

Tables 10 and 11present count rates and compressional wave velocity data respectively for the second phase of this experiment. This data is also presented in Figures 31 and 32. The arrival time of each compressional wave signal was divided by the transducer separation distance inside the column (241.3 mm) thereby giving the raw compressional wave velocity. The results of the post-test water content measurements are given in Table 12. Following completion of the test, the position of the soil surface was 838 mm from the top of the column. This translates to about 715 mm of sedimentation/consolidation settlement (final sample thickness = 162 mm). Small vertical drainage channels were also observed to have developed in the consolidated soil layer and extended to the soil surface. These drainage channels were attributed to "piping". The change in sediment temperature observed during the experiment is presented in Figure 33 while Figure 34 presents data from the 6 pore pressure transducers. Figure 35 also presents excess pore pressure data but in the format typically used to present excess pore pressure data from 1 g column experiments (McDermott, 1992).

Table 10 Count Rates Through Consolidating Kaolin Clay - SCOL12

Elapsed Time (minutes)	Counts/minute	Comments
0	14686	1g
2	14645	1g
7	14750	1g
10	14511	During Spinup
11	14242	During Spinup
12	14314	100 g / 1 min. duration
13	14337	100 g / 1 min. duration
14	14281	100 g / 1 min. duration
15	14158	100 g / 1 min. duration

16	14456	100 g / 1 min. duration
17	14099	100 g / 1 min. duration
18	14410	100 g / 1 min. duration
19	14196	100 g / 1 min. duration
20	13951	100 g / 1 min. duration
21	13611	100 g / 1 min. duration
22	13505	100 g / 1 min. duration
24	13197	100 g / 1 min. duration
25	12976	100 g / 1 min. duration
26	12666	100 g / 1 min. duration
27	12465	100 g / 1 min. duration
28	12442	100 g / 1 min. duration
29	12165	100 g / 1 min. duration
30	12202	100 g / 1 min. duration
31	12028	100 g / 1 min. duration
32	11961	100 g / 1 min. duration
33	11917	100 g / 1 min. duration
34	12057	100 g / 1 min. duration
35	11955	100 g / 1 min. duration
36	11797	100 g / 1 min. duration
37	11691	100 g / 1 min. duration
38	11651	100 g / 1 min. duration
39	11771	100 g / 1 min. duration
40	11630	100 g / 1 min. duration
41	11704	100 g / 1 min. duration
42	11655	100 g / 1 min. duration

43	11535	100 g / 1 min. duration
44	11468	100 g / 1 min. duration
45	11647	100 g / 1 min. duration
47	11580	100 g / 1 min. duration
48	11482	100 g / 1 min. duration
49	11557	100 g / 1 min. duration
52	11534	100 g / 1 min. duration
54	11322	100 g / 1 min. duration
55	11469	100 g / 1 min. duration
56	11531	100 g / 1 min. duration
58	11511	100 g / 2 min. duration
60	11098	100 g / 2 min. duration
64	11489	100 g / 2 min. duration
69	11473	100 g / 2 min. duration
73	11439	100 g / 3 min. duration
84	11372	100 g / 3 min. duration
92	11358	100 g / 3 min. duration
97	11338	100 g / 3 min. duration
100	11367	100 g / 3 min. duration
108	11171	100 g / 3 min. duration
114	11222	100 g / 3 min. duration
124	11170	100 g / 3 min. duration
130	11323	100 g / 3 min. duration
137	11290	100 g / 5 min. duration
149	11331	100 g / 5 min. duration
160	11255	100 g / 5 min. duration

168	11313	100 g / 5 min. duration
176	11267	100 g / 5 min. duration
191	11224	100 g / 5 min. duration
198	11254	100 g / 5 min. duration
205	11196	100 g / 5 min. duration
228	11275	100 g / 5 min. duration
251	11235	100 g / 5 min. duration
276	11220	100 g / 5 min. duration
281	11174	100 g / 5 min. duration
283	11020	100 g / 1 min. duration
284	11189	100 g / 1 min. duration
286	11080	100 g / 1 min. duration
287	11180	100 g / 1 min. duration
298	11691	1 g
299	11446	1 g
300	11577	1 g
301	11387	1 g
302	11342	1 g

Table 11 Compressional Wave Velocities Through Kaolin Clay - SCOL12

Elapsed Time (minutes)	**P-Wave Arrival Time (μs)**	**Uncorrected P-Wave Velocity (m/s)**	**Corrected P-Wave Velocity (m/s)**	**Acceleration Level (g)**
0	176.5	1367.14	1468.30	1
2	177.66	1358.21	1459.40	1
4	177.84	1356.84	1458.00	1

7	177.92	1356.23	1456.20	1
12	177.84	1356.84	1456.80	1
16	177.84	1356.84	1456.80	1
18	176.96	1363.59	1463.60	10
20	175.92	1371.65	1471.60	10-100
22	176.18	1369.62	1466.50	100
24	176.62	1366.21	1465.30	100
26	178.44	1352.28	1452.30	100
28	178.28	1353.49	1450.90	100
30	178.6	1351.06	1452.00	100
32	178.52	1351.67	1469.80	100
34	178.3	1353.34	1453.90	100
36	178.18	1354.25	1455.00	100
38	177.84	1356.84	1457.40	100
41	177.46	1359.74	1460.30	100
43	177.42	1360.05	1460.50	100
45	177.26	1361.28	1461.70	100
47	177.22	1361.58	1462.00	100
49	177.1	1362.51	1462.90	100
51	176.82	1364.66	1465.00	100
53	176.14	1369.93	1470.20	100
55	176.62	1366.21	1466.50	100
57	176.44	1367.60	1467.80	100
59	176.36	1368.22	1468.30	100
61	175.64	1373.83	1473.90	100
63	175.7	1373.36	1473.30	100

65	175.98	1371.18	1471.10	100
67	176.08	1370.40	1470.30	100
75	175.46	1375.24	1474.90	100
79	175.0	1378.86	1478.40	100
84	175.0	1378.86	1478.00	100
91	175	1378.86	1479.70	100
100	174.76	1380.75	1478.70	100
107	174.84	1380.12	1480.80	100
114	174.56	1382.33	1480.80	100
121	174.52	1382.65	1484.00	100
127	174.1	1385.99	1482.00	100
133	174.32	1384.24	1483.60	100
141	174.08	1386.14	1482.90	100
147	174.16	1385.51	1483.70	100
154	174.04	1386.46	1486.00	100
163	173.7	1389.18	1484.40	100
165	173.9	1387.58	1484.00	100

Table 12 Post-Test Water Contents - SCOL12

Position Below Top of Consolidated Soil Layer (mm)	**Water Content (%)**
0 (Top)	120
81 (Middle)	74
162 (Bottom)	59

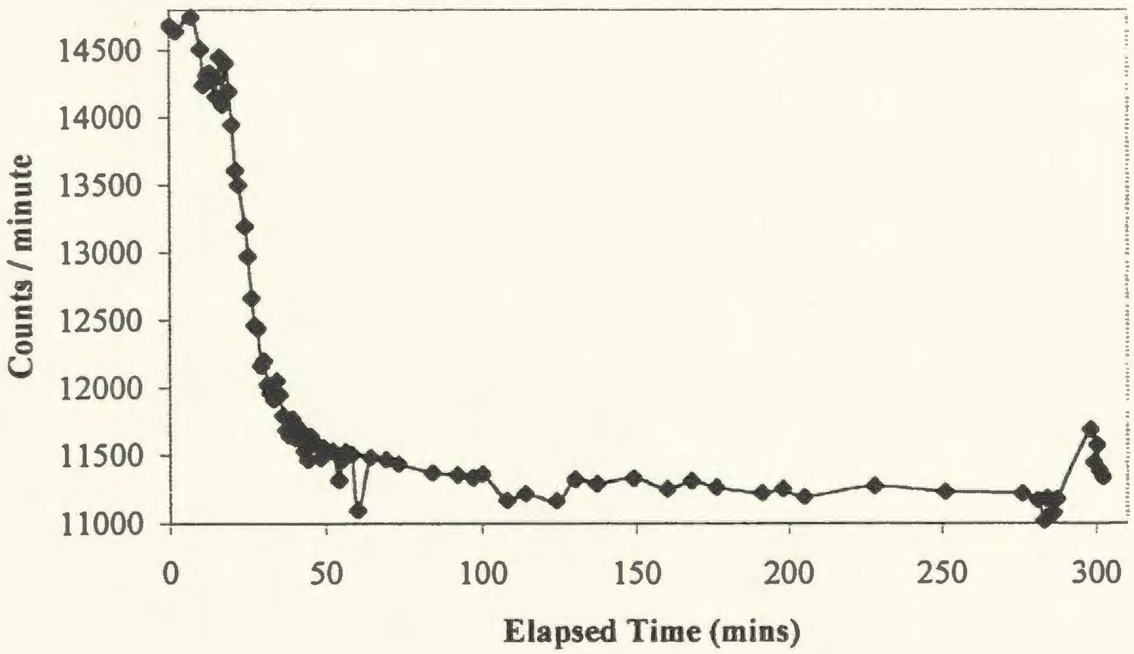


Figure 31 Count Rate vs. Elapsed Time - SCOL12 Phase 2

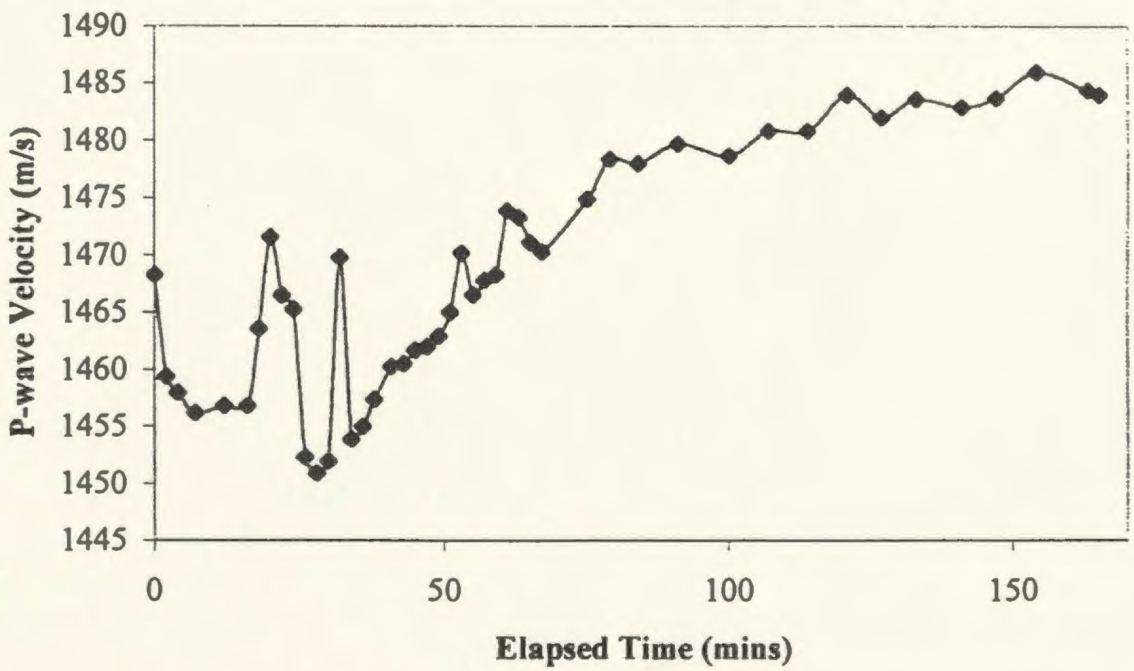


Figure 32 Compressional Wave Velocity vs. Elapsed Time - SCOL12 Phase 2

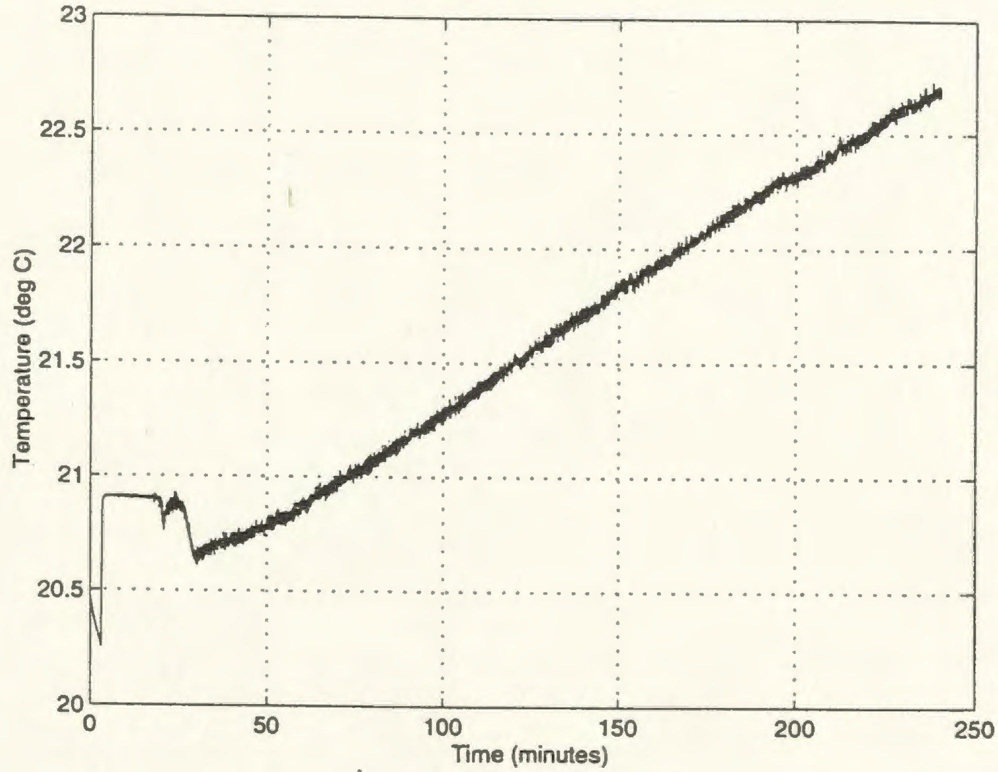


Figure 33 Temperature Variation - SCOL12 Phase 2

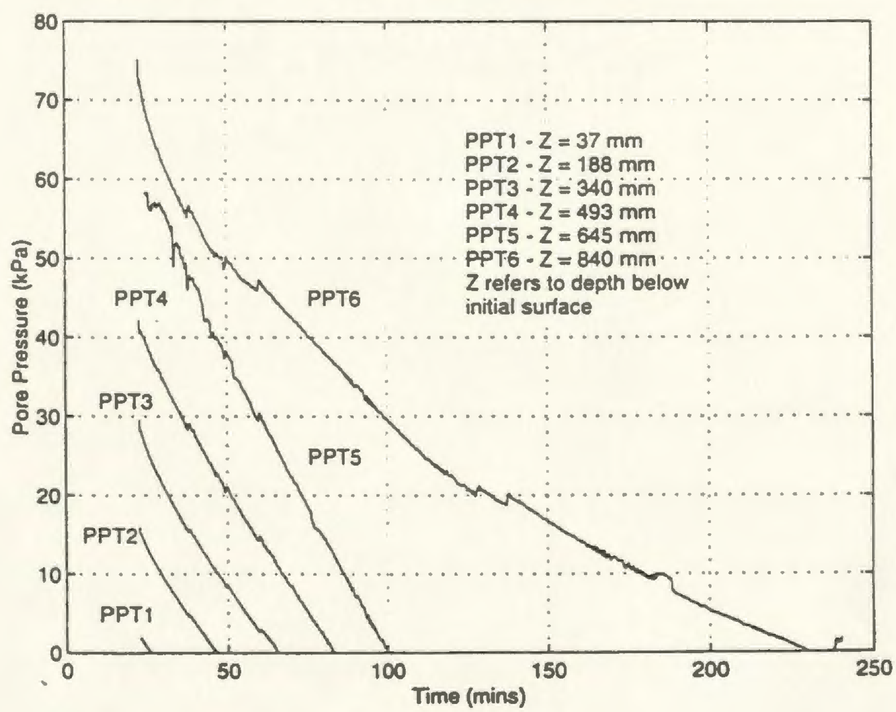


Figure 34 Excess Pore Pressure Dissipation - Phase 2

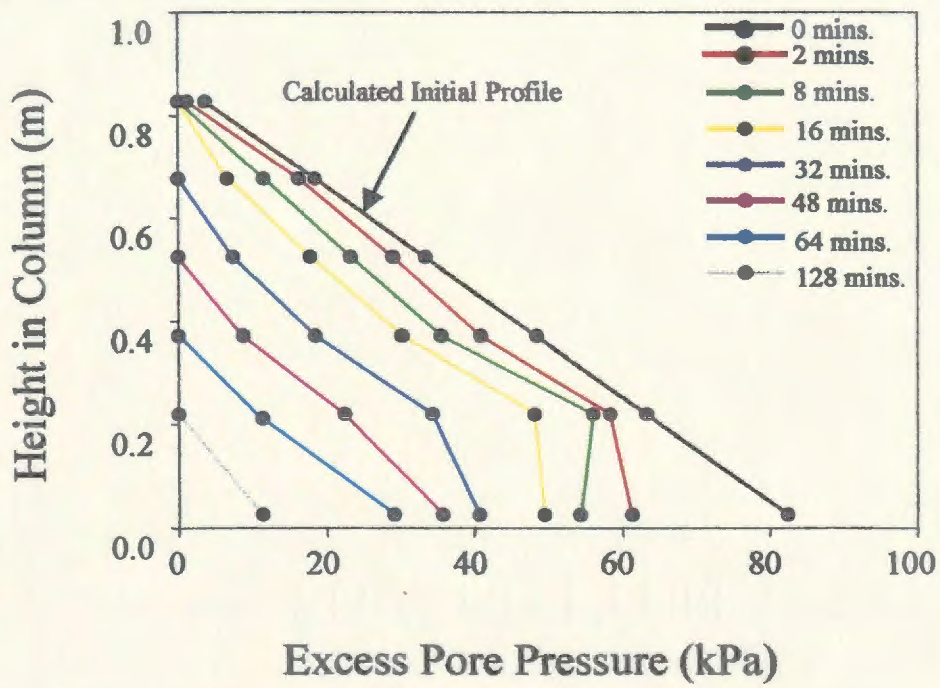


Figure 35 Excess Pore Pressure Dissipation
(Typical 1 g Data Format)

6.2.7 Discussion - SCOL12 Phase 2

Compressional wave velocity is affected by temperature fluctuations. As a result, the measured compressional wave velocities had to be corrected for temperature changes that occurred during the test. It was also necessary to correct for system delays to account for the travel time of the signals through the cables and through the epoxy face of each transducer. The following steps outline the procedure used to correct the raw velocities for temperature and system delays:

1. Sediment temperatures were obtained for the corresponding times at which compressional wave velocities were recorded.
2. The following equation from Clay and Medwin (1977) was used to calculate the expected velocities for the known temperatures from Step 1 (with salinity, S = 0 %):

$$v = 1449.2 + 4.6T - 0.055T^2 + 0.00029T^3 + (1.34\text{-}0.010T)(S\text{-}35) + 0.016z \qquad [14]$$

where: v = velocity (m/s)

T = Temperature (°C)

S = salinity (parts per thousand, ppt)

z = depth (m)

3. The system delay was determined by subtracting the expected velocity from the measured velocity (for the same temperature). This system delay was then added to the measured compressional wave velocities.
4. Using Clay and Medwin (1977), a baseline compressional wave velocity at 20°C was calculated.
5. The velocities from step 3 were then adjusted to a baseline reference temperature of 20°C yielding corrected sediment compressional wave velocities.

Section 6.2 explained the development of the gamma ray density measurement system and the linear relationship between bulk density and the natural log of count rate that was determined from the initial calibration test. However, this calibration could not be directly applied to this particular test. The original system calibration was performed with a different experimental setup in which the source and detector were spaced at a larger distance apart than that in the actual SCOL12 column test. Also, several months had passed between performing the initial calibration test in the small tub, incorporating the system in the column, and performing the column test. As a result, the gamma ray source had possibly decayed somewhat during this time and it was believed this would also affect the calibration. A second calibration relationship was developed to convert the count rates measured during the SCOL12 centrifuge test to actual bulk density measurements.

This was accomplished using data from the actual SCOL12 test; the average 100 g count rate through water (from SCOL12 Phase 1), the average 100 g count rate through the kaolin soft soil measured immediately after reaching test speed at the beginning of the test (assuming no change in initial slurry density had occurred up until that time) and the average 100 g count rate through the consolidated kaolin soil measured immediately prior to stopping the centrifuge at the end of the test (assuming the post-test water content/density measured at the end of the test was the same immediately prior to stopping). The three different density values (water, initial kaolin slurry density and post-test kaolin slurry density) were plotted against the corresponding count rates. The results are presented in Figure 36. Even though there were only three data points used to develop this calibration, the linear relationship which exists between bulk density and the natural log of count rate was demonstrated by the results of the initial calibration test. For this reason, it was believed that three data points were sufficient to define the calibration relationship for the column test.

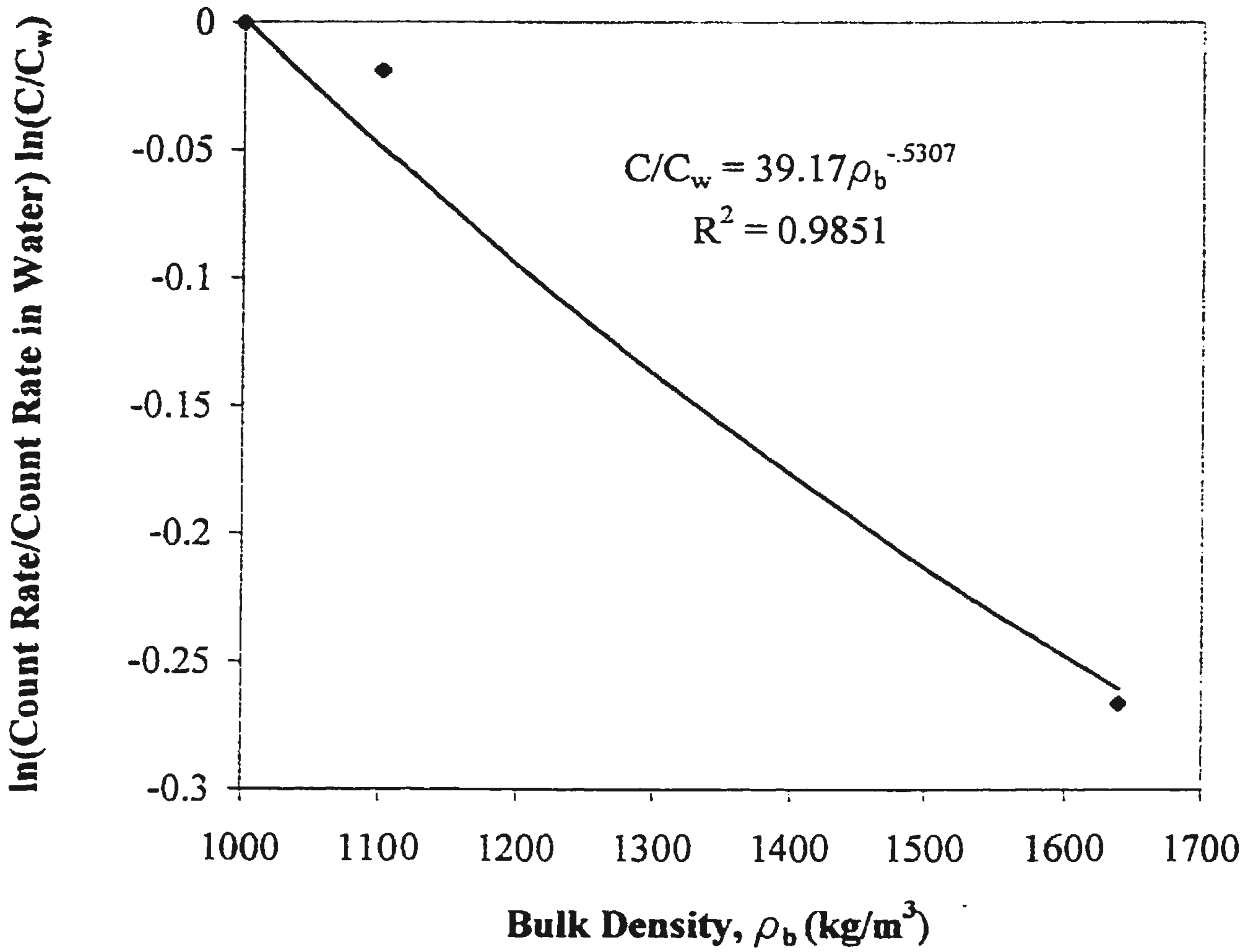


Figure 36 SCOL12 Calibration of Bulk Density Measurement System

Using this calibration, the bulk densities were derived from the count rates measured during the SCOL12 centrifuge experiment and plotted against the corresponding normalized P-wave velocities. The results are presented in Figure 37. The P-wave velocities were normalized using the compressional wave speed in fresh water at 20°C. This value was calculated to be 1481.6 m/s using the equation given by Clay and Medwin (1977). Figure 38 presents a comparison between the SCOL12 data and data from Urick and Hampton (from Ogushwitz (1985)).

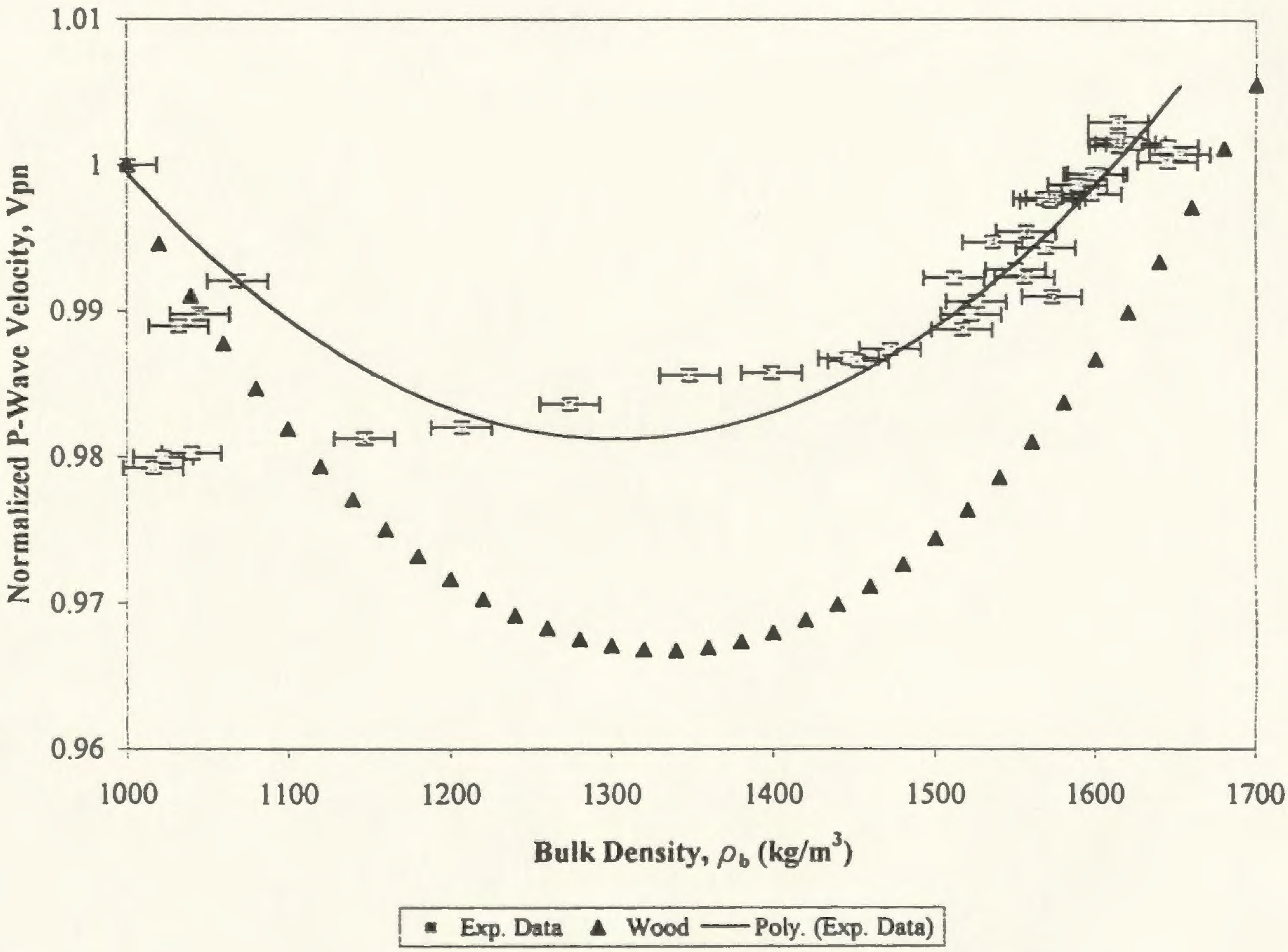


Figure 37 SCOL 12 - Correlation Between Bulk Density and Normalised Compressional Wave Velocity

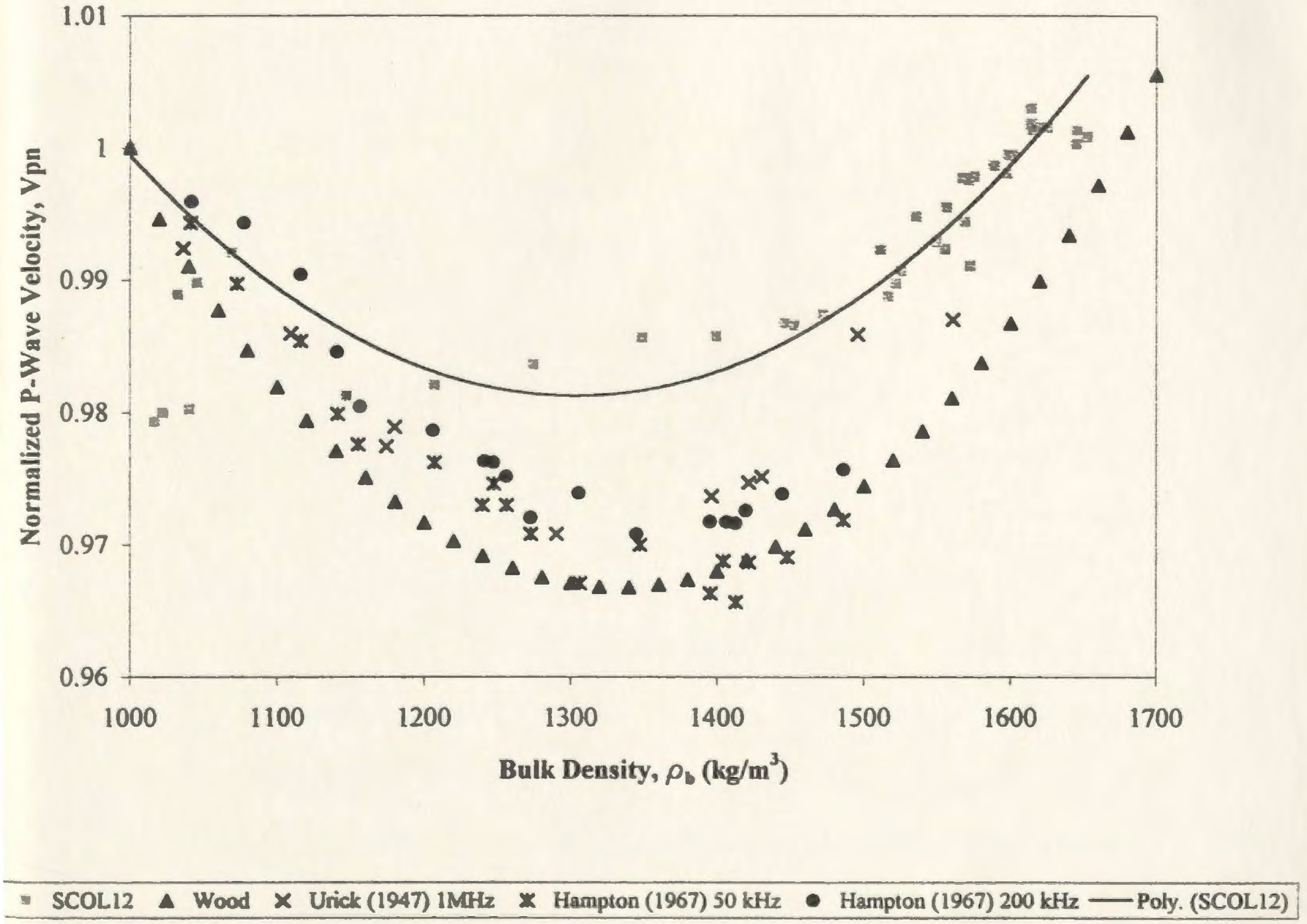


Figure 38 Comparison Between SCOL12 Data and 1g Data

There was a gradual increase in bulk density from about 1100 kg/m^3 at the beginning of the test to a maximum value of about 1650 kg/m^3 near the end of the centrifuge experiment. Similar laboratory experiments conducted under normal gravitational conditions which have been reported in the literature (McDermott, 1992) indicate that the relationship between sediment bulk density and normalized P-wave velocity can be approximated using a quadratic relationship. The same behaviour was observed in the centrifuge results. The following equation describes the relationship between sediment bulk density and normalised P-wave velocity for this centrifuge experiment:

$$V_{pn} = (2 \times 10^{-7})\, \rho_b^{\,2} - (5 \times 10^{-4})\, \rho_b + 1.3158 \qquad [15]$$

The best fit line indicates that a minimum normalized compressional wave velocity occurred at a bulk density of about 1300 kg/m^3 which is also reasonably close to the minimum density value observed in the 1g data. The minimum normalised compressional wave velocity occurred about 15 minutes into the test or about 37 minutes after initial slurry deposition.

The data points which lie below Woods equation between bulk densities of 1000 and 1100 kg/m^3 are questionable. Woods equation represents a lower limit for the relationship between bulk density and normalised compressional wave velocity and it was expected that all of the data would lie above this curve. The reason for this unexpected result is possibly due to the location of the gamma ray source and GM tube with respect to the bottom of the column. The gamma ray density measurement equipment was positioned about 40 mm above the base of the column during the centrifuge test. As sedimentation/consolidation occurred, the soil layer was formed from the bottom of the column upwards. It is possible that the gamma ray system did not register a significant density change until the consolidating soil layer reached some critical thickness and it was closer to the level of the density measurement system.

There are much fewer data points in the initial portion of the curve than at higher densities. This is due to the fact that consolidation occurred very quickly at the beginning of the test and a density of about 1400 kg/m^3 was reached about 42 minutes following deposition or about 20 minutes after reaching test speed.

There is reasonably good agreement among the centrifuge data, Urick's 1 MHz data and Hampton's data up to a density of about 1200 kg/m^3. At higher densities, the level of agreement is not as good. This is possibly due to the fact that the data from both Urick and Hampton was obtained through a kaolinite suspension. In the authors opinion, it would be difficult to maintain a soil suspension at such high values of bulk density. Woods equation has also been presented in Figure 36 and 37. It can be seen that the centrifuge data is reasonably close to Woods equation at lower densities. However, at higher densities (> 1200 kg/m^3) the data deviates from Woods equation which indicates the development of a consolidating soil bed. An unexpected observation was the fact that at higher densities, the data seem to converge with Woods equation. Woods equation was developed to predict compressional wave velocities through soil suspensions. However, the centrifuge compressional wave signals were generated through a consolidating soil layer. The convergence may also be attributed to the fact that there was some uncertainty in the compressibility of kaolinite grains used to generate Woods equation.

Immediately after deposition of a very dilute soil suspension (high water content and low density), sedimentation takes place. Sedimentation occurs as the soil particles (or flocs) deposited at the bottom of the column begin to consolidate due to self weight and the self weight of the material continually being deposited on top. It is difficult to estimate the elapsed time for the sedimentation phase of the centrifuge experiment because centrifuge rotation was started about 10 minutes after slurry deposition and test speed was achieved about 22 minutes after deposition. During the acceleration phase at the beginning of the test, the gravitational force acting on the soil particles is constantly changing and the sedimentation rate is constantly increasing. There is evidence to suggest that the sedimentation phase was complete before reaching test speed. An estimate of the maximum sedimentation time for an individual soil particle can be obtained using Stokes Law (eqn. 1). Using a mean grain diameter for kaolin of 2.4 μm (Ogushwitz, 1985), a specific weight for kaolin particles of 25.8 kN/m^3 and a fluid dynamic viscosity of 1.005×10^{-3} Ns/m^2, the velocity of an individual kaolin particle undergoing sedimentation at normal gravity would be about 5.1×10^{-6} m/s in fresh water. In other words, it would take about 54 hours or 2.3 days for a kaolin particle to reach the bottom of the 1 m high column. Since soil flocs are formed following deposition due to electrostatic forces between particles and the diameter of individual flocs would be several times larger than the average particle diameter, the actual sedimentation time for individual flocs would be significantly less.

There is also some disagreement in the literature with regards to the centrifuge scaling relationship for sedimentation. For example, You and Znidarcic (1994) state that the following relationship should be used to scale sedimentation times from model to prototype:

$$T_m = 1/N^2\, T_p \tag{16}$$

where: T_m = model time

T_p = prototype time

N = centrifuge gravitational acceleration, (N x g)

Other researchers conclude that the centrifuge time scaling exponent for sedimentation varies between 1 and 2 depending on the soil type (Bloomquist and Townsend (1984)). If the scaling relationship is taken as 1 and using the sedimentation time for an individual soil particle, which was estimated using Stokes Law, the centrifuge sedimentation time would have been about 33 minutes, assuming no floc formation. If the scaling relationship is taken as either 1.25 or 2 and if floc formation is considered, then sedimentation was complete before test speed was achieved.

However, the behaviour of PPT's 2-6 of Figure 34 is very similar to the behaviour of PPT's 2 and 3 from SCOL06 and contrary to the theory that sedimentation was complete before test speed was achieved. The general trend was a linear decrease in excess pore pressure until hydrostatic conditions were achieved. The slope of each data trace for PPT 2-6 is approximately equal. Therefore, it is reasonable to assume that the consolidating soil layer did not extend beyond the level of PPT 6.

The measured final water content at the bottom of the column was 59% which corresponds to a void ratio of 1.55. Using Al-Tabba's (1987) void ratio vs. mean effective stress relationship and a final bulk density of about 1600 kg/m^3, the predicted final void ratio was about 1.32. This predicted final void ratio is about 15% lower than the actual final void ratio indicating that the soil was not fully consolidated at the end of the test.

An estimate of the amount of time required to achieve 90% consolidation of the soil layer can be made using the following relationship:

$$t = H^2_{dr} * T / c_v \qquad [17]$$

where: T = time factor (= 0.848 for 90% consolidation)

c_v = coefficient of consolidation (mm^2/s)

t = time (seconds)

H_{dr} = length of the longest drainage path (mm)

Using Al-Tabba's (1987) value for c_v (0.1 mm^2/s) and assuming drainage from the top of the sample only (single drainage) and a soil layer thickness of 162 mm, the amount of time required to achieve 90% consolidation is approximately 62 hours. However, if c_v = 0.5

(Clegg (1981), Table 2), the estimated consolidation time reduces to about 12 hours. The centrifuge experiment was terminated after about 4 hours but a void ratio estimation indicated that the soil may not have reached 90% consolidation at the end of the test. It is clear that the estimated time to achieve 90% consolidation does not correspond to the centrifuge consolidation time, even though the soil layer was not fully consolidated. This may be attributed to "piping" (vertical drainage channels) within the soil which would accelerate the consolidation process.

Liu (1990) described how the size ratio (ratio of initial sediment height to diameter) affected settling behaviour of speswhite kaolin clay and the formation of vertical drainage channels during consolidation. He stated that the higher the size ratio, the greater chance of developing vertical drainage channels. Most of the size ratios studied by Liu (1990) were lower than the size ratio of the column (~4). If vertical drainage channels develop within the soil, there may also be drainage of excess pore water pressure along the column side walls (i.e. similar to two way drainage conditions). Assuming two way drainage, the estimated consolidation time for SCOL12 was about 15 hours, based on Al-Tabba's (1987) value of c_v and approximately 3 hours based on the c_v value of Clegg (1981). It is clear that the consolidation time estimated using Al-Tabba's (1987) c_v value is still much higher than the centrifuge consolidation time of about 3 hours but a close approximation results using Clegg's (1981) value of c_v. The discrepancy between the estimated and actual consolidation time can therefore be mainly attributed to uncertainty in the c_v value and the fact that the soil did not reach 90% primary consolidation during the test.

7.0 COMPARISON OF BULK DENSITY - NORMALISED COMPRESSIONAL WAVE VELOCITY CORRELATIONS (SCOL06 AND SCOL12)

Figure 39 presents bulk densities and corresponding normalised compressional wave velocities for both centrifuge column experiments. Figure 40 presents a comparison of both centrifuge experiments with 1g data from Urick and Hampton. Woods equation which represents a lower bound for the relationship between bulk density and normalised compressional wave velocity for a soil suspension is also presented in both figures for comparison.

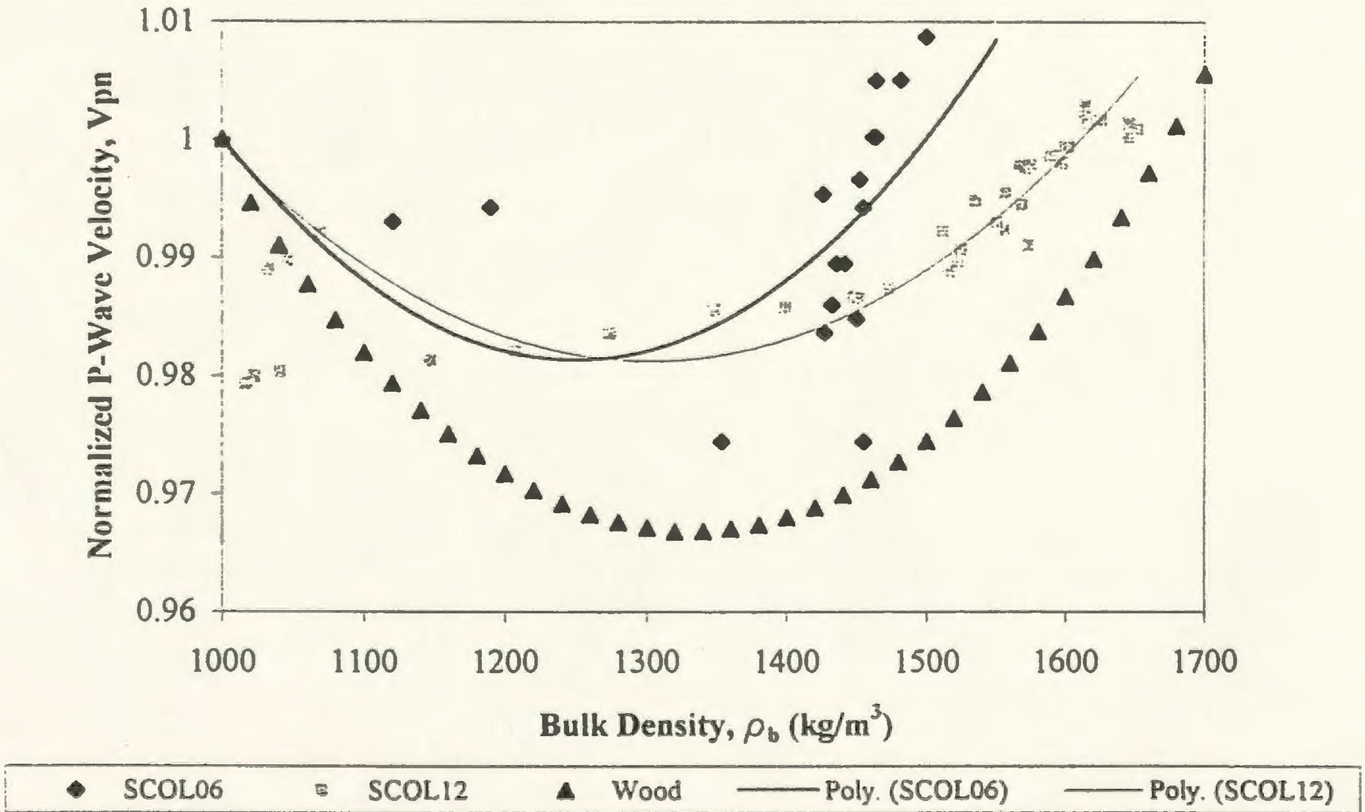


Figure 39 Comparison Between SCOL06 and SCOL12 Correlations

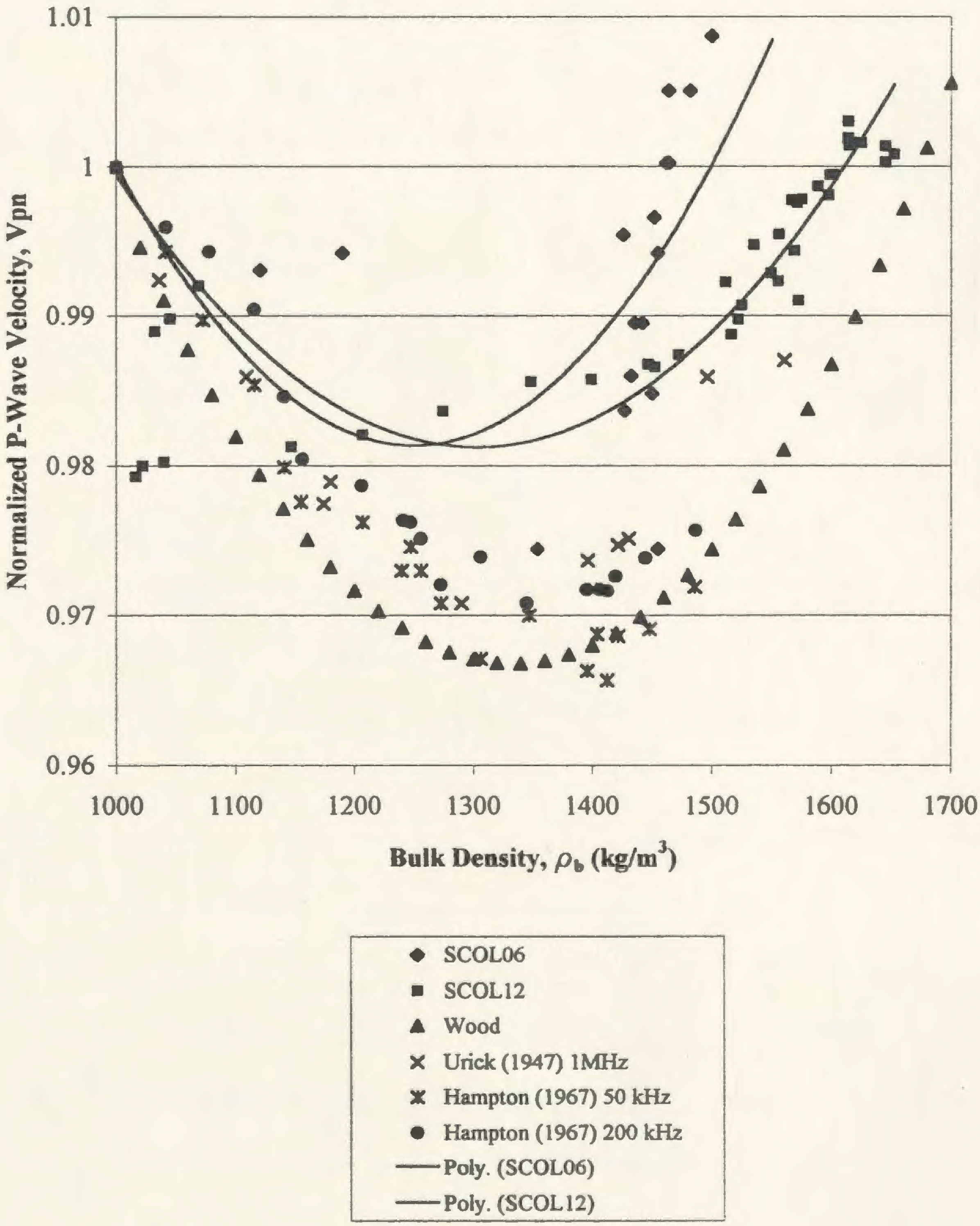


Figure 40 Comparison Among SCOL06, SCOL12 and 1g Data

There are certain similarities between both centrifuge tests. Both data sets follow a quadratic relationship, as expected, and the minimum normalised compressional wave velocity occurred at approximately the same value of bulk density in both centrifuge experiments. This value was about 1300 kg/m^3. This is consistent with the 1g data. For both experiments, the normalised compressional wave velocity initially decreased to some minimum value and then increased and eventually exceed unity. This indicates that the compressional wave velocity through the sediment eventually exceeded the compressional wave velocity through water for both experiments.

There were also certain differences in the two data sets. Even though the same transducers were used in both tests, the minimum value of normalised compressional wave velocity was not the same for both experiments. This difference can be partly attributed to the error in the compressional wave velocity measurement system. Also, the SCOL06 compressional wave data was not corrected for temperature fluctuations during the centrifuge test. Temperature data from SCOL12 indicated that temperature tended to increase as the test progressed. It is reasonable to assume that the soil temperature also increased for test SCOL06. A temperature increase during SCOL06 would have resulted in an increase in compressional wave velocity and therefore an increase in normalised compressional wave velocity. One would therefore expect that if the SCOL06 compressional wave data had been corrected for temperature fluctuations, they would be closer to the compressional wave velocities measured during SCOL12.

At a bulk density value of about 1350 kg/m^3, the two curves overlapped. The final soil bulk density for SCOL12, measured using the gamma ray system, was higher than the final soil bulk density for SCOL06, measured using the electrical resistivity system. This was consistent with the physical measurements performed after the test. The final water content at the level of the gamma ray density measurement system was 59%. However, the SCOL06 water content near the bottom of the column was measured to be 72%. The reason for the difference in final water content, even though initial densities, acceleration levels, and initial excess pore pressures were similar, is not completely clear. The main difference between the two tests was the presence of the vertical ABS pipes to house the gamma ray source and detector inside the column. The difference may be attributed to "piping" of the material. The presence of the vertical ABS pipe sections may have influenced the formation of vertical drainage channels thereby allowing the sediment of SCOL 12 to consolidate faster and to a higher degree than the SCOL06 sediment.

8.0 CONCLUSIONS

A review of the literature indicates that the settling column is a useful tool for the investigation of soft soil consolidation phenomena. Column experiments have been conducted on a wide variety of soil types including clay minerals such as kaolinite, natural marine sediments and oilsands tailings. Columns can be instrumented with a variety of devices to measure geotechnical and geophysical soil parameters such as excess pore pressure, compressional wave velocity, shear wave velocity and bulk density (water content). Column experiments have been previously conducted in the geotechnical centrifuge but these tests were conducted at lower g levels and were done mainly to observe settlement behaviour. Prior to this experimental program, geophysical measurements in the geotechnical centrifuge were limited to the shear wave measurements through sand conducted by Japanese researchers. There is no documented evidence of compressional waves being generated through consolidating soils in the geotechnical centrifuge.

A major disadvantage of 1 g column experiments is the amount of time required to conduct an experiment. Depending on the type of soil, typical column experiments can require months or even years to complete. As a result, 1 g column experiments are very inefficient for the study of soft soil sedimentation/consolidation. Another disadvantage associated with 1 g column experiments is that the effective stresses generated in a typical column test are typically very small ($< \sim 3$ kPa) and do not accurately reflect in-situ effective stresses. This also questions the accuracy of effective stress measurements from 1 g column tests.

It has been demonstrated that the settling column idea can be used in the geotechnical centrifuge to investigate the consolidation characteristics of soft soils. However, certain precautions must be taken to ensure that the column is structurally safe under high gravity levels. The P-wave transducers, constructed using piezoelectric material contained in PVC mounting systems, are an effective and reliable technique for generating P-waves through soft soils undergoing selfweight consolidation in the geotechnical centrifuge. The design of these transducers was based on the P-wave transducer design of McDermott (1992) but with several improvements to increase transducer efficiency.

The quadratic relationship between bulk density and normalised compressional wave velocity for kaolin clay, first determined through the use of 1 g column tests, has also been developed through the use of centrifuge modelling. The excess pore pressures developed during the centrifuge tests were much higher than those developed during 1 g column testing and about 90% dissipation of excess pore pressure was achieved after approximately three hours of centrifuge testing.

There is uncertainty with respect to the use of Woods equation to represent a lower bound in the relationship between compressional wave velocity and soil bulk density. Woods equation is applicable to soil suspensions. The soils created during the centrifuge testing program were initially suspensions but very quickly developed into consolidating soils with measurable effective stresses.

There were advantages and disadvantages to conducting the centrifuge tests at an acceleration level of 100 g. This high stress level demonstrated that the column apparatus was structurally sound and that the compressional wave and density measurement systems operated properly at these high acceleration levels. Another advantage was that dissipation of excess pore pressure was achieved within several hours as opposed to several months with 1 g column tests. However, soil consolidation is characterised initially by a very rapid settlement rate which dramatically decreases as excess pore pressures are dissipated. For this reason, it was difficult to obtain a large amount of data in the initial portion of the bulk density-normalised compressional wave velocity plot at this acceleration level.

Even though hydrometer tests were not conducted on samples of the consolidated soil, it can be concluded with reasonable certainty that particle segregation did occur during the centrifuge column tests. Several researchers state that a critical mixing void ratio exists above which particle segregation will occur, even under 1 g conditions. Particle segregation will result in different soil properties between model and prototype which violates a basic law of centrifuge modelling.

9.0 RECOMMENDATIONS

The bulk density system used during this experimental program enabled one to measure bulk density at one location only at the bottom of the column. A bulk density profiling system must be developed so that a density profile can be obtained along the entire column length. Soil calibration samples must also be developed and the density measurement system accurately calibrated prior to any centrifuge column test.

The centrifuge on-arm data acquisition should be further developed so that it is not necessary to generate and detect compressional waves through the centrifuge slip rings. This would improve the accuracy of the compressional wave measurement system. The column must also be instrumented with more compressional wave transducers so that the variation in compressional wave velocity along the entire column length can be obtained.

The C-CORE settling column should be instrumented with at least two more pore pressure transducers placed near the bottom of the column. This would increase the amount of excess pore pressure dissipation data obtained from the area of greatest interest.

Future testing should be conducted to establish the critical mixing void ratio for speswhite kaolin clay. It is recommended that a 10 g acceleration level is chosen initially and the critical void ratio established for this acceleration level. One should then determine the critical void ratio for higher g levels.

A sample extruder must be developed so that the consolidated clay can be extruded from the column following centrifuge testing. This would allow one to perform a water content profile of the clay sample and to obtain samples for hydrometer analyses to investigate whether particle segregation has occurred.

It is imperative that a direct comparison be made between a model and a prototype column test. The prototype scale column height that can be used is limited to about 6 m. Therefore, the centrifuge experiment can be conducted at 10 g with a 0.6 m high soil column. It would also be possible to verify the centrifuge scaling laws for self/weight sedimentation and consolidation from a comparison of the centrifuge and 1 g results.

A need exists for a system to accurately measure the amount of surface settlement during consolidation. This experimental program relied on the pore pressure transducers as a means to estimate the position of the water/sediment interface. Since the transducer positions are fixed, the location of the sediment/water interface was obtained by recording the time at which the pressure at the transducer location reached hydrostatic conditions. An improved system would enable one to continuously monitor the location of the sediment/water interface. It is therefore recommended that the possibility of purchasing an acrylic column be investigated.

The bulk density measurement system utilised two ABS pipe sections, placed inside the column, to house the gamma ray source and detector. The effect of placing these two 38 mm diameter pipe sections inside the settling column must be investigated further. From a comparison of the two tests described in this thesis, it is not clear whether the presence of the ABS pipe sections had a significant effect on the results of the second test.

The electronic systems developed during the course of this testing program proved to be very robust and usually operated properly under high acceleration levels in the centrifuge. However, the circuit boards were usually mounted on temporary wooden mounting plates and were not enclosed inside protective casings. To prevent accidental damage to the column electronic systems, which may occur just prior to a centrifuge flight and delay the test for several days, it is recommended that all electronic circuitry be enclosed and attached to permanent mounting plates.

All testing was conducted using 250 kHz compressional wave transducers. Future testing must be conducted to investigate the effect of varying the compressional wave transducer frequency on the resulting correlation.

10.0 REFERENCES

Al-Tabba, A. (1987). *Permeability and Stress-Strain Response of Speswhite Kaolin.* Ph.D. Thesis, University of Cambridge.

Been, K. (1980). *Stress Strain Behaviour of a Cohesive Soil Deposited Under Water.* Ph.D. Thesis, University of Oxford.

Been, K. and Sills, G.C. (1981). "Self-Weight Consolidation of Soft Soils: An Experimental and Theoretical Study", Geotechnique, 31, No. 4, pp. 519-535.

Berre, T. and Iversen, K. (1972). "Oedometer Tests With Different Specimen Heights on a Clay Exhibiting Large Secondary Compression", Geotechnique, 22, No. 1, pp. 53-70.

Bloomquist, D.G. and Townsend, F.C. (1984). "Selfweight Consolidation of Very Soft Clay by Centrifuge", Sedimentation Consolidation Models; Predictions and Validation, R.N. Yong and F.C. Townsend, Eds., ASCE, New York, pp. 565-579.

Clay, C.S. and Medwin, H. (1977). *Acoustical Oceanography.* John Wiley and Sons, New York, pp. 3-5.

Dyer, K.R. (1986). *Coastal and Estuarine Sediment Dynamics.* John Wiley and Sons, P. 202-230.

Eckert, W.F., Masliyah, J.H., Gray, M.R. and Fedorak, P.M. (1996). "Prediction of Sedimentation and Consolidation of Fine Tails", American Institute of Chemical Engineering Journal, Vol. 42, No. 4, pp. 960-972.

Elder, D. (1985). *Stress, Strain and Strength Behaviour of Very Soft Soil Sediment.* Ph.D. Thesis, University of Oxford.

Gibson, R.E., England, G.L. and Hussey, M.L.J. (1967). "The Theory of One Dimensional Consolidation of Saturated Clays", Geotechnique, 17, pp. 261-273.

Hamilton, E.L. (1971). "Elastic Properties of Marine Sediments", Journal of Geophysics Research, 76, 2, pp.579-604.

Holtz, R.D. and Kovacs, W.D. (1981). *An Introduction to Geotechnical Engineering.* Prentice-Hall, Inc., New Jersey, pp. 199-375.

Illangasekare, T.H., Znidarcic, D., Al-Sheridda, M. and Reible, D.D. (1991). "Multiphase Flow in Porous Media", Centrifuge '91, H-Y. Ko and F.G. McLean, Eds., Rotterdam, pp. 517-523.

Jackson, P.D. (1975a). *Electrical Properties of Irish Sea Sediments.* Ph.D. Thesis, University of Wales.

Jackson, P.D. (1975b). "An Electrical resistivity Method for Evaluating the In-Situ Porosity of Clean Marine Sands", Marine Geotechnology, Vol. 1, No. 2, pp. 91-115.

Jackson, P.D., Taylor Smith, D., and Stanford, P.N. (1978). "Resistivity-Porosity-Particle Shape Relationships for Marine Sands", Geophysics, Vol. 43, No. 6, pp. 1250-1268.

Kearey, P. and Brooks, M. (1991). *An Introduction to Geophysical Exploration.* Blackwell Scientific Publications, Oxford, U.K., pp. 21-29.

King, A.D., Hansen, D. and Neiffer, R. (1993). "Determination of Soil Parameters by Self-Weight Consolidation in a Chemical Centrifuge", Memorial University of Newfoundland, Faculty of Engineering and Applied Science Technical Report Series, Report No. 93003.

Kitazume, M., Terashi, M. and Aihara, N. (1993). "Centrifuge Model Tests on the Consolidation Behaviour of Soft Clay With Fabri-Packed Sand Drains", Geosynthetics '93, pp. 393-402.

Lee, K. and Sills, G.C. (1981). "The Consolidation of a Soft Soil Stratum, Including Self Weight Effects and Large Strains", International Journal for Numerical and Analytical Methods in Geomatics, 5, pp. 405-428.

Lin, T.W. and Lohnes, R.A. (1984). "Sedimentation and Self Weight Consolidation of Dredge Spoil", Sedimentation Consolidation Models; Predictions and Validation, R.N. Yong and F.C. Townsend, Eds., ASCE, New York, pp. 464-480.

Liu, J. (1990). *Determination of Soft Soil Characteristics.* Ph.D. Thesis, University of Colorado.

Liu, Y., Caughill, D. and Burns, R. (1994). "Consolidation of SUNCOR Nonsegregating Tailings", 47th Annual Canadian Geotechnical Conference, pp. 504-513.

Martinez, R.E. (1987). *Consolidation Properties of Phosphatic Clays From Automated Slurry Consolidometer and Centrifugal Model Tests.* Ph.D. Thesis, University of Florida.

Martinez, R.E., McVay, M., Bloomquist, D. and Townsend, F.C. (1987). "Consolidation of Slurried Soils", Hydraulic Engineering, pp. 285-290.

McCann, D.M. (1968). *Acoustic Properties of North Atlantic Cores.* Ph.D. Thesis University of Wales.

McDermott, I.R. (1992). *Seismo-Acoustic Investigations of Consolidation Phenomena*. Ph.D. Thesis, University of Oxford.

McDermott, I.R. and King, A.D. (1998). "Use of a Bench-Top Centrifuge to Assess Consolidation Parameters", Tailings and Mine Waste '98, Rotterdam, pp. 281-288.

Michaels, A.S. and Bolger, J.C. (1962). "Settling Rates and Sediment Volumes of Flocculated Kaolin Suspensions", Industrial & Engineering Chemistry Fundamentals, 1, pp. 24-33.

Mikasa, M. (1963). "The Consolidation of Soft Clay", Japan Society of Civil Engineering, pp. 21-26.

Mikasa, M. and Takada, N. (1984). "Selfweight Consolidation of Very Soft Clay by Centrifuge", Sedimentation Consolidation Models; Predictions and Validation, R.N. Yong and F.C. Townsend, Eds., ASCE, New York, pp. 121-141.

Ogushwitz, P.R. (1985). "Applicability of the Biot Theory.II. Suspensions", Journal of the Acoustic Society of America, 77, pp. 441-451.

Paulin, M.J. and Phillips, R. (1995). "Centrifuge Modelling Relevant to the Lateral Loading of Pipelines; Year 2 Summary Report". Contract report for NOVA Corporation of Alberta, C-CORE Publication 95-C7.

Schiffman, R.L., Pane, V. and Gibson, R.E. (1984). "The Theory of One-Dimensional Consolidation of Saturated Clays; An Overview of Nonlinear Finite Strain Sedimentation and Consolidation", Sedimentation Consolidation Models; Predictions and Validation, R.N. Yong and F.C. Townsend, Eds., ASCE, New York, pp. 1-29.

Scott, J.D., Dusseault, M.B. and Carrier, W.D. (1986). "Large-Scale Self-Weight Consolidation Testing", Consolidation of Soils: Testing and Evaluation, ASTM STP 892, R.N. Yong and F.C. Townsend, Eds., American Society for Testing and Materials, Philadelphia, pp. 500-515.

Shumway, G. (1958). "Sound Velocity vs. Temperature in Water-Saturated Sediments", Geophysics, Vol. 23, No. 3, pp. 494-505.

Sills, G.C. and Thomas, R.C. (1984). "Settlement and Consolidation in the Laboratory of Steadily Deposited Sediment", Proc. IUTAM/IUGG Symposium, University of Newcastle-upon-Tyne, September, Seabed Mechanics, Eds., Bruce Denness, Graham & Trotman Ltd., London.

Sorenson, J.A. and Phelps, M.E. (1987). *Physics in Nuclear Medicine*. W.B. Saunders Company, Philadelphia, pp. 65-68.

Stoll, R.D. (1989). *Sediment Acoustics*. Springer-Verlag, New York.

Stone, K.J.L., Randolph, M.F., Toh, S. and Sales, A.A. (1993). "Evaluation of the Consolidation Behaviour of Mine Tailings", Research Report No. G1070, Geomechanics Group, University of Western Austrailia.

Takada, N. and Mikasa, M. (1986). "Determination of Consolidation Parameters By Selfweight Consolidation Tests in Centrifuge", Consolidation of Soils: Testing and Evaluation, ASTM STP 892, R.N. Yong and F.C. Townsend, Eds., American Society for Testing and Materials, Philadelphia, pp. 548-566.

Tan, S., Tan, T., Ting, L., Yong, K., Karunaratne, G.P. and Lee, S. (1988). "Determination of Consolidation Properties for Very Soft Clay", Geotechnical Testing Journal, GTJODJ, Vol. 11, No. 4, pp. 233-240.

Terzaghi, K. (1925). "Principles of Soil Mechanics", Engineering News Record, 95, pp. 265-295.

Williams, D.J. (1988). "Consolidation, Crusting and Loading of a Soil Slurry at 1 and 100 Gravities", 5th Annual Geomechanics Conference, Sydney, Austrailia.

Yong, R.N. (1984). "Particle Interaction and Stability of Suspended Solids", Sedimentation Consolidation Models; Predictions and Validation, R.N. Yong and F.C. Townsend (eds.), ASCE, New York, pp. 30-59.

You, Z. and Znidarcic, D. (1994). "Initial Stage of Soft Soil Consolidation", Centrifuge '94, Lee & Tan, Eds., pp. 399-403.

Appendix A

Column Design Calculations

Calculate hoop stress in 1/4" thick steel rings:

Assumptions:

1. Gravity level = 125 g.
2. Density = 1600 kg/m$^{3.}$
3. Neglect strength offered by PVC pipe for calculation purposes.

$\sigma_h = \rho\, g\, h\phi / 2\, t$

where: σ_h = hoop stress
ρ = density
g = gravity level
t = thickness
h = height

$\sigma_h = (1600)(9.81)(125)(1)(.3) / (2)(0.00635) = 46.3$ MPa. O.K.

Therefore, four steel rings placed along the bottom portion of the column will be sufficient to carry the developed stresses.

Calculate strength of steel ring connection:

$\sigma_h = 46.3$ Mpa

- for design purposes, assume tensile stress in rings = 46.3 MPa

Try 5 6 mm ϕ bolts:

$F = \sigma / A$, $A = (0.00635)(.150) = 0.000953$ m^2.

$F = (46.3\text{ MPa})(0.000953\text{ m}^2.) = 44.1$ kN Total Force

44.1 kN / 5 = 8.8 kN per bolt

Calculate tensile strength of each bolt:

$T_r = 0.75\, \phi_b\, n\, A_b\, F_u$

where: ϕ_b = bolt factor = 0.67
n = number of bolts = 1
A_b = bolt cross sectional area
F_u = specified minimum tensile strength

If A325M bolt grade is selected, F_u = 830 Mpa.

For a 6 mm diameter bolt, T_r = 11.8 kN.

Now, force developed in each bolt = 8.82 kN < 11.8 kN.

Therefore, 5 6 mm diameter A325M bolts are suitable for the connection on each steel supporting ring.

Calculate stress in top plate:

ϕ = 360 mm (O.D.)
t = 25.4 mm

Material: Mild Steel

q = pressure = ρ g h

q = (7860)(9.81)(125)(.0254) = 244.8 kPa.

Calculate moment at center of plate due to self weight, M_c:

$M_c = q\, a^2\, (3 + \upsilon)/16$ where: υ = Poisson's ratio for steel (assume 0.3)
a = radius of lid

M_c = 1636 N.m

$\sigma = 6\, M_c/t^2$ = 15.2 MPa

Since stress in top plate is much less than steel yield stress, plate is O.K..

Calculate pressure in PVC pipe at top of steel rings:

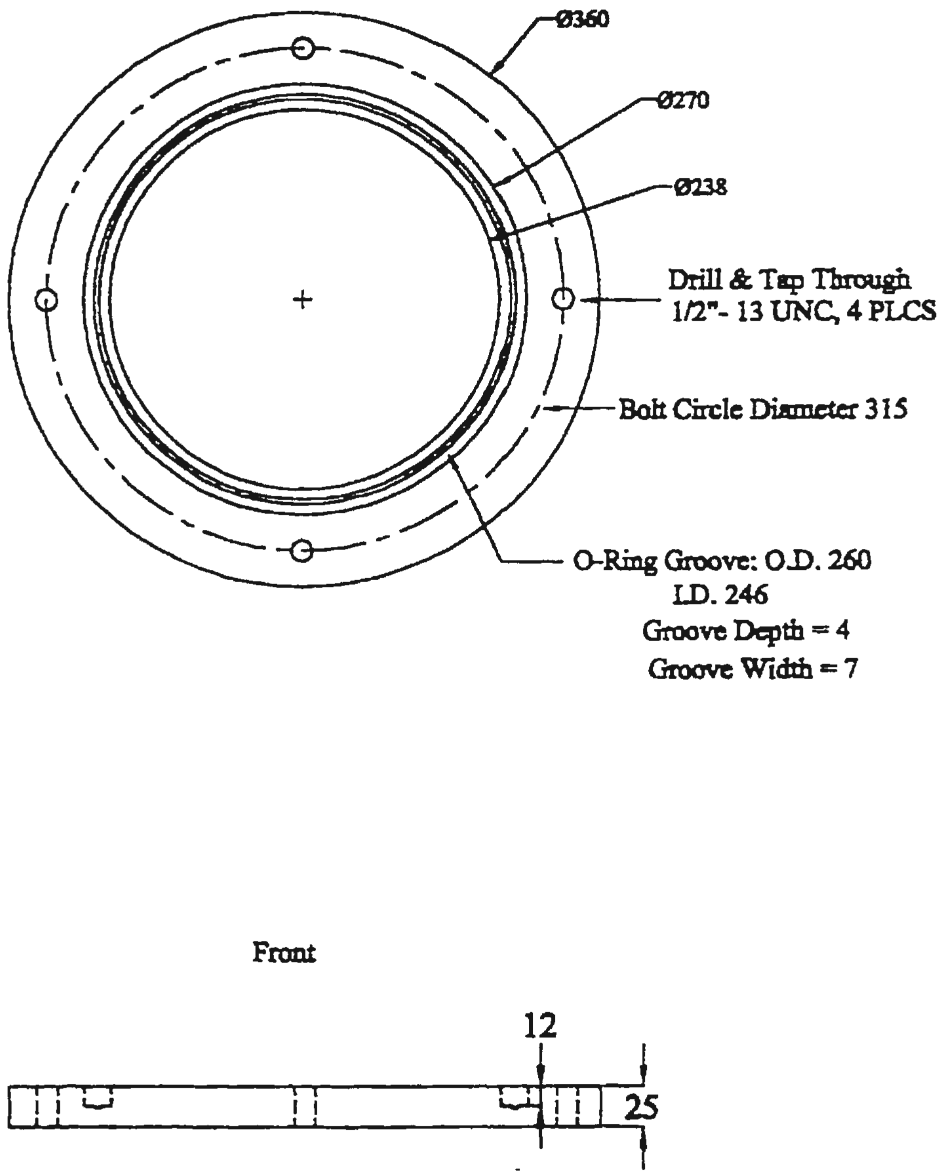
P = ρ g h = (1600)(9.81)(125)(.385) = 755.3 kPa = 109.5 psi

Now, working pressure for Schedule 80 PVC pipe = 230 psi.

109.5<< 230 psi. Therefore, O.K..

Appendix B

Column Drawings



Notes: 1. Material: Mild Steel
2. All dimensions in mm.

Figure B1 Column Lid / Base

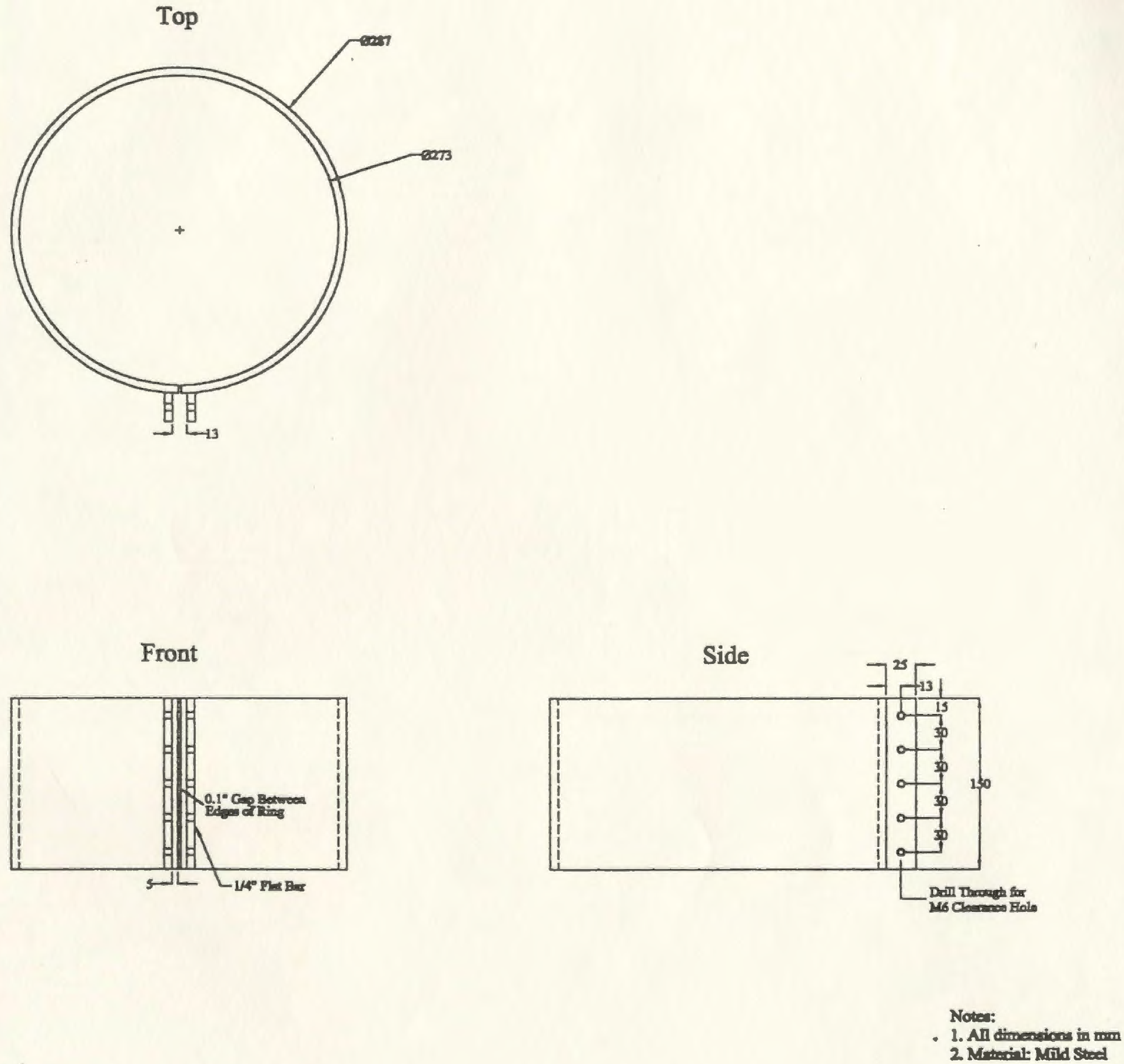
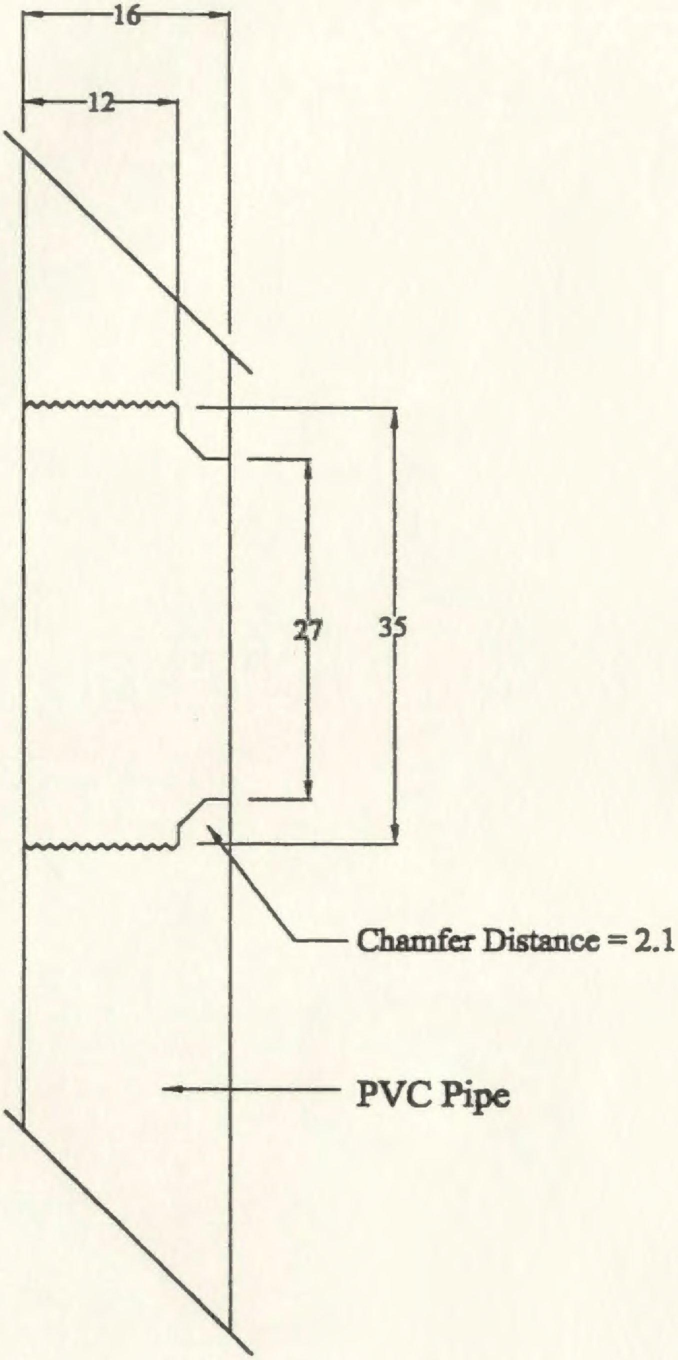
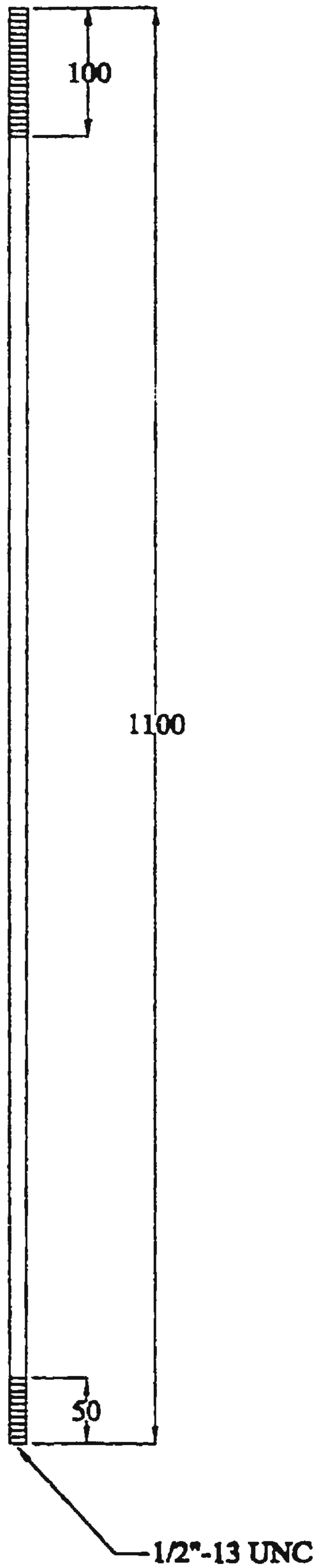


Figure B2 Steel Support Ring



Notes:
1. All Dimensions in mm

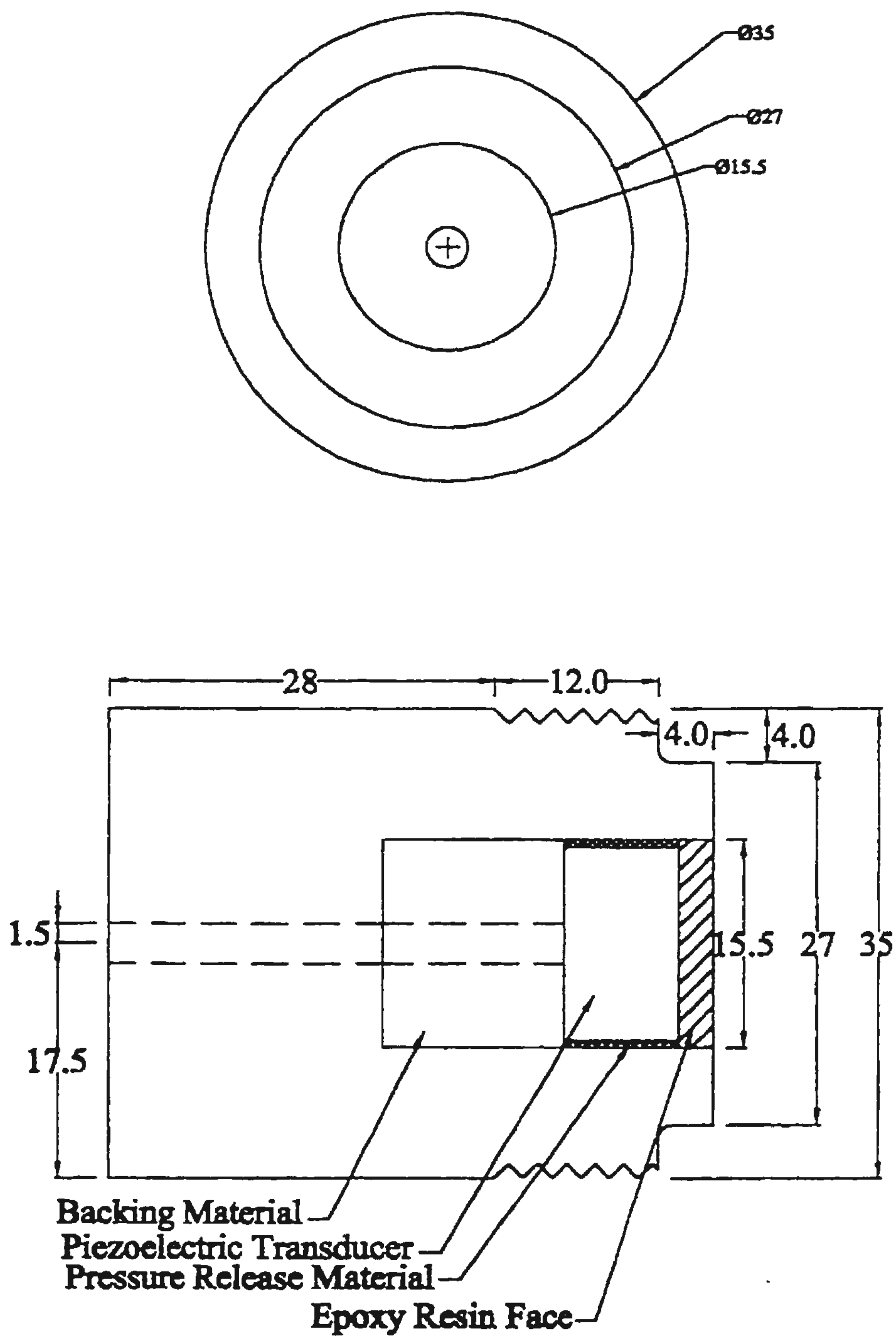
Figure B3 Compressional Wave Transducer Hole Details



Notes:
1. Material: Mild Steel
2. All Dimensions in mm

Figure B4 Column Tie Rod Details

Plan



Notes:

1. Bore 24 mm deep
2. Material - PVC

Figure B5 Compressional Wave Transducer (Plan and Profile)

Appendix C

Impedance Analyser Data

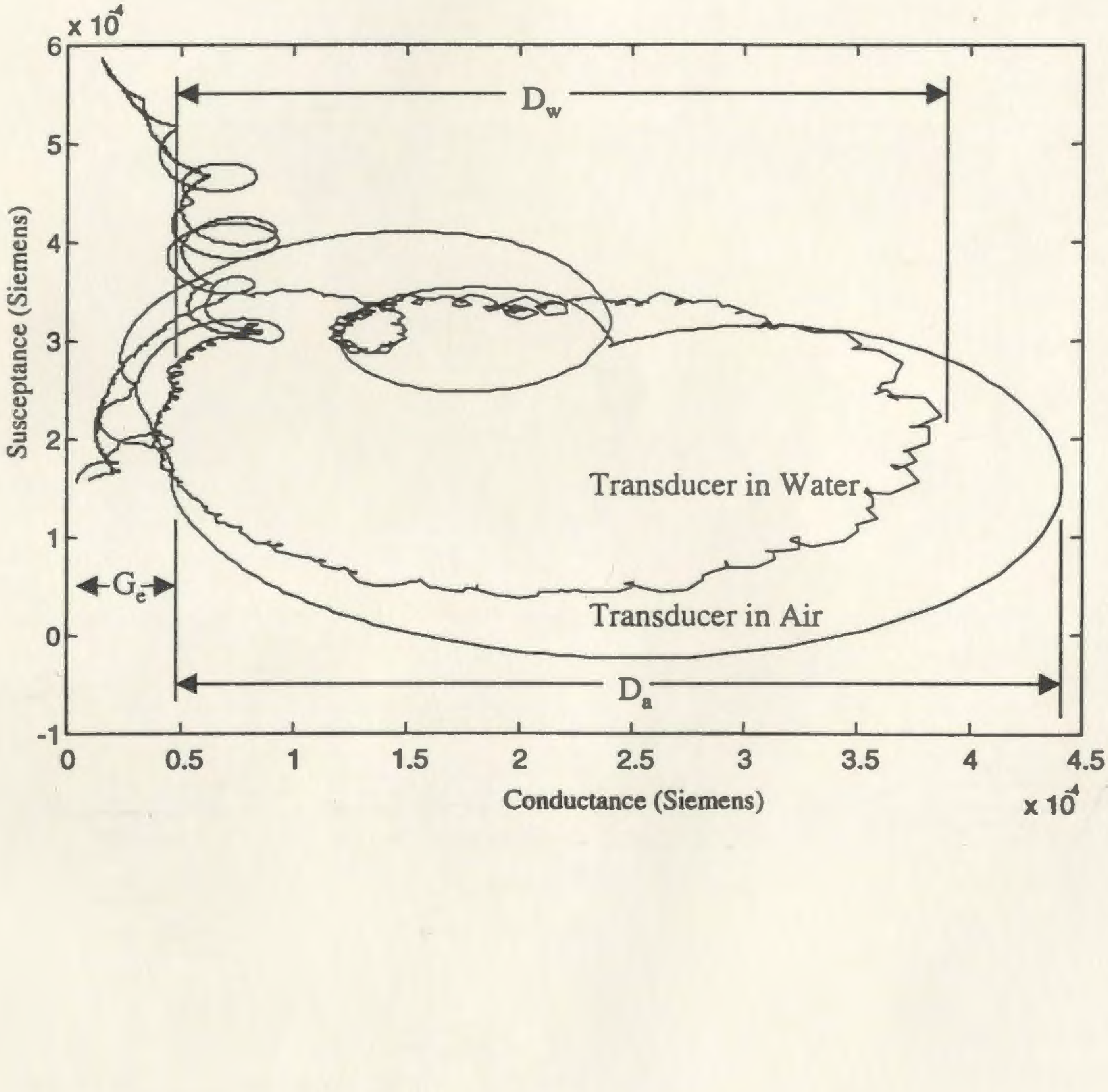


Figure C1 Example of Admittance Circle

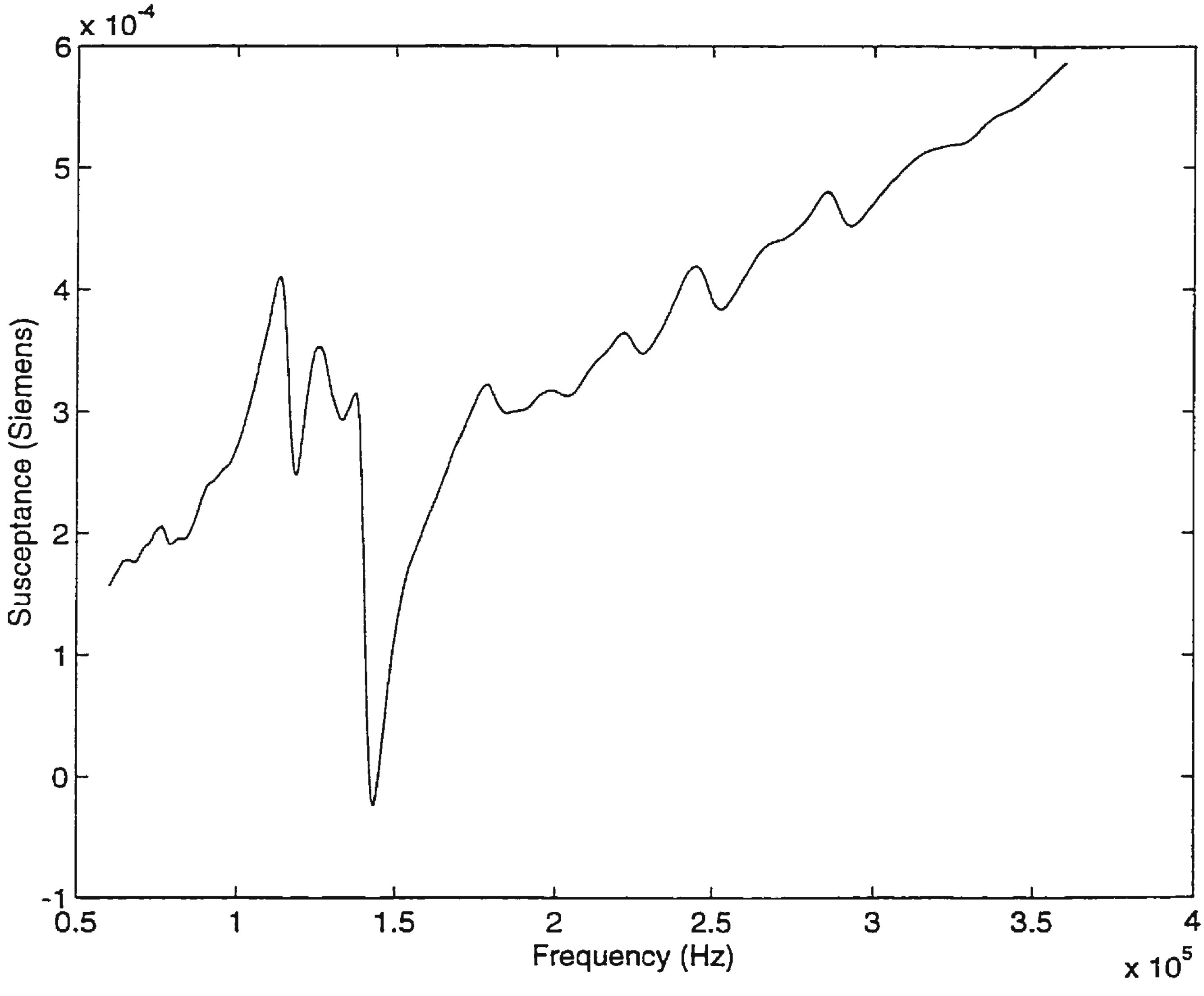


Figure C2 Frequency vs. Susceptance - Transducer 1 in Air

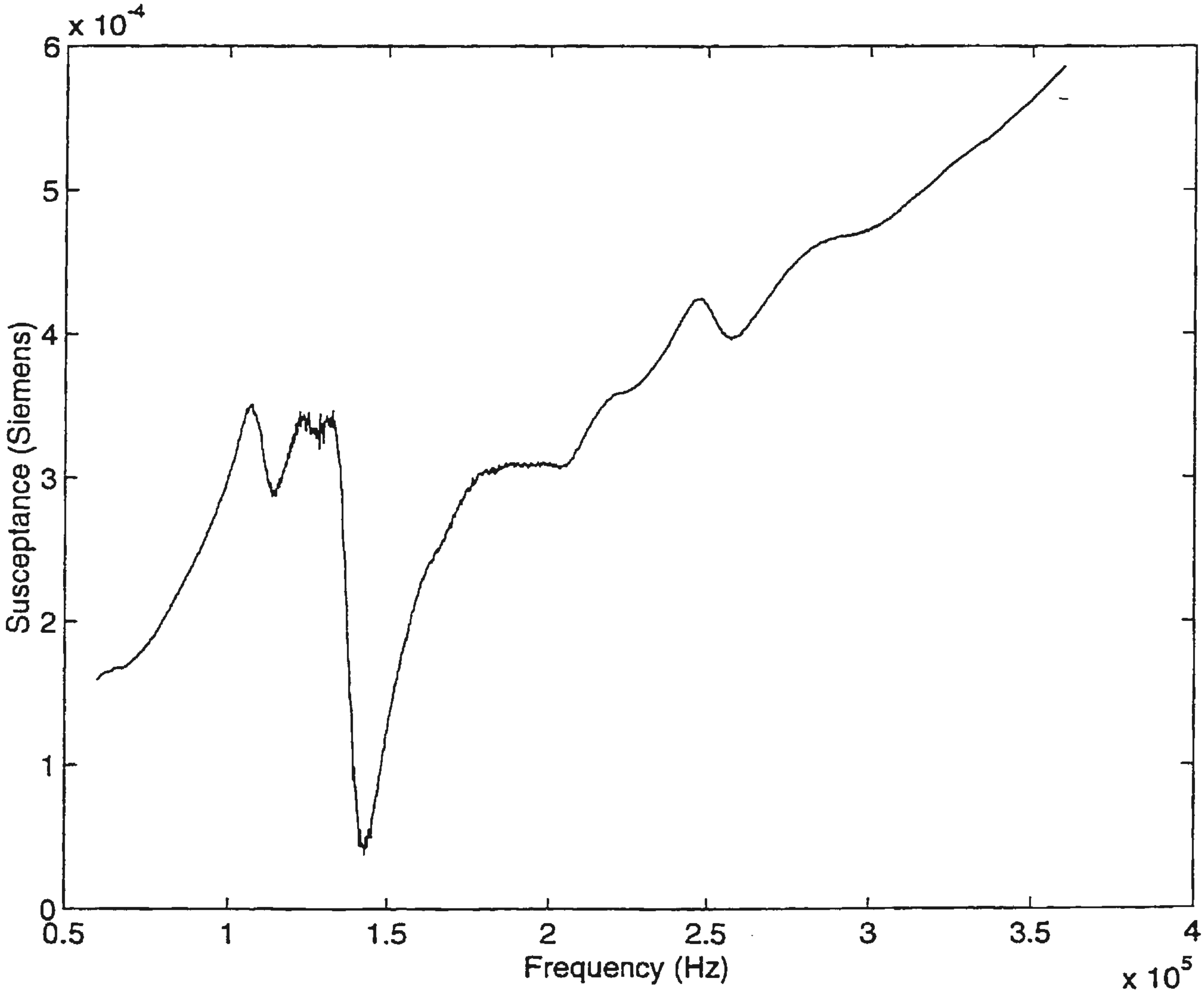


Figure C3 Frequency vs. Susceptance - Transducer 1 in Water

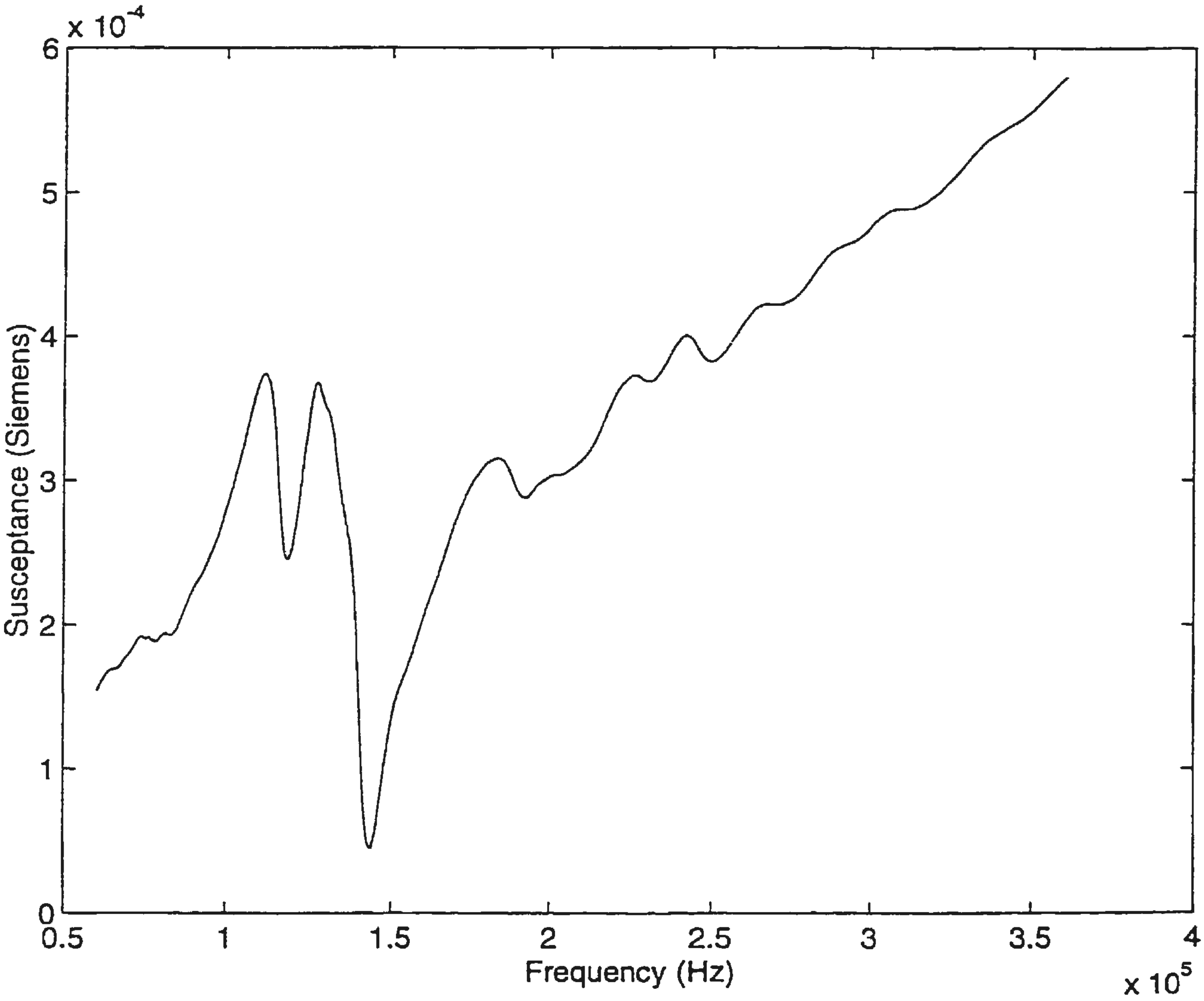


Figure C4 Frequency vs. Susceptance - Transducer 2 in Air

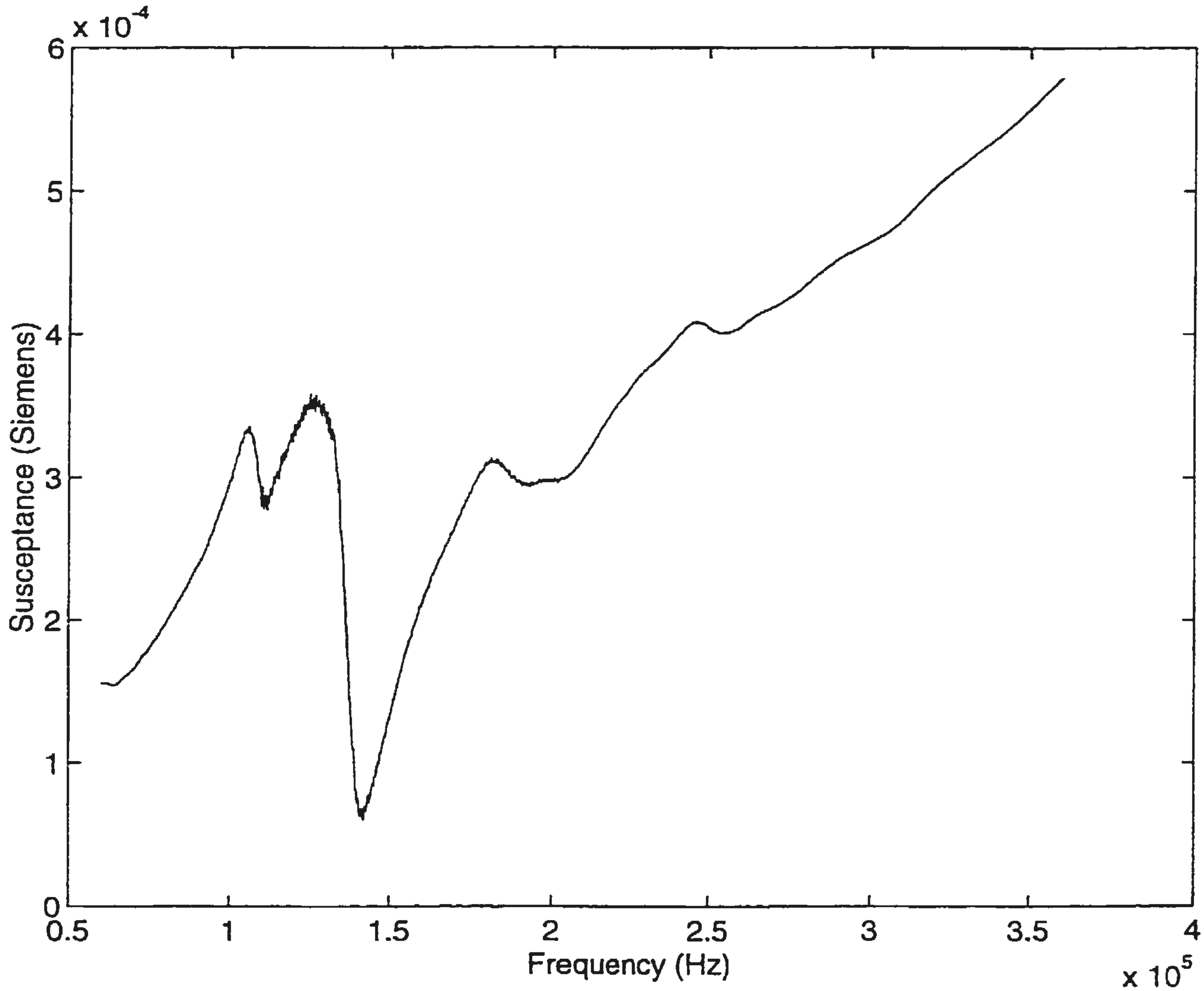


Figure C5 Frequency vs. Susceptance - Transducer 2 in Water

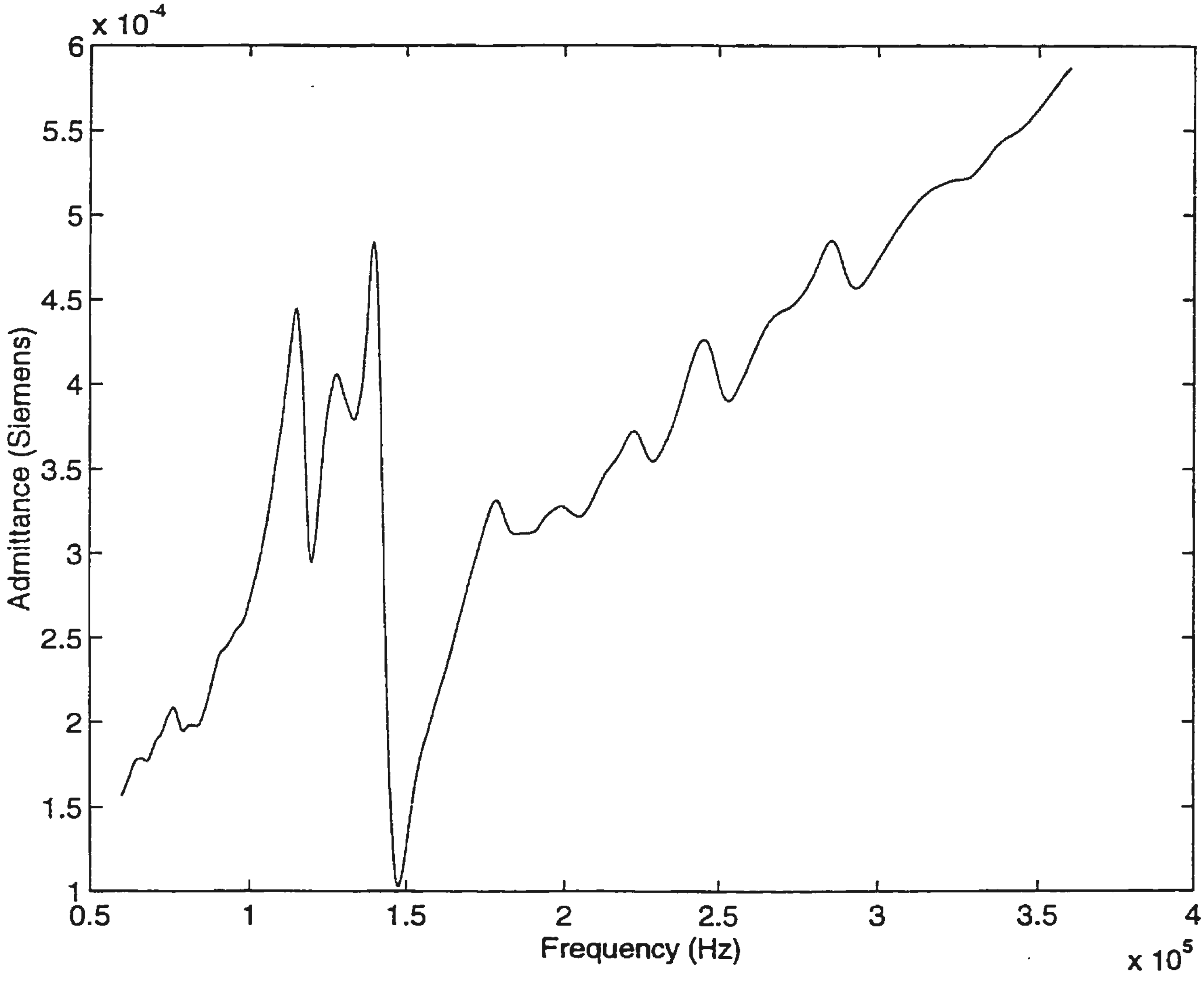


Figure C6 Frequency vs. Admittance - Transducer 1 in Air

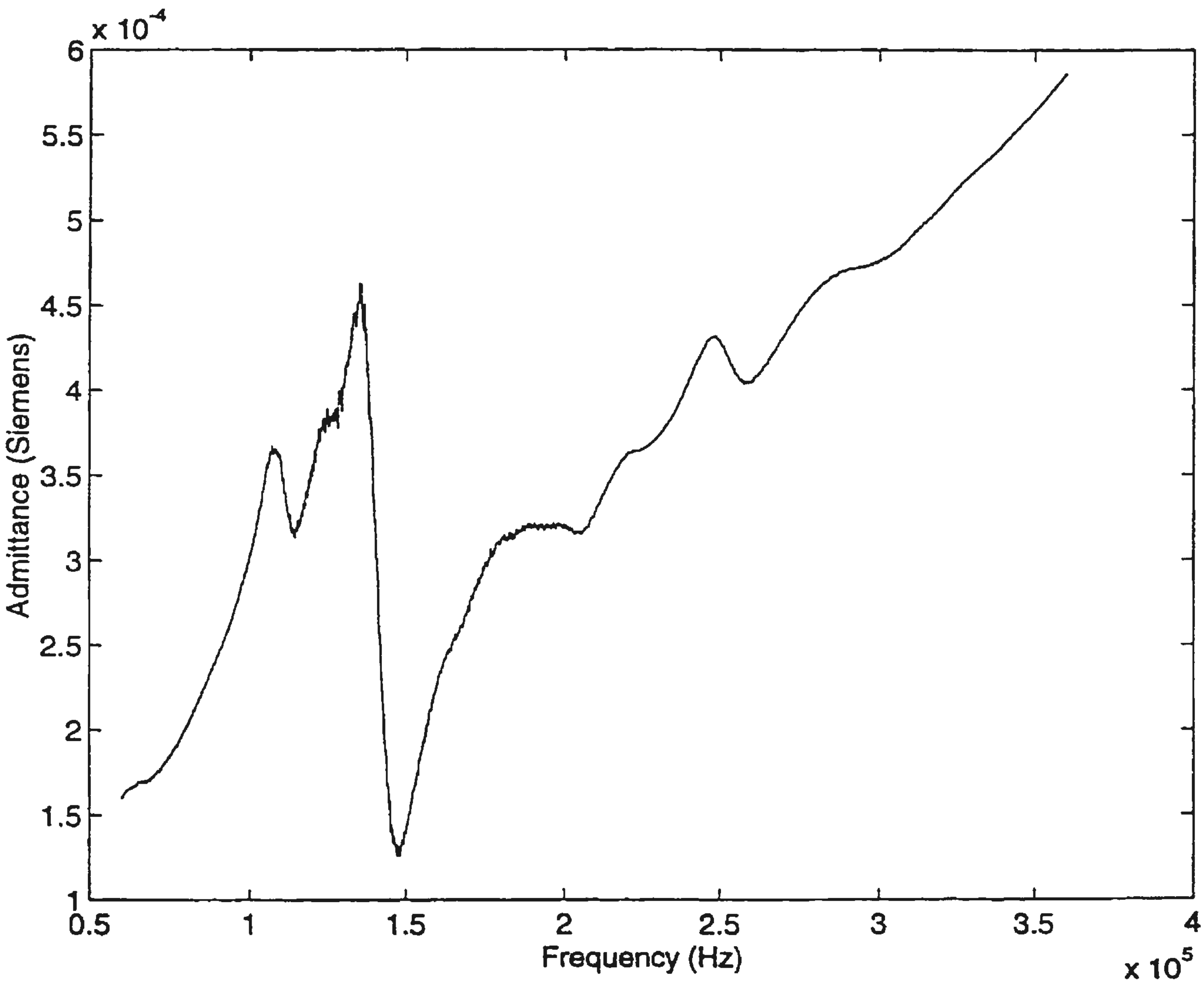


Figure C7 Frequency vs. Admittance - Transducer 1 in Water

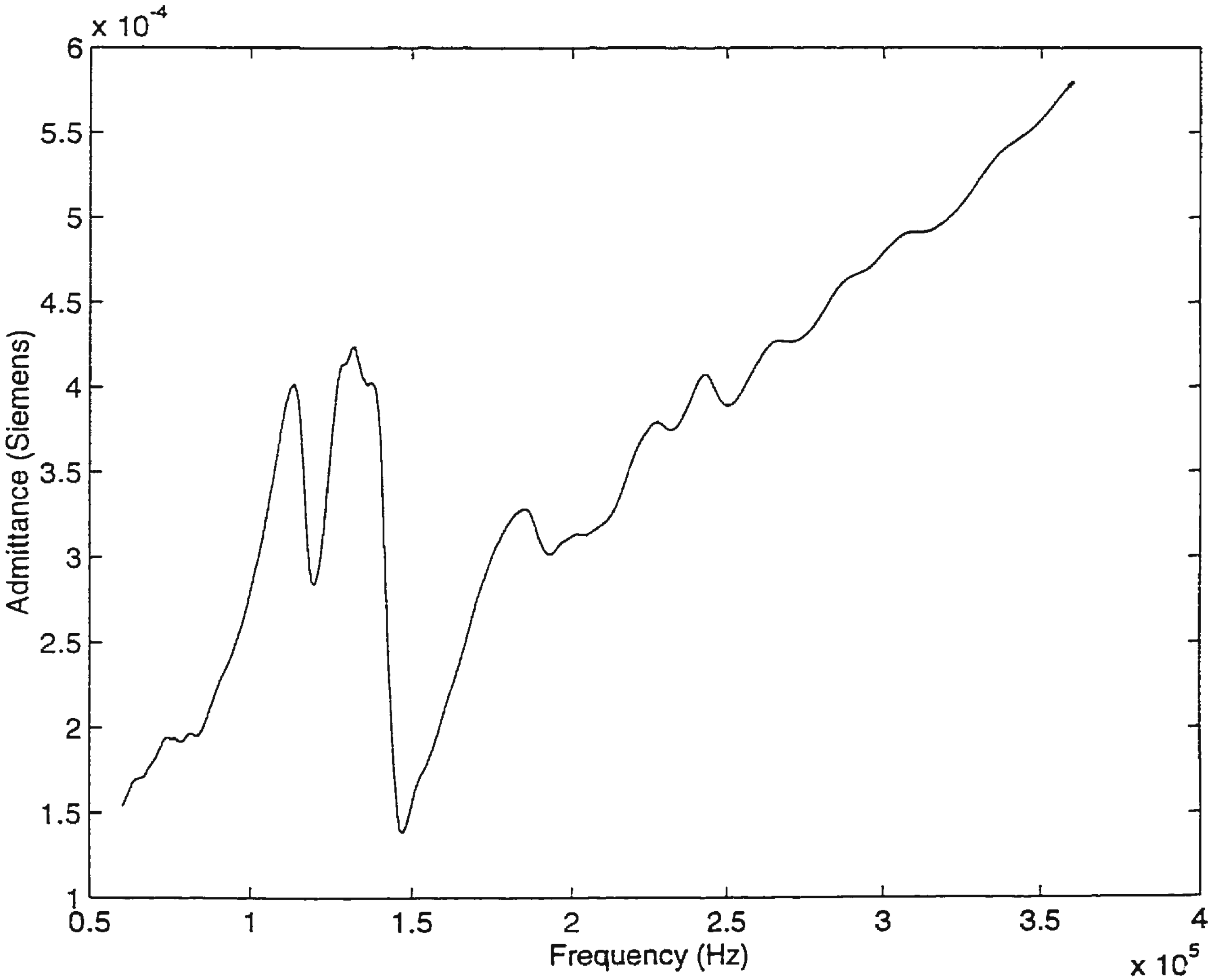


Figure C8 Frequency vs. Admittance - Transducer 2 in Air

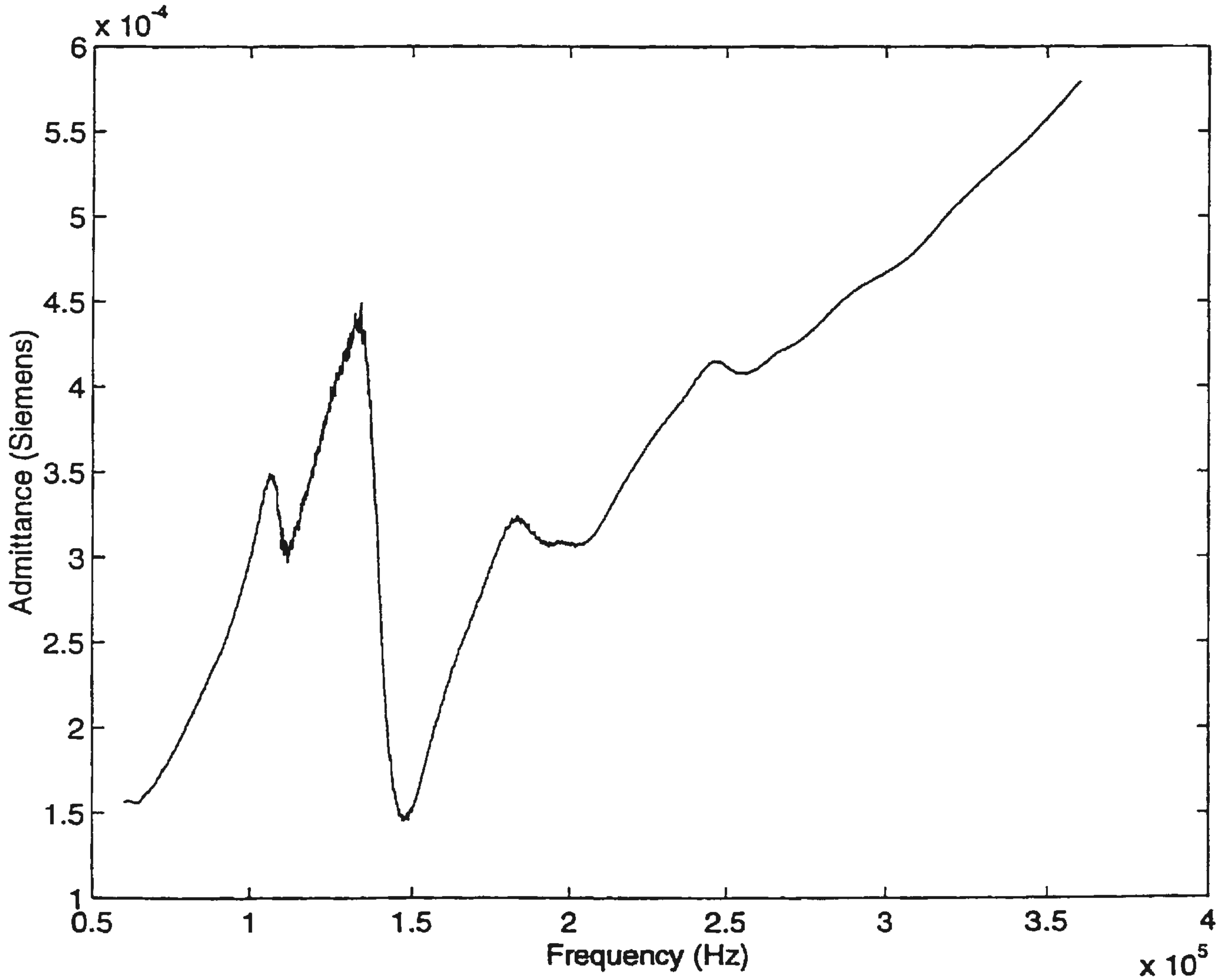


Figure C9 Frequency vs. Admittance - Transducer 2 in Water