UNIVERSITY OF ALBERTA

GEOTECHNICS OF OIL SAND FINE TAILINGS

BY

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to my dear departed mother

&

to my father and sisters
who helped me to get here

&

to my husband
who helped me to get through

ABSTRACT

In Alberta, Canada, oil sand is mined and processed to recover synthetic crude oil. The fine waste stream from the oil extraction process is called fine tails. Presently, 400 million m^3 of fine tails is held in tailings ponds. Understanding the consolidation behavior of fine tails and the factors affecting it are important for the design of disposal options.

A slurry consolidometer has been developed to allow for large deformations during consolidation and to allow permeability tests to be performed on the fine tails. Unlike normal soils, it was found that the amount of compressibility of the fine tails is influenced by the initial void ratio of the sample, which suggests that aging changes the microstructure of the fine tails. A phenomenon of permeability testing of slurries is that the flow velocity is not constant and decreases with time to a steady state value. The flow velocity, used to calculate the hydraulic conductivity, was studied and it was determined that the steady state velocity should be used to determine the field permeability. The permeability of the fine tails is also influenced by hydraulic gradient and bitumen content.

Though the fine tails appears to consolidate in the tailings ponds, the in situ measured effective stresses are very small. This suggests that much of the decrease in void ratio may be due to a creep mechanism. Slurry consolidometer tests have been performed to measure the creep properties of fine tails and fine tails-sand mixes. Fine tails has a large creep index when compared to other clayey soils with values similar to that of organic silt.

Cavity expansion tests have been developed to measure the thixotropic gain in strength in the oil sand fine tails. Five different water content fine tails were tested at time intervals ranging from 0 to 450 days. The thixotropic strength increased quadratically with the age of fine tails and was still increasing at the end of the test period of 450 days. The thixotropic strength ratio was at a minimum at 150% water content. The water content of the fine tails significantly influenced the thixotropic strength development. The

change in strengths for the cavity expansion, vane shear and viscometer test procedures followed similar trends but the cavity expansion tests gave substantially higher strengths.

A self weight consolidation test in a 10 m high standpipe has been conducted for more than 10 years. The rate of consolidation in the ten metre standpipe was in good agreement with the predicted values based on the slurry consolidometer tests and the finite strain consolidation theory when the appropriate permeability relationship was used.

The wet landscape option is being considered as a long range disposal alternative for the oil sand fine tails by depositing them in the mined-out pits and capping them with water. A large strain consolidation model using laboratory test results was used to analyze the field behavior of fine tails in the wet landscape disposal option. The effects of fine tails depth, and provision of bottom drainage in the disposal option were analyzed. Effects of variations in the compressibility and permeability relationships on the model predictions were investigated. Variations in the compressibility characteristics had little effect on the predictions. However, small variations in the permeability characteristics caused severe differences in the predictions, especially for long term consolidation which suggests that the permeability of the fine tails governs its consolidation behavior.

ACKNOWLEDGMENTS

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1. INTRODUCTION

1.1 Statement of the Problem

In mining extraction operations, the resulting waste tailings are generally in the form of a slurry. Such slurried mineral wastes include oil sands fine tails, phosphatic clays, coal tailings, FGD sludge, alumina red mud and dredged materials. These wastes are commonly deposited hydraulically and contained in some type of pond-dyke arrangement which can cover an area up to tens of square kilometers. Design problems associated with these ponds include storage capacity, embankment stability, seepage, and land reclamation. The physical properties of the waste material which affect the design evaluations are the same as in any other geotechnical investigation: compressibility, permeability, and shear strength.

If the solid particles are essentially sand to silt sized, the slurry material will sediment rapidly to its final void ratio and little consolidation will occur as additional material is deposited above it. In this case measuring the required physical properties can be accomplished with standard field and laboratory tests. Many mineral slurries, however, contain a significant clay or colloidal fraction. The presence of clay can result in a high initial void ratio and a large volume change as additional loads are imposed. It has been found that Terzaghi's classical consolidation theory, which assumes infinitesimal strain, does not apply in these cases and it is essential to use a finite strain consolidation theory.

In Northern Alberta, oil sands deposits are mined and processed to remove oil from the sand mass. The resulting tailings are pumped to a containment dyke where the majority of the sand settles out, leaving the ponds filled with a water-clay-bitumen slurry termed as fine tails. Water is the major portion of the fine tails where, after sedimentation, it occupies approximately 10 to 15 times the volume of the mineral solids and bitumen combined. The fine tails, by economic necessity, must rely upon self weight consolidation for its densification. Self weight consolidation is slow in the Suncor and Syncrude tailings ponds and it is necessary to continually enlarge the containment ponds and dykes to hold the rapidly accumulating large volumes of material. It is necessary, therefore, to understand the consolidation behavior of the fine tails before an assessment of the effectiveness and feasibility of long range disposal plans can be made.

1.2 Oil Sands Tailings

1.2.1 Introduction

The Alberta oil sands (Figure 1.1), which exists in the Northern part of the province, contains approximately 900 billion barrels of crude bitumen of which 250 billion barrels can be economically exploited to produce light synthetic crude oil (Berkowitz and Speight, 1975). The Athabasca deposit is the largest of the three main deposits which underlie an area of 48,000 square kilometers. The Athabasca deposit has an area of 32,000 square kilometers and contains 869 billion barrels of bitumen (Outrim and Evans, 1978). The average thickness of the deposit varies from 6 to 90 m with about 10% having less than 45 m of overburden. This depth of overburden has made the deposit economically feasible for commercial oil sands surface mining operations. Details of the encompassing geology can be found elsewhere (Berkowitz and Speight, 1975; Isaac et al., 1982).

Athabasca oil sands is a Lower Cretaceous fine to medium grained uniform sand which is predominantly quartz and has 31% average porosity. The sand grains are covered by a layer of water, which are then enveloped by bitumen (Dusseault, 1977). The water and the bitumen form continuous phases throughout the oil sands structure. Gases are dissolved within the liquid phases and depending on in situ temperatures and pressures are occasionally present as a free gas (Scott and Kosar, 1984). The bitumen content (as a percentage of the bulk mass) ranges from 0 to 20% and averages about 11%. The amount of water (relative to the bulk mass) varies from 3 to 6% and averages around 5%. The commercial oil sands consist of approximately 70% sand and 14% fines (< 45 μm) with the fines occurring in clay-shale seams and lenses. Clay (< 2 μm) accounts for approximately 32% of the fines.

1.2.2 Oil Sand Operations

The two oil sands surface mining operations (Syncrude Canada Ltd. and Suncor Inc.) in the Athabasca oil sands deposit use a similar process for oil recovery. The overburden (surficial glacial deposits and the Clearwater Formation) is removed by truck and shovel and transported to waste dumps. The oil sands is mined using draglines or truck and shovel and conveyed to extraction plants. The material is processed using the Clark Hot Water Extraction Process, a froth flotation process. Oil is extracted by mixing disaggregated oil sands with water, raising the temperature to $85^{o}C$ with steam, buffering to 8.5 pH with NaOH, and mixing and separating with flotation (Dusseault et al., 1988). The bitumen is upgraded to a conventional refinery feedstock by coking to remove carbon and hydrotreating to add hydrogen (Dusseault and Scott, 1982). Details of the extraction process can be found elsewhere (Camp, 1977; Adams, 1985).

At Syncrude Canada Limited, 1 m^3 of oil sands feed requires 1.9 m^3 of water (or 15 m^3 water for 1 m^3 of crude synthetic oil). The resulting tailings stream occupies a volume of more than 2.5 times the original oil sands (Scott and Dusseault, 1982; Scott et al., 1985; MacKinnon, 1989). The tailings stream amounts to a volume of approximately 115 million m^3 per year (Scott et al., 1985; Fair and Hanford, 1986) and is composed of a mixture of sand, fines, hot water, caustic soda (NaOH), and unextracted bitumen (Bromwell Engineering Inc., 1983). This fluid waste is contained in a tailings pond in the northern part of lease 17. The pond is built around the Beaver Creek Valley and is enclosed by dykes of compacted tailings sand and flat upstream beaches.

The tailings stream at a solids content of 40% to 60% is dominated by sand sized quartz grains mixed with a small portion of fine grained materials and contains about 0.5% by total mass of unrecovered bitumen. Scott and Cymerman (1984) reported that at Syncrude Canada Limited, the tailings stream amounts to 130 x 10^6 tonnes per year and MacKinnon (1989) reported that from 1978 to 1987 the tailings solids amounts to 573 x 10^6 tonnes. The pond is designed to cover 22 square kilometers, with a projected surface water area of 17 square kilometers. MacKinnon (1989) also reported that approximately 44 x 10^6 m^3 of sand was deposited per year and from 1978 to 1987, 300 x 10^6 tonnes was deposited. The remaining solids form a thin slurry

(7% to 10% solids) that enters the pond. Approximately 14 x 10^6 m^3 of liquid fine tails is generated each year. Settling and consolidation results in a release of water that can be reused. About 70% of the water requirements are reclaimed from the pond. The fluid volume in the tailings pond is growing at a rate of about 0.25 m^3 per tonne of oil sands.

The major components of the tailings pond include water (clarified water in the surface zone and water in the fine tails); mineral solids such as sand, silt and clays; dissolved solids (organic and inorganic components, process chemicals, and leachates); and unrecovered bitumen. The tailings pond (Figure 1.2) shows stratification with several distinctive zones (MacKinnon, 1989). The free water zone (0 to 10 m depth) is a low density zone above the fine tails interface with a low suspended solids content (< 0.05%). This zone is well mixed and undergoes seasonal changes. A fine tails interface zone exists from 10 to 11 m with a rapid increase in solids content. An immature fine tails zone from 11 to 17 m depth is the upper 7 m of the fine tails where initial settling and consolidation of the fine tails occurs during the first 2 to 3 years after deposition. The mature fine tails zone (>17 m depth) is the lower fine tails zone where the later stages of consolidation occur. In this zone, the solids content increases with depth primarily through a suspension of coarser sand solids. The fine tails in this zone is greater than three years of age.

Sedimentation is an efficient process in the tailings pond (MacKinnon, 1981; Roberts et al., 1980). The fines settle rapidly to a density at which the particles form a soil structure in which the interparticle stresses impede further settlement. Additional reduction in volume can only take place by consolidation under selfweight which is very slow.

1.2.3 Oil Sands Fine Tails

Scott and Cymerman (1984) reported typical grain size distribution curves for the tailings stream and the fine tails which are shown in Figure 1.3. The grain size analysis of the fine tails shows that it is about 5% fine grained sand, 30% silt size and 65% clay size (Scott and Dusseault, 1982). The clay mineralogy of the fine tails reflects the average clay mineralogy of the McMurray Formation. Kaolinite and illite clays dominate, smectite is present as a minor ingredient and comes almost exclusively from the upper portion of the formation. Vermiculite is present in small amounts and comes from the lower half of the formation, a trace of chlorite and mixed layer clays are also present (Kasperski, 1992).

The fine tails mineral composition is consistent over time and place in the pond because the ore-body mineralogy is consistent and is blended before extraction. As well, mixing occurs during extraction, transportation, and deposition (Dusseault et al., 1989). Dereniwski and Mimura (1993) summarized that the fine tails is dominantly kaolinite (55% to 65%) and illite (30% to 40%) with minute traces of mixed layer clay minerals.

The bitumen content of the fine tails based on total mass averaged around 2% (MacKinnon and Sethi, 1993). If the bitumen is calculated as a percent of the mass of the mineral solids, its content is 6.5%. Devenny (1993) reported that the liquid limit ranges from 60% to 70% and the plasticity index varies from 30% to 40%. The range reflects the effects of ionic concentration, clay mineralogy and bitumen content. The higher values in Atterberg limits in general indicate greater bitumen content and finer grained fine tails. Plasticity of oil sands fine tails is compared with other mine tailings in Figure 1.4 (Carrier et al., 1983). Kaolinite dominates the slimes from the china clay tailings (Kessick, 1979). Phosphate slimes are composed of 25% to 33% clay sized minerals of which smectite is the primary clay mineral (Carrier et al., 1983). The tailings from the processing of bauxite to aluminum, known as red muds, are dominantly sodium alumino silicate, which comes from kaolinite.

The oil sands fine tails exhibit thixotropy which was first reported by Kessick (1979). The clay minerals present in the fine tails do not show significant thixotropic effects. The additives, intense agitation and the elevated temperature during the extraction process makes the clay particles well dispersed. Kessick (1979) documented three conditions necessary to form a gel structure: the presence of residual bitumen, a clay bound organic component to confer surface activity and clay particles well dispersed initially. Some of the organic material in the fine tails is intimately bound to the clays

(Scott et al., 1985; Kessick, 1979). Humic materials and asphaltenes are suggested to be the dominant organic material. Scott et al. (1985) reported an increase in organic content as grain size decreased and attributed the high gel strength to the high bitumen content as ordinary kaolinite/illite slurries do not show such increases. Danielson and MacKinnon (1990) concluded that the presence of bitumen does not appear to be a significant factor for the yield strength in viscometer testing.

Scott et al. (1985) reported that the significance of the thixotropic strength in the fine tails is reflected in the hindrance of the consolidation process at low stresses. The development of an effective stress without a decrease in void ratio retards the consolidation process under low self-weight stresses. According to Scott et al. (1985) thixotropy loses its significance as the effective stress increases.

1.2 Objectives of the Research Program

The main objective of this thesis is to study the consolidation behavior of oil sands fine tails. The specific objectives are to

1. determine the compressibility characteristics of fine tails
2. evaluate the permeability testing procedures and determine the hydraulic conductivity of fine tails
3. study the creep behavior of fine tails
4. study the thixotropic strength development
5. verify the finite strain consolidation theory
6. predict the fine tails behavior in a wet landscape disposal option

1.3 Scope of Thesis

Slurry consolidation tests are performed and analyzed to determine the compressibility, permeability and creep parameters. It was found that the compressibility of high void ratio materials is influenced by the initial void ratio. In order to understand the effect of initial void ratio, consolidation tests are performed on three materials; 20%, 25% and 30% solids content fine tails from the Syncrude tailings pond.

Cavity expansion tests are performed and analyzed to measure the undrained strength variation with time. The materials tested are at 20%, 25%, 30%, 40% and 50% solids contents. The tests are done on different ages of fine tails up to 450 days to determine the thixotropic strength properties.

A ten meter high standpipe test is monitored and the test results are analyzed to verify the finite strain consolidation theory. Two metre high standpipe tests are also conducted and analyzed. Analyses are performed to determine the relative significance of compressibility and permeability on the consolidation behavior of fine tails. From these analyses representative compressibility and permeability values are used in the finite strain consolidation theory to evaluate the wet landscape disposal option.

1.4 Organization of Thesis

Slurry consolidation tests are described in Chapter 2. This chapter presents the consolidation and permeability equipment, the fine tails material, the testing methods and the test results for fine tails.

Chapter 3 describes the permeability testing and the factors affecting the hydraulic conductivity of fine tails slurries. The hydraulic conductivity values for oil sands fine tails and fine tails - sand mixes are presented.

Chapter 4 explains the creep mechanisms and the factors affecting creep behavior. The creep parameters for oil sands fine tails are presented and compared with other clayey soils.

Cavity expansion tests and the thixotropic behavior of oil sands fine tails are described in Chapter 5. The cavity expansion test results are compared with vane shear test and viscometer test results.

The fine tails behavior in the wet landscape disposal option is described in Chapter 6. The effect of sand layers and the height of deposition is also shown. The effect of compressibility and permeability values on consolidation predictions are also discussed.

The last chapter integrates the different aspects of the consolidation behavior of fine tails that were discussed in the previous chapters and summarizes the main conclusions. The recommendations for future research are also given in Chapter 7.

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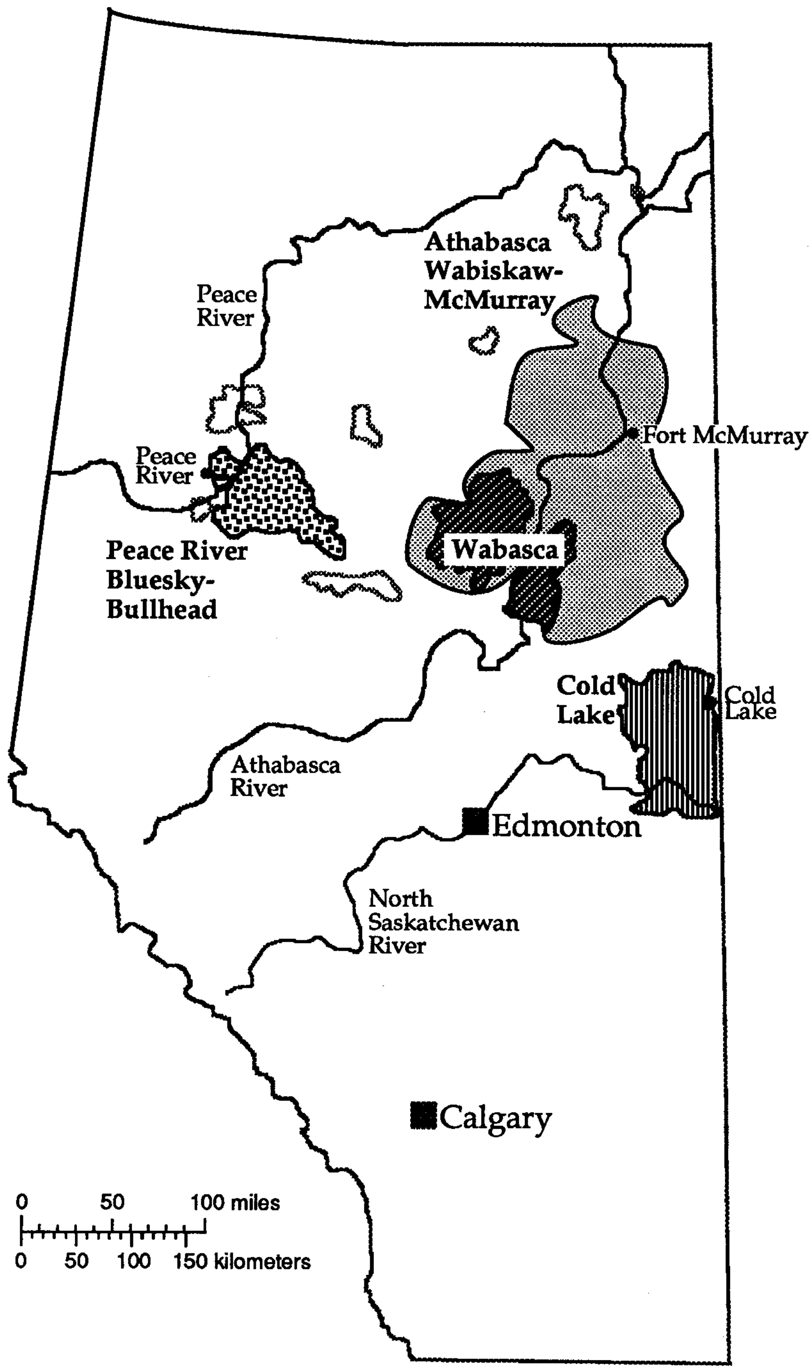

Figure 1.1 Alberta Oil Sands

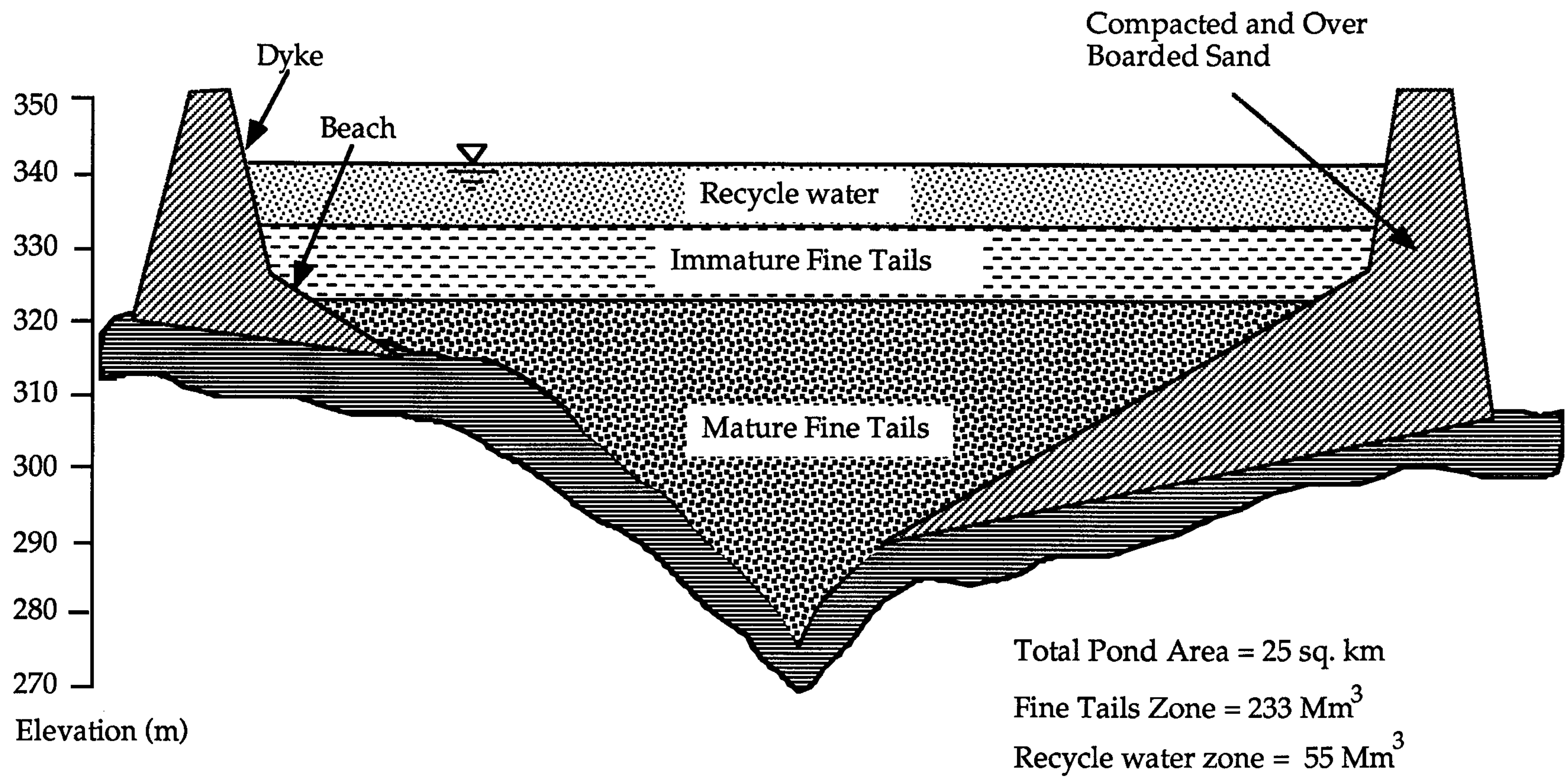

Figure 1.2 Cross Section of the Syncrude Tailings Pond
(modified after MacKinnon, 1989)

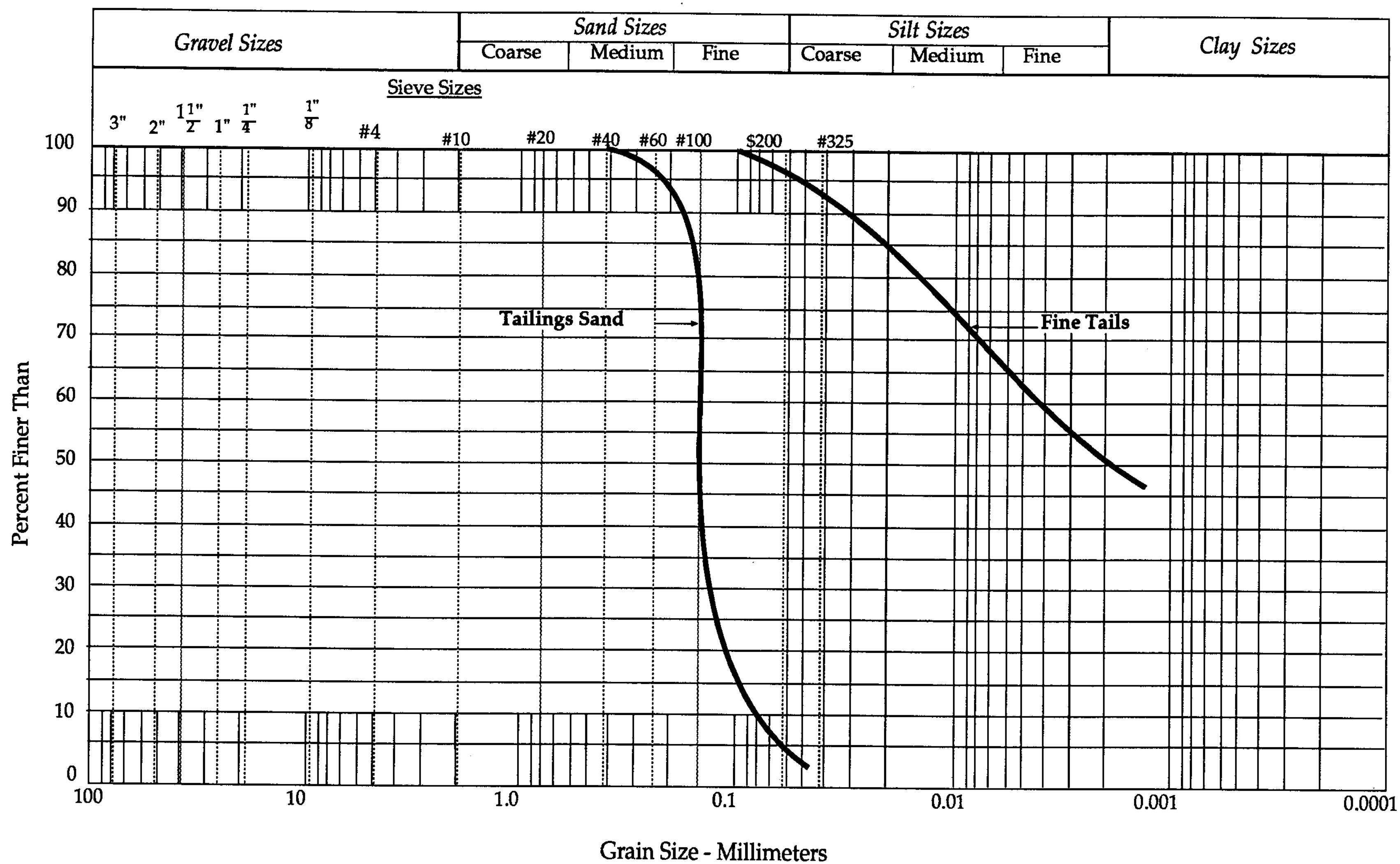

Figure 1.3. Typical Grain Size Distribution of Fine Tails (after Scott and Cymerman, 1984)

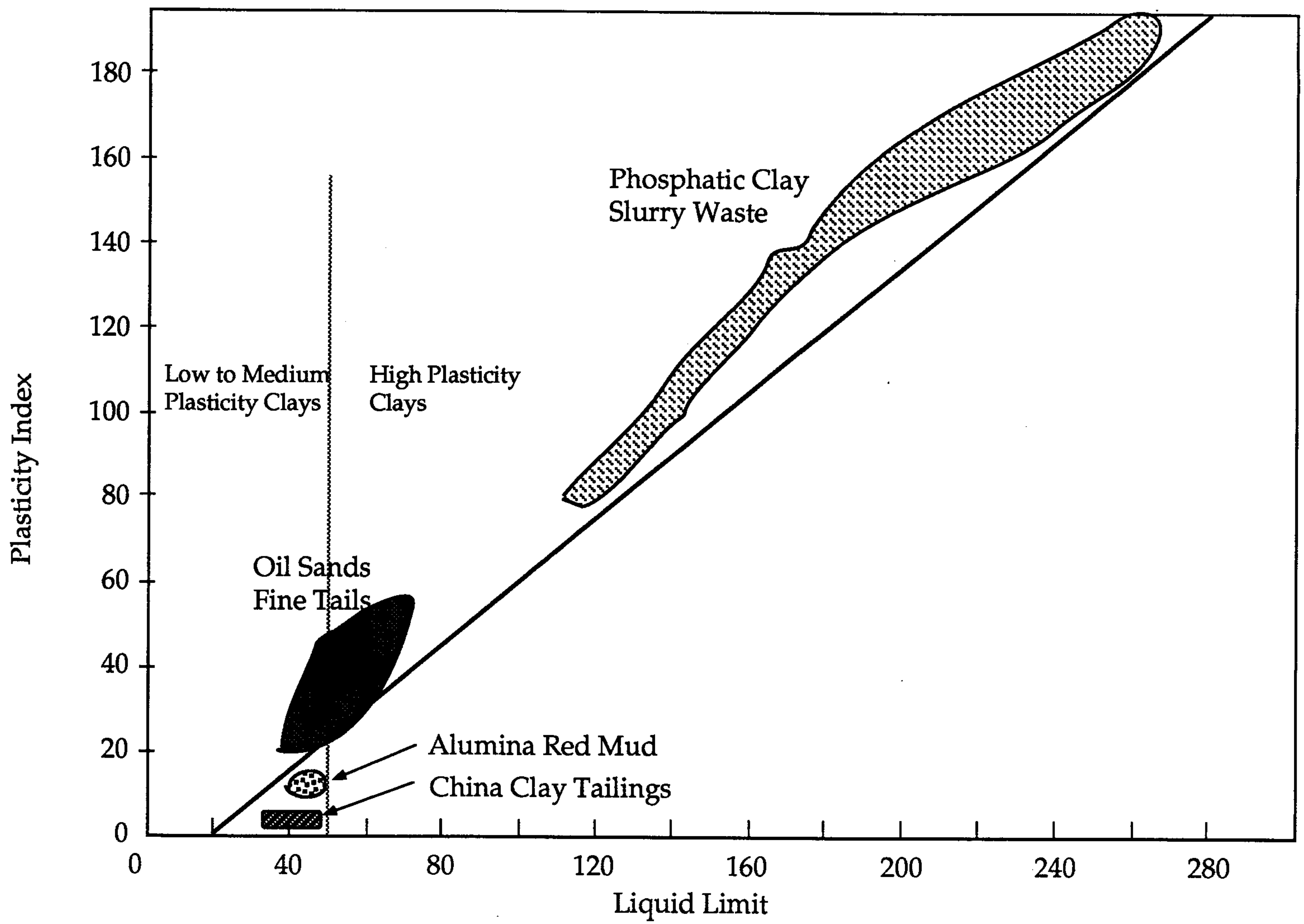

Figure 1.4 Plasticity of Mine Wastes (after Carrier et al., 1983)

2. CONSOLIDATION TESTING OF OIL SAND FINE TAILS[1]

2.1 Introduction

In mining operations, the waste material from a mineral processing plant is generally in the form of a slurry. The slurry is deposited hydraulically and contained in some type of pond-dyke arrangement. Design problems associated with these ponds include storage capacity, embankment stability, seepage, and land reclamation. The physical properties of the waste material that affect the design of the waste storage facility are compressibility, permeability, and shear strength. Many waste mineral slurries contain significant amounts of silt and clay fraction. The presence of clay can result in a very high initial void ratio and subsequently very large volume changes during consolidation.

In northern Alberta, oil sand deposits are mined and processed to recover heavy oil from two oil sand plants, Syncrude Canada Ltd. and Suncor Inc. The tailings streams are composed of about 85% sand and 15% fines by weight plus a small amount of heavy oil that bypasses the extraction plant. Syncrude and Suncor produce, respectively, about 480,000 mt/d and 170,000 mt/d of tailings, which range in solids content from 40% to 60%. The tailings sand forms the pond dikes and beaches and accumulates at a rate of 225,000 m^3/d. Approximately one-half of the fines in the tailings streams flow into the pond to form a fine tails deposit. After sedimentation and approximately 2 yr of consolidation, the fine tails form a deposit at about 30% solids, which accumulates at a rate of 45,000 m^3/d. Approximately 325 million m^3 of this fine tails are presently held in the tailings ponds. The material, by economic necessity, must rely on self-weight consolidation for its densification. Self-weight consolidation is very slow in the tailings ponds. The result is a necessity to continually enlarge the containment ponds and dikes to hold the rapidly accumulating large volumes of material. Therefore, it is necessary to

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[1] A version of this chapter has been published. Suthaker N. N. and Scott J. D. Proceedings of the International Land Reclamation and Mine Drainage Conference and the Third International Conference on the Abatement of Acidic Drainage, Pittsburgh, PA, April 24-29, 1994, 4: 399-406.

understand the consolidation rates and behavior of the fine tails before an assessment of the effectiveness and feasibility of long range disposal plans can be made.

The consolidation behavior of the fine tails is controlled by the compressibility, permeability, thixotropic strength, and creep properties of the material. Laboratory tests have been performed to measure these properties and their variations with porosity and time. The fine tails material is unique because of its properties and the vast volume that must be disposed in an acceptable environmental and economical manner. The consolidation and permeability equipment, the fine tails material, and the testing methods are described in the following sections.

2.2 Consolidation Testing

The standard oedometer test is developed for small-strain consolidation problems and generally adopts small-strain theory. In the oedometer test, constant loads are applied to the sample, and the settlement with time is observed for each load. This allows a relationship between stress and void ratio, coefficient of compressibility, and, by using theory, a coefficient of consolidation and then the hydraulic conductivity to be obtained.

There are two significant experimental difficulties in using the standard oedometer test for the fine tails. The main problem is the large strain that takes place. The other problem is the determination of hydraulic conductivity. Permeability testing may cause seepage-induced consolidation. There are several methods of determining the coefficient of consolidation and calculating the hydraulic conductivity from the laboratory data. All incorporate the assumption that the compressibility and the coefficient of consolidation are constant during each load increment. This assumption presents the major restriction to the applicability of the analytical procedures. The accuracy of the determination of the coefficient of consolidation is inversely proportional to the size of the load increment (Znidarcic et al. 1984).

2.2.1 Modified Consolidation Testing

Advanced testing techniques have been developed to overcome the problem of the test duration of large-strain oedometer tests. Such tests include the constant rate of deformation (CRD) test (Lee 1981), the constant hydraulic gradient (CHG) test (Lowe et al. 1969), and the constant rate of loading (CRL) test, all of which are faster than the step-loading oedometer test. These test procedures employ an inversion of the Terzaghi infinitesimal consolidation theory to yield usable results. Therefore, their applicability is restricted by this theory's assumptions (Znidarcic et al. 1984).

The CRD test has to be done at the field deformation rates since the test results are dependent on the rate of deformation. The assumptions required for the reduction of data include infinitesimal strain, constant coefficient of consolidation, and the assumptions regarding the relationships between either the void ratio and time or the void ratio distribution in the sample. Neither assumption can be validated (Znidarcic et al. 1984).

In the CHG test, the loading rate is continually adjusted through a feedback mechanism, such that the pore pressure at the undrained boundary remains constant; hence the hydraulic gradient is constant within the sample throughout the test. The assumptions include infinitesimal strains, constant permeability, a linear compressibility, and a constant void ratio throughout the sample (Znidarcic et al. 1984).

The CRL tests (Aboshi et al. 1979) and constant loading tests (Janbu et al. 1981) have similar assumptions and restrictions (Znidarcic et al. 1984). The seepage test (Imai 1979) does not rely on Terzaghi's theory for consolidation parameters. However, sample rebound at the completion of this test may lead to erroneous results.

The above-mentioned tests, except for the seepage test, with their analyses have been set up for soils that obey the assumptions involved in Terzaghi's infinitesimal consolidation theory. These tests are not applicable to highly compressible soils such as dredged materials and mine waste slurries, which undergo large deformations during consolidation. To overcome such setbacks, slurry consolidation tests have been developed for these very soft materials.

2.2.2 Slurry Consolidation Testing

Slurry consolidometers have been developed to allow large deformations during consolidation and to allow permeability tests to be performed on the samples. The test is conducted similar to the oedometer test in which a step-loading procedure is applied, allowing for the direct determination of the effective stress-void ratio relationship. The permeability-void ratio relationship can be determined by conducting permeability tests on the sample after each load increment. Seepage-induced consolidation can be prevented using a load-fixing device. Therefore the major difficulty remaining with the procedure is the test duration.

Though several procedures have been developed to decrease the time involved in consolidation testing of clayey soils, they involve inverting the complex finite-strain consolidation theory, and therefore their usefulness depends on the assumptions used in the inversion process. Therefore in this research, a long test duration was accepted and a slurry consolidometer with step loading procedure was used.

Imai (1981) noted that for highly active soil slurries that undergo hindered settling during the sedimentation phase, the initial void ratio at the start of consolidation is strongly dependent on the initial void ratio of the slurry. In Imai's study, several compression curves were found in the low-pressure range (less than 0.1 kPa). Above this effective stress value, there was a unique compression curve. To investigate the effect of the initial void ratio on the consolidation of oil sand fine tails, three different initial solids contents were employed in this research.

2.3 Permeability Testing

To understand the consolidation properties of a soil, laboratory testing is necessary to determine its hydraulic conductivity. For soils undergoing large changes of void ratio, it is necessary to determine the hydraulic conductivity as a function of the void ratio.

2.3.1 Indirect Methods of Measuring Permeability

Indirect methods of determining the hydraulic conductivity are carried out by inverting a consolidation theory and applying it to the rate of compressibility data obtained from a consolidation test. The most common method of indirectly determining the hydraulic conductivity in the standard oedometer test is either from the logarithmic time plot or square root time plot of the consolidation of a soil under a constant stress. These methods of determining hydraulic conductivity have been found to be unacceptable for slurry consolidation because of experimental (e.g., high strain rates) and analytical reasons (Terzaghi's assumptions).

Tavenas et al. (1983) indicated that the back-calculated values of permeability underestimated the measured values by up to six times for soft clays. The reason for the difference is the assumptions in Terzaghi's consolidation theory. The assumptions that are mostly violated are constant permeability, constant compressibility, and hence a constant coefficient of consolidation. Tavenas et al. concluded that indirect methods are unacceptable in determining the permeability characteristics of soft clays.

2.3.2 Direct Methods of Measuring Permeability

The direct method involves forcing a fluid through the soil and monitoring either the rate of flow through the soil or the hydraulic head changes induced by it. The two most widely used methods of directly determining the hydraulic conductivity of a soil are the constant-head and the falling-head permeability tests.

The advantages of these two tests are their simplicity in testing procedure, apparatus, and evaluation of the test data. The disadvantage of the conventional methods is the length of time required when performing low-gradient permeability tests. When direct methods are used for highly compressible slurries, low gradients must be used since large gradients induce consolidation due to seepage. The falling-head test is not feasible with low gradients.

A constant-head technique was chosen for this research because it allows for extremely small head drops during the test and because of its ability to monitor time effects on flow rate. A variable-head test is not feasible for both of these reasons.

2.4 Experimental Program

To determine the compressibility and permeability data for the oil sand fine tails, a large-strain slurry consolidometer was used (Figure 2.1). This apparatus allows a void ratio-effective stress relationship and a permeability-void ratio relationship to be obtained.

A step-loading procedure, similar to a standard oedometer test, was employed owing to its simplicity and the ability to perform permeability testing after the completion of consolidation under each load increment. A constant-head technique was employed for the permeability portion of the test. Since a change in pore fluid chemistry may affect the permeability, tailings pond water was used as permeant to achieve consistent results with the field conditions.

Common to all slurry consolidation test methods is the danger of consolidating the slurry during the permeability test. Consolidation occurs when the stresses introduced by the seepage force of the permeant are greater than the stress under which the sample was previously consolidated. To overcome this problem, a clamping system was used to fix the loading ram, which sits on the soil, to prevent its movement. This system then allows an upward hydraulic gradient to be applied to the sample, up to a gradient where the induced seepage stress is equivalent to the previously applied consolidation pressure.

Oil sand fine tails with three different initial solids contents (20%, 25% and 30%) were tested in the slurry consolidometers. Liquid and plastic limits of fine tails are approximately 59% and 23%, respectively. Bitumen content by mass of the fine tails is 3% of the mineral solids. The applied stress increments were equal to the previously applied stress. The consolidation tests and permeability tests lasted for up to 2 yr. This elapsed time is from the start of the self-weight consolidation stage to the end of the last permeability test of the final load increment.

2.5 Results and Discussion

Figure 2.2 shows a typical change of void ratio with elapsed time. For some of the smaller applied increments, it was necessary to stop the consolidation before the displacement had leveled out on a logarithm time plot. Although on the log time plot the material appears to be rapidly consolidating, the material has entered the 10^4-min log cycle, and any additional change in void ratio would be very small.

Excess pore pressures were measured at five ports using a pressure transducer. The ports are numbered from the top (Figure 2.1). Figure 2.3 shows a typical excess pore pressure variation with time. Initially, the consolidation stress is the midheight self-weight in the sample. Subsequently, the stress is the midheight self-weight plus the stress applied by the loading ram.

Figure 2.4 shows a typical relationship of flow velocity with elapsed time during a permeability test. The flow velocity decreased until it approached a steady-state value. The flow velocity at this steady state was used to determine the hydraulic conductivity. Several permeability tests were run at different gradients, and this phenomenon was found to occur repeatedly. This finding suggests that the decrease in flow velocity may have been triggered by seepage forces and is reversible. The bitumen in the fine tails might account for this phenomenon. Although considered as a solid in calculations, the bitumen is deformable under stress and not totally rigid.

This deformable quality could allow bitumen to move to block pore throats while being subject to a seepage stress. Once the seepage has stopped, the bitumen would revert to its initial position or shape. A similar drop in flow velocity was also experienced at lower void ratios; however, the drop in flow velocity from initial to steady state became less as the void ratio decreased. It would be expected that the flow velocity drop would be less, because at very low void ratios very little movement of the bitumen can take place.

Figure 2.5 depicts the compressibility of the fine tails. The results for the 30% initial solids content test were obtained from Pollock (1988). It is clear from Figure 2.5 that the effect of initial void ratio is substantial in the low effective stress range that exists in the tailings pond. When the effective stress reaches about 100 kPa, the effect of initial void ratio becomes small. The conclusion is that the compressibility of the fine tails is dependent on the initial void ratio of the sample. Usually, in soil mechanics, the recompression of overconsolidated soils follows a different void ratio-effective stress relationship until the virgin compression state is reached. However, fine tails are not overconsolidated soils; they are underconsolidated and therefore should follow the virgin compression line irrespective of the initial void ratio. The experimental results suggest that the process of aging develops different fine tails microstructures. Therefore a time-dependent factor has to be taken into account which connects the individual void ratio-effective stress relationships.

Figure 2.6 shows all the hydraulic conductivity values for different void ratios. Unlike the void ratio-effective stress behavior, void ratio-permeability behavior does not appear to be influenced by the initial void ratio. It suggests that different micropore structures control the permeability and the compressibility.

2.6 Conclusions

A slurry consolidometer has been developed to allow large deformations during consolidation and to allow permeability tests to be performed on the fine tails. Development of a top cap clamping system has permitted permeability testing to be conducted on the high-void-ratio slurries without introducing seepage-induced consolidation. The permeability test results revealed a time-dependent flow discharge at a constant gradient. The flow velocity would drop up to two orders of magnitude before reaching a steady-state value. The steady-state values were used to calculate the hydraulic conductivity.

Unlike normal soils, the compressibility of the fine tails is controlled by the initial void ratio of the sample, which suggests that aging changes the microstructure of the fine tails and hence the compressibility. The above analysis suggests that there is a time-dependent parameter involved in the compressibility of fine tails. The older the fine tails, that is, the longer they have consolidated in the tailings pond, the smaller the void ratio they will reach under an applied effective stress. Therefore a single void ratio-effective stress relationship is not sufficient to describe the consolidation behavior. On the other hand, the permeability is not influenced by the initial void ratio, which suggests that permeability and compressibility are controlled by different elements of pore structure.

The above properties have been measured to allow the development of an analytical model for the long-term consolidation behavior of fine tails. The design of the eventual disposal and reclamation procedures and facilities for the oil sand fine tails requires such an analytical model.

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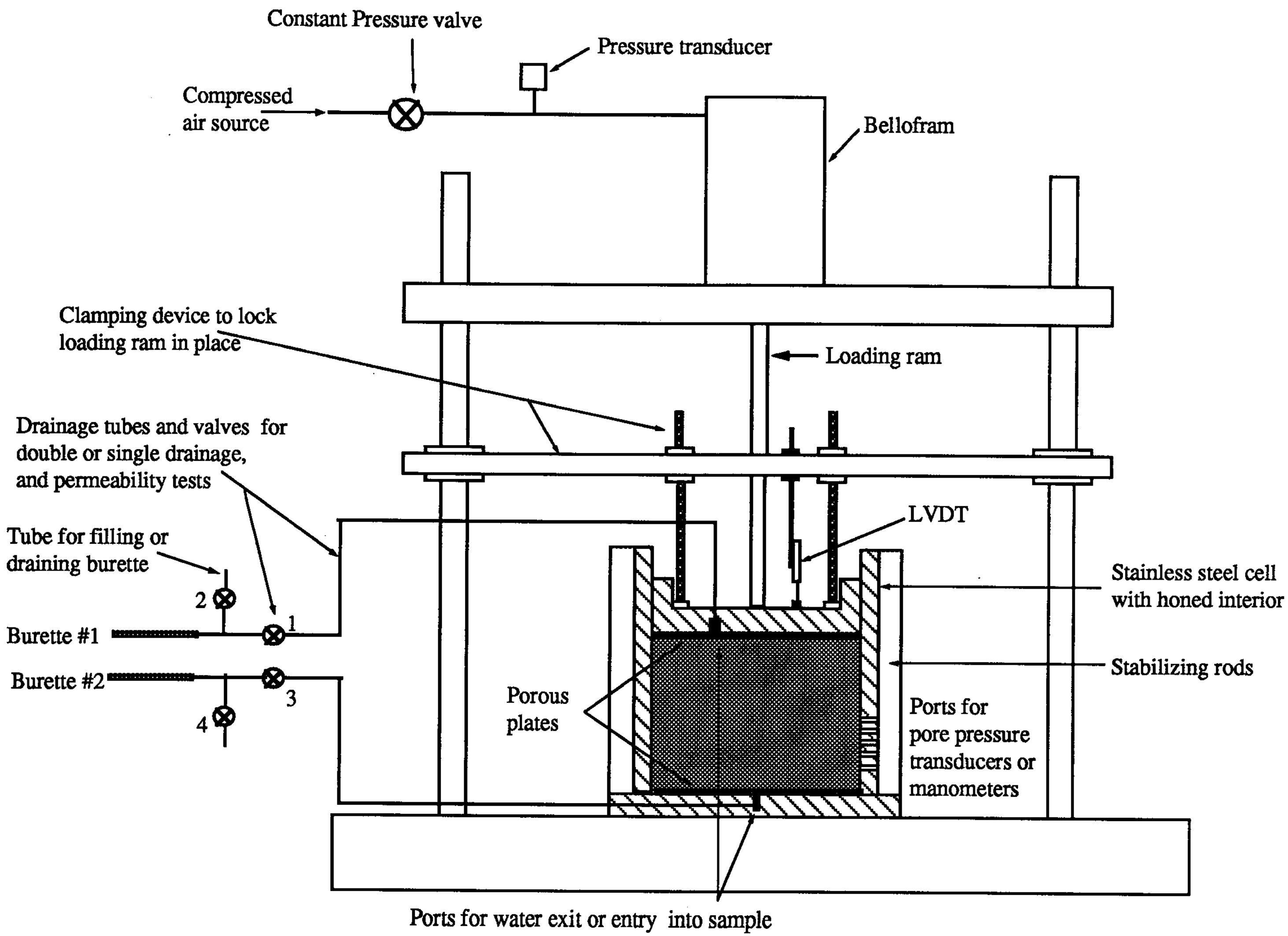

Figure 2.1 Slurry Consolidometer

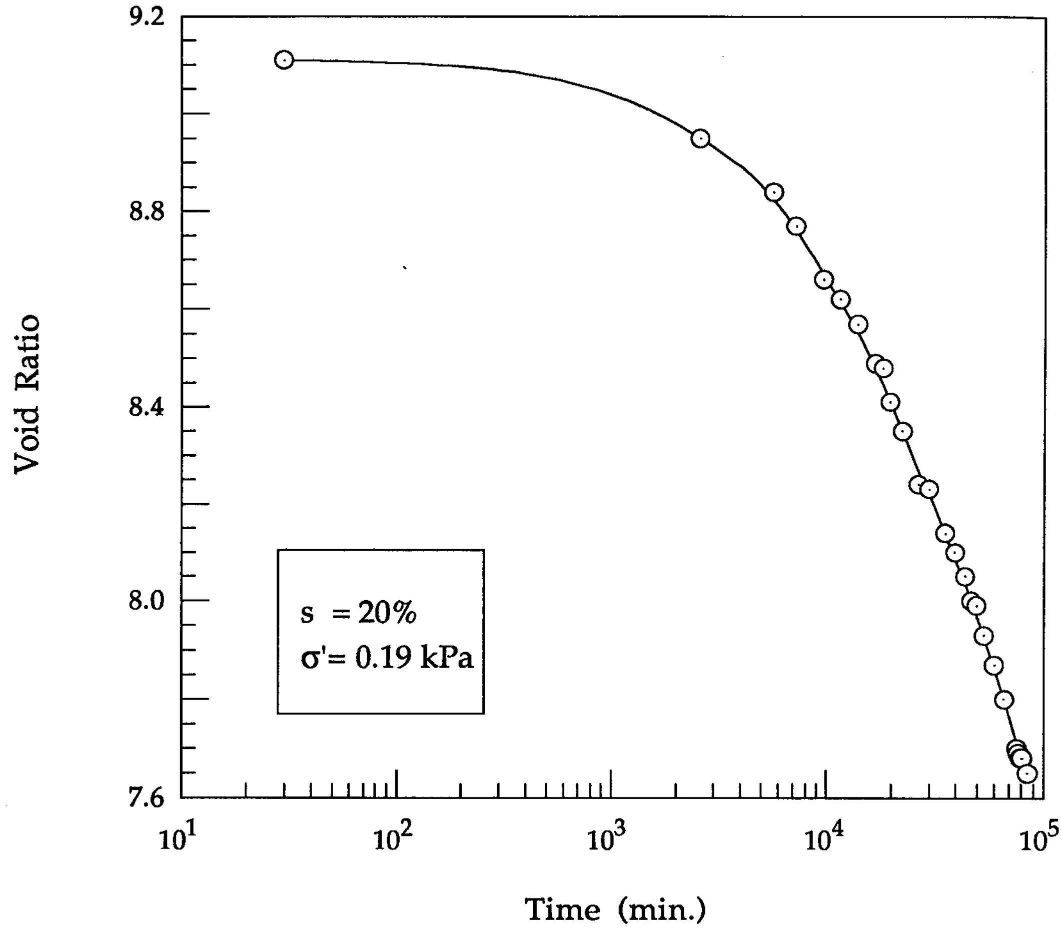

Figure 2.2 Void Ratio Variation with Time

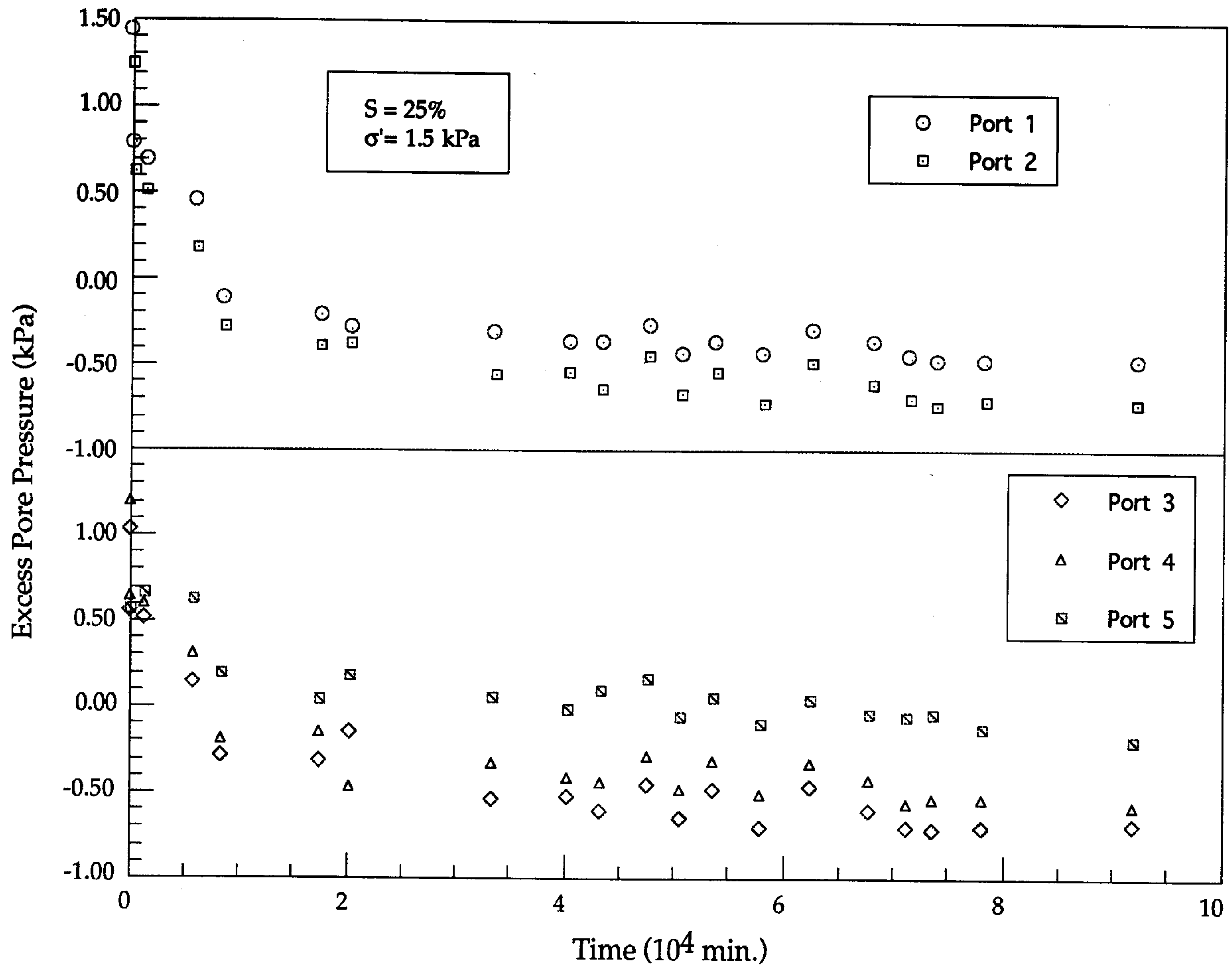

Figure 2.3 Excess Pore Pressure Variation with Time

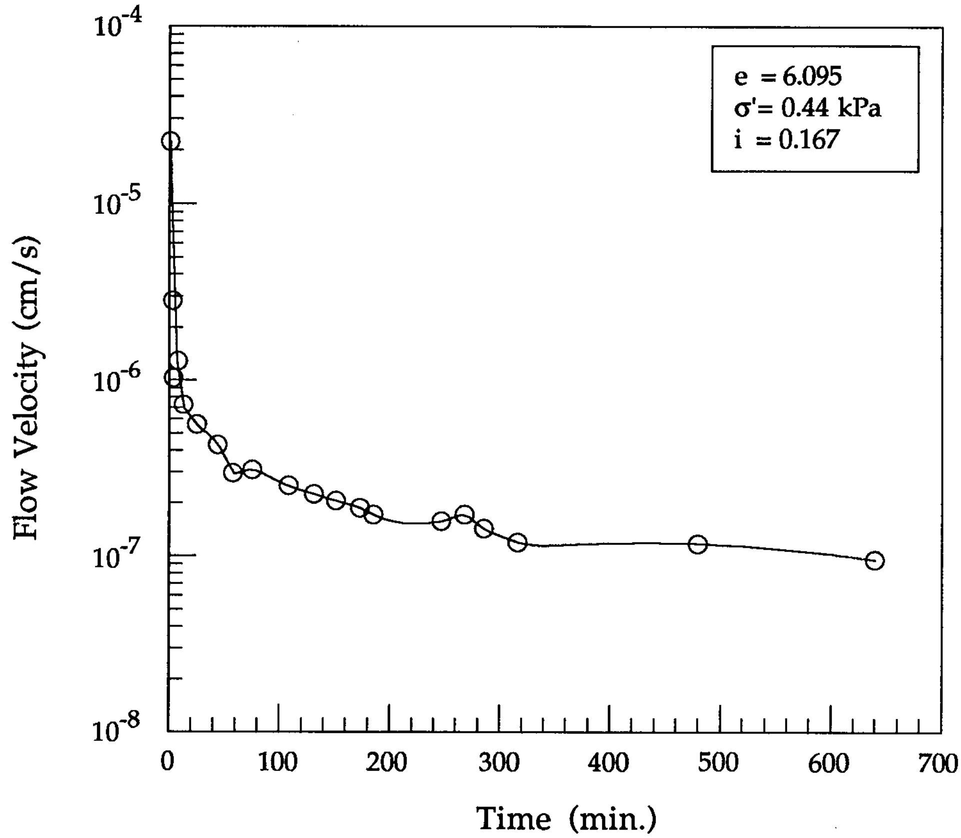

Figure 2.4 Variation of Flow Velocity with Time

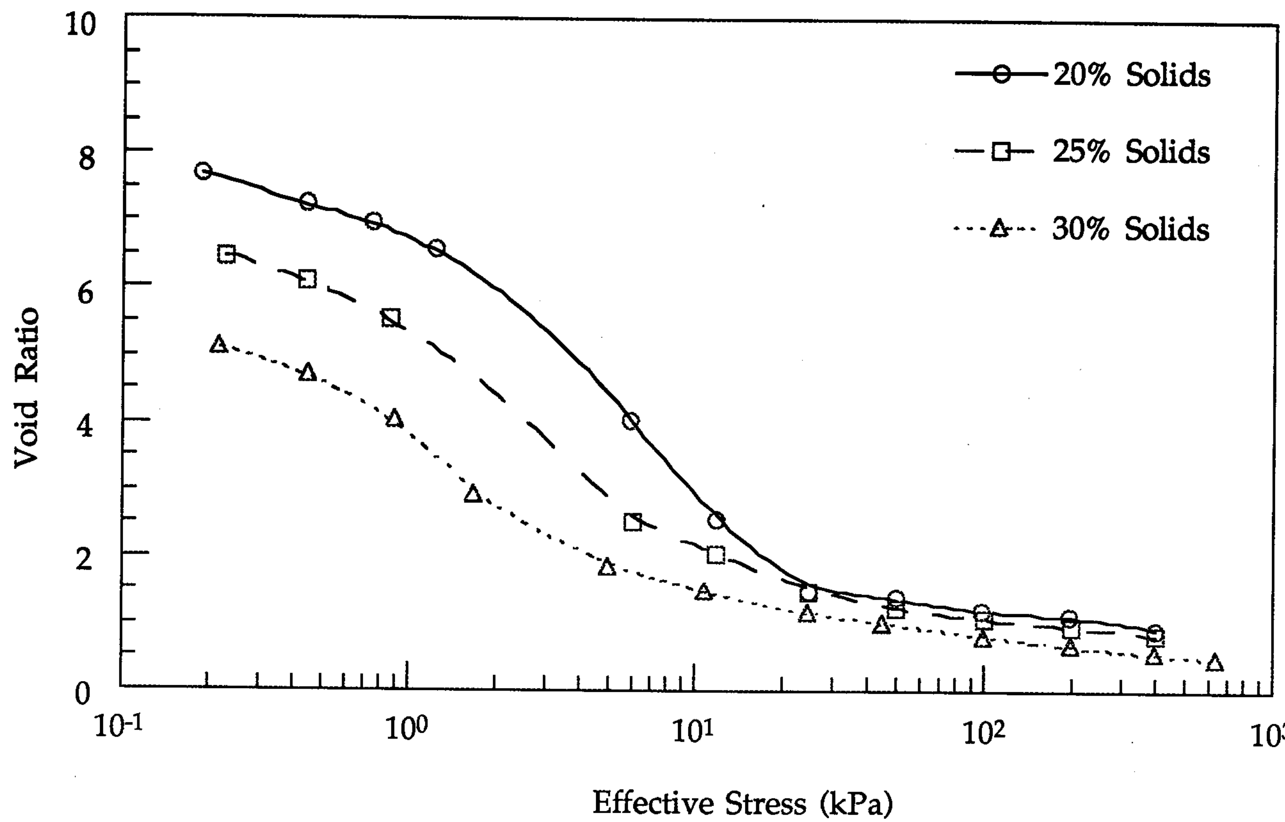

Figure 2.5 Compressibility of Fine Tails

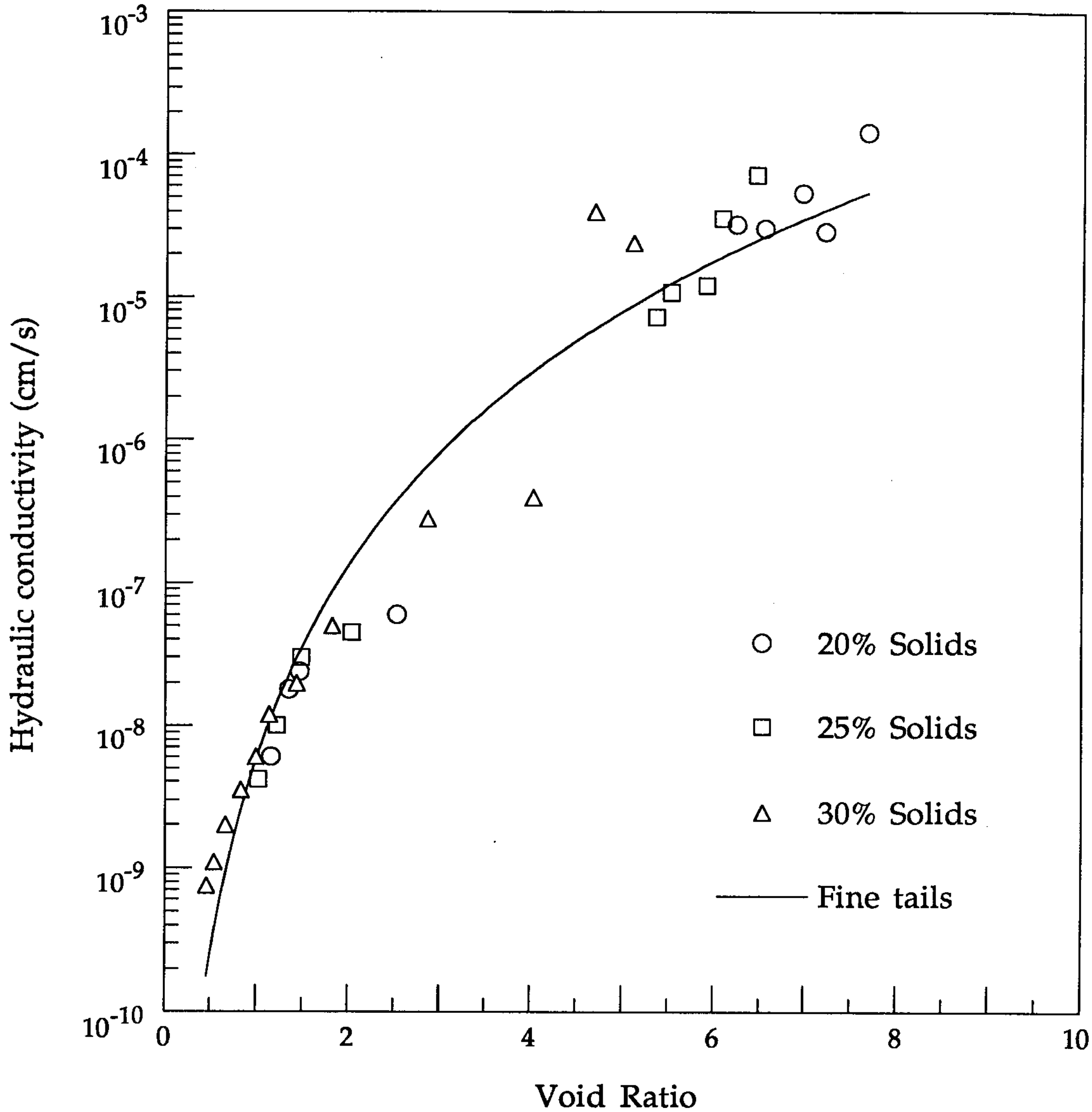

Figure 2.6 Permeability of Fine Tails

3. MEASUREMENT OF PERMEABILITY IN OIL SAND TAILINGS [1]

3.1 Introduction

In mining industries, the resulting waste stream is often in the form of a slurry. In oil sand processing the fine tails, essentially a mixture of fine sand, silt and clay size particles in water with a trace of bitumen, are confined in tailings ponds. To assess the effectiveness and feasibility of long range disposal plans for fine tails, an estimation of the consolidation rates and amounts is necessary. Since fine tails undergo large settlements during consolidation, it is necessary to use a large strain consolidation theory to analyze their consolidation behavior. The large strain consolidation theories are based on the permeability - void ratio relationship and effective stress - void ratio relationship (Gibson et al. 1967).

There are several methods of determining the permeability - void ratio relationship in the laboratory. Hydraulic conductivity of slurries can be determined either directly or indirectly. The direct method involves forcing a permeant through a specimen and monitoring the rate of flow, or the hydraulic head changes induced by it. Indirect methods of determining the hydraulic conductivity are done by inverting a consolidation theory and applying it to the data obtained from a consolidation test. The different testing methods (direct and indirect) are reviewed in this paper. Several problems have been encountered in the permeability measurement of fine tails due to its high water content and bitumen content.

The objective of this study is to establish a method for determining the permeability of fine tails. This paper describes the test apparatus used in this study and the testing procedure. Also discussed are the nature of the fine tails material and the influence of factors affecting the permeability of fine tails and fine tails - sand mixes: flow velocity, hydraulic gradient, fines content and bitumen content. Representative hydraulic conductivities for oil sand fine tails and for nonsegregating fine tails - sand mixes are determined.

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[1] A version of this chapter has been submitted for publication in Canadian Geotechnical Journal

3.2 Indirect Methods of Determining Hydraulic Conductivity

There are several kinds of consolidation tests available such as the step loading test, controlled gradient test, constant rate of deformation test, constant rate of loading test (Znidarcic et al. 1984) all of which use the inversion of a consolidation theory to determine the hydraulic conductivity. The most common method of indirectly determining the permeability is by inverting Terzaghi's theory. Olson and Daniel (1981) reported that for this method, the measured to back calculated permeability ratio ranged from 0.9 to 5. Lun and Parkin (1985) gave the possible reasons for such variations as the length of duration of the previous load increment, and the load ratio increment. Tavenas et al. (1983) indicated that the back calculated values underestimated the measured values up to 6 times, attributing such differences to the assumptions of Terzaghi's consolidation theory. The assumptions most often violated are constant permeability and constant compressibility. The study by Tavenas et al. (1983) concluded that indirect methods are unacceptable in determining the permeability of highly compressible natural clays.

Several authors have proposed the indirect measurement of permeability using finite strain consolidation theories (Tan et al. 1988; Znidarcic et al. 1986; Been and Sills 1981). All these studies suffer from the restrictive assumptions made to solve the equations limiting their applicability to problems where linear or constant material properties are good approximation of real behavior. Therefore, only the direct measurement of permeability has been used in this study.

3.3 Direct Methods of Determining Hydraulic Conductivity

3.3.1 Constant Head and Falling Head Test

Traditionally, permeability testing of soils is conducted by using constant head or falling head methods in the laboratory. These two methods have been widely used in geotechnical laboratories owing it to the simplicity and the availability of equipment at reasonable cost (Aiban and Znidarcic 1989). The constant head test is usually preferred for the permeability measurements of granular materials because the flow reaches the steady state rapidly and the test can be completed in a short period. However, it takes considerable time to complete a test with fine grained soils and, in such instances the falling head test is generally used (Olson and Daniel 1981).

Several disadvantages inherent to both constant and falling head tests have been discussed by Olsen et al. (1985), and Hardcastle and Mitchell (1974). In both methods, flow rates are obtained by conventional volume measurement techniques where the maximum resolution is of the order of 10^{-3} ml. For this resolution, accurate measurements of hydraulic conductivity of soil with low permeability can be achieved only if the imposed gradients are very high or the tests last for a long time (Alva-Hurtado and Selig 1981). If the burettes are replaced by capillary tubes in order to increase resolution, contamination of the tubes will lead to a non zero contact angle between the water and the glass which will affect the imposed gradient and produce erroneous results. (Olson and Daniel 1981).

The higher imposed gradients will usually produce a large variation of effective stress in the sample causing it to become less homogeneous (Aiban and Znidarcic 1989). In the falling head test, the hydraulic gradient changes continuously with time. Therefore, the falling head test is always done in a transitional state and the effective stress within the sample changes continuously which in turn changes the volume and, therefore, permeability. Analysis of the falling head test accounts for the changing gradients but the change in corresponding volume and permeability are usually ignored. Several modifications have been proposed for falling head tests such as the rising tail water test and the automatic falling head permeameter test but all use the same falling head test principle (Tan 1989; Daniel 1989).

3.3.2 Flow Pump Test

Olsen (1966) proposed the flow pump technique for measuring permeability of fine grained soils. In the flow pump test a known constant quantity of flow is forced through the sample by the pump and the corresponding pressure difference is measured by a differential pressure transducer, which is used to determine the hydraulic gradient. This is exactly the opposite concept of the conventional constant head test in which a known constant hydraulic gradient is imposed across the sample and the corresponding flow is measured. A pump with a very slow flow rate must be used (Olsen et al. 1985). Aiban and Znidarcic (1989) reported that the constant head test and flow pump test yield the same hydraulic conductivity values when the sample is tested under similar conditions.

3.3.3 Restricted Flow Test

Sills et al. (1986) proposed a restricted flow test which continuously determines permeability during consolidation and also had the provisions for a separate permeability measurement at the beginning or the end of a consolidation test. In the restricted flow test, the total stress increment is applied to one face of the sample and one way drainage is allowed from that face through a restrictor. Pore pressures at the drained and undrained faces of the sample are measured using transducers. The hydraulic gradient can be calculated using the sample height and the pore pressure difference at any time, and the flow rate can be obtained either by direct measurement or by calculation from the sample compression. The main difficulty of this approach is achieving an accurate measurement of the difference between the pore pressures on the undrained and drained faces. Initially, the two pore pressures are large (similar to the total stress) and pressure transducers that can monitor these will not provide an accurate measurement of the small difference between the pore pressures. A differential pressure transducer may be used to measure the difference directly.

3.3.4 Seepage Test

In this test, the specimen is subjected to the seepage of water by the application of a constant head difference across the specimen, so that the pore pressure distribution within the specimen is measured at different points. The flow of water through the specimen is measured, and the test continues until steady state pore pressure readings are obtained. After the steady state condition is established, seepage is stopped and the specimen is then sliced to obtain the void ratio distribution in the sample. During the slicing of the specimen some rebound will occur and therefore the measured void ratios may be higher than those during the steady state condition and must be corrected with a material balance calculation. From the pore pressure distribution, the hydraulic gradient can be calculated, and by using the measured flow rate, the void ratio - permeability relationship can be obtained.

3.4 Permeability Testing of Fine Tails Slurries

In choosing a test procedure for this research, the fine grained nature of the fine tails placed restrictions on the method chosen. High hydraulic gradients, which occur during the falling head test, cause consolidation by the seepage forces and, hence, make this test undesirable. Also, time effects on flow velocity cannot be studied. The constant head test which was used allows for extremely small head drops and allows the study of time effects (Olsen et al. 1985). Though the flow pump test was considered, it could not be used because the flow rate in this study is in the range of 10^{-5} to 10^{-8} cm/s. In the tests discussed in this study, the initial sample heights were about 26 cm while the head differences applied across the sample were about 2 to 5 cm .

Although the constant head test is usually performed in oedometer and triaxial cells (Leroueil et al. 1992), a triaxial cell could not be used since the sample could not stand by itself because of the high initial water content of the fine tails. The standard oedometer cell cannot be used for fine tails because it undergoes large strains during consolidation and the small sample thickness used in the standard oedometer is not adequate for consolidation and permeability measurements. The slurry consolidometer was designed so that permeability measurements could be taken at different void ratios by consolidating the sample under load increments.

3.5 Testing Equipment

Figure 3.1 shows the experimental setup, with the slurry consolidometer about 30 cm in height and 20 cm in diameter. When performing permeability tests on fine tails slurries, the applied hydraulic gradients can cause consolidation during the test (Pane et al. 1983). To overcome this, a clamping device consisting of a horizontal steel bar (50 mm by 50 mm) fastened to two vertical frame rods, was set up to prevent consolidation. Two steel threaded rods (9.5 mm) were fastened to the top cap and were allowed to travel vertically through the steel bar through bored holes. To prevent any further movement at the end of consolidation increment, the two rods are clamped to the steel bar. The LVDT was kept in place to monitor the exact location of the top cap throughout the process in order to ensure any occurrence of volume change during permeability testing. Flow in and out of the sample is measured during permeability testing to ensure any occurrence of volume change of sample.

This apparatus ensures that hydraulic gradients could be used up to the value that causes the seepage pressure to equal the previously applied stress (σ'), where the maximum head difference (Δh) for each increment is σ'/γ_w. Since the top cap is fixed in space and the soil beneath is consolidated under a stress σ', the soil will not further consolidate unless subjected to a stress (seepage pressure) greater than σ'.

It was possible to monitor the flow with horizontal burettes whose inside diameter was small enough to maintain a vertical meniscus because the flows were small (initially due to the low gradients and then due to low permeability). Burettes of the same size were used to monitor the inflow and outflow eliminating the need for meniscus correction. The burettes used were either 5 or 10 ml capacity depending on the flow, and a digital stopwatch was used for timing the flow during the test.

The type of fluid used in permeability tests can influence the measured hydraulic conductivity values. Budhu et al. (1990) have reported that when clay rich soils are saturated with an organic fluid the permeability measured with the organic fluid as permeant is greater than the measured permeability when the soil is saturated with water and water is the permeant. The permeant was selected to determine the hydraulic conductivity values consistent with those in the field. The tailings pond water was used as permeant for the fine tails and water collected during the consolidation test was used for nonsegregating tailings and fine tails-sand mixes for the permeability tests.

At the end consolidation under each load increment, the top cap was fixed in place by means of the clamping system as previously described. The end of consolidation was determined by using the measured settlement with time and measured pore pressures with time. Once the desired height difference (hydraulic gradient) between the inflow and outflow burettes was set, the valves were opened and the flow rate was monitored. Knowing the surface area (A), volume of flow (ΔV), and time interval (t), the hydraulic conductivity (k) can be calculated from a rearrangement of Darcy's law

$$k = \frac{\Delta V}{i\,A\,t} \qquad (3.1)$$

where, i is the hydraulic gradient equal to the hydraulic head difference (Δh) divided by the height of the sample. Several tests were done with different hydraulic gradients at each void ratio in order to check Darcy's law, after which the load was increased to the next load increment and the sample was allowed to further consolidate under the load increment. After the consolidation ceased, the procedure was repeated to measure the permeability at the new void ratio.

3.6 Test Materials

In northern Alberta, Canada, open pit mining to produce synthetic crude oil from oil sand is carried out on a two large scale operations, by Syncrude Canada Ltd. and Suncor Inc. The waste tailings stream is composed of about 85% sand and 15% fines at solids contents from 40% to 60%. The tailings pond dykes and beaches are formed by the sand and some fines. Approximately one-half to two-thirds of the fines and most of the water flow into ponds to form fine tails deposits. Approximately 400 million cubic metres of fine tails are presently held in the tailings ponds. Since the grain size distribution, bitumen content, and mineralogy affect the permeability of fine tails, their material properties have to be characterized. The fine tails will be referred to as Syncrude fine tails or Suncor fine tails depending on their mine origin. In the oil sands industry, fines are defined as < 45 μm and this definition is used here. Silt size range is from 2 μm to 45 μm and clay size is < 2 μm.

In this study, the 20% and 25% initial solids content fine tails consist of about 3% fine grained sand, 42% silt and 55% of clay size particles, while the 30% initial solids content fine tails has 8% sand, 45% silt and 47% clay size particles. The sand content in the tailings pond increases with depth and solids content. The clay mineralogy of the fine tails reflects the average clay mineralogy of the clay-shale strata in the McMurray Formation, which is dominated by kaolinite and illite clays. Smectite and vermiculite which come almost exclusively from the upper and the lower half of the formation respectively are present in small amounts and a trace of chlorite and mixed layer clays are also present in fine tails (Kasperski 1992). Dereniwski and Mimura (1993) reported that the fine tails are dominantly kaolinite (55 - 65%) and illite (30 - 40%) with minute traces of mixed layer clay minerals.

The bitumen content of the fine tails, based on the total mass of the fine tails, averages around 2% (MacKinnon and Sethi 1993). If the bitumen is calculated as a percent of the mass of the mineral solids, its content is 6.5%. The specific gravity of the fine tails varies between 2.1 and 2.5 due to varying amounts of bitumen which has a specific gravity of 1.03. If the bitumen content of the fine tails is known, the following equation can be used to calculate the specific gravity of the bulk fine tails,

$$G_{FT} = \frac{1+b}{b/G_b + 1/G_s} \tag{3.2}$$

where, G_{FT} is specific gravity of fine tails, G_b is specific gravity of bitumen, G_s is specific gravity of mineral grains which is 2.65, and b is bitumen content. The average unit weight of fine tails is about 12 kN/m^3. Devenny (1993) reported that the liquid limit of fine tails ranges from 60% to 70% and the plasticity index varies from less than 30% to 40%. In this study, the liquid limit varied between 40% to 60% and plasticity index varied between 20% to 35%. The range in Atterberg limits reflects the effects of ionic concentration, clay mineralogy and bitumen content. The higher values in Atterberg limits probably indicate greater bitumen content and high clay contents.

The permeability tests were performed on fine tails samples with different initial solids content, which is mass of solids divided by the total mass with bitumen considered as solids. The effect of hydraulic gradient was investigated in four slurry consolidometer tests with the fine tails at 20% initial solids content. The effect of bitumen was studied with fine tails of 10% initial solids (Bromwell Engineering Inc. 1983) and three slurry consolidometer tests were performed with 20%, 25% and 30% initial solids. In order to study the effects of the presence of sand in fine tails, three different fine tails - sand mixes were studied (Pollock 1988) (Table 1). The grain size distributions are shown in Figure 3.2. The Suncor nonsegregating tailings tests consisted of 14 slurry consolidometer tests on samples (initial solids contents of 40 to 65% and fines contents of 14 to 38%) in which sulfuric acid along with quick lime or fly ash were added in different amounts (Tables 1). The Syncrude nonsegregating tailings tests consisted of five slurry consolidometer tests on quick lime added samples (initial solids contents of 52 to 56% and fines contents of 12 to 26%)(Table 1).

3.7 Evaluation of Factors Affecting the Hydraulic Conductivity of Fine Tails

3.7.1 Flow Velocity Variation with Time for Fine Tails

Typical measured flow velocities are shown in Figure 3.3 and Figure 3.4. The measured flow velocities were not constant, but decreased with time and then reached a steady state, the time dependent velocity phenomenon existing even at low void ratios (Figure 3.4). However, the drop in velocity from initial to steady state becomes less as the void ratio decreases. It would be expected that the drop in velocity would be less at low void ratios because little change in the specimen can occur during the permeability test. Several tests were run at certain gradients to check whether this transient phenomenon was repeatable, with Figure 3.5 showing the results of one such test, which shows its repeatability. It is interesting to note that even after hours of flow, the initial condition seems to be re-attainable after only a few minutes of no flow. The tests shown in Figure 3.5 were not extended to the steady state condition, but to a time sufficient to check for repeatability. The time between the tests varied from 5 to 10 minutes.

Olsen et al. (1985) also noted similar flow velocity changes in slurries and suggested that this initial transient response can be due to: i) undissolved air in the equipment and/or specimen; ii) compliance in the equipment; iii) the inertia that must be overcome in changing the velocity of the pore fluid from zero to its steady state value; and iv) time dependent changes in the volume or distribution of pore space in a specimen.

If there is undissolved air in the equipment or specimen, the flow velocity will: i) increase with time due to air going into solution and not blocking the pore throats; or ii) decrease with time because of the fluid filling the voids left by the air going into solution. However in this study, the measured inflow and outflow with time are similar (Figures 3.6 and 3.7), which indicate that no undissolved air could be present in the sample. The repeatability of the transient behavior also suggests that undissolved air was not present as it would have to reappear in the same voids after 5 min to 10 min of no flow. Therefore the transient behavior cannot be attributed to undissolved air in the specimen or in the equipment .

The effect of equipment compliance on the response time depends on both the rigidity of the equipment and the magnitude of the externally applied gradient. The slurry consolidometer was made of stainless steel with a 7.45 mm wall thickness and was calibrated with water pressure which showed there was no compliance effect on the measurements. The external gradient applied was generally less than 0.2 and at low void ratios it was less than 0.6, and applied heads were in the range of 2 to 5 cm across the sample. Such small pressures will have negligible volume change effects on the tubings connecting the burettes and consolidometer. Figures 3.6 and 3.7 (similar inflow and outflow with time) suggest that there is no volume change in the equipment. Therefore, the compliance of the equipment could not be accountable for the transient behavior.

Inertia effect can create a time lag between the inflow and outflow measurements because it must be overcome in changing the velocity of the pore fluid from zero to the steady state value. Figures 3.6 and 3.7 show that there was no time lag between the inflow and outflow. Therefore inertia effect could not be responsible for the transient behavior.

Therefore, it appears that time dependent changes in the volume of the specimen or in the distribution of pore space in the specimen must account for the transient behavior. The top cap was locked in place and the LVDT did not record any movement of the top cap which indicates that there was no volume change in the slurry consolidometer. The measured inflow and outflow (Figures 3.6 and 3.7) also show this. Volume change in the sample was prevented because the permeability testing imposed no additional effective stress in the specimen. Therefore, the transient behavior cannot be attributed to time dependent volume change of the specimen. It is concluded that time dependent changes in the distribution of pore space must account for the transient behavior as described below.

The repeatability of this behavior suggests that whatever is causing the decrease in flow velocity is triggered by seepage force and is reversible. In high void ratio slurries, particles can move under the seepage stress and rearrange the distribution of pore space. The ratio of initial flow velocity to steady state velocity for the fine tails is about 300 to 400 at void ratios around 6 and is about 20 at a void ratio of about 1.5 (Figure 3.8). The variation in this ratio is compatible with the possibility of movement of fine particles into pore throats between coarser particles, which would occur more readily at high void ratios. For nonsegregating tailings, the ratio of initial flow velocity to steady state flow velocity is much less than that of the fine tails (Figure 3.9). Since the fines in nonsegregating tailings are in a flocculated or aggregated state, movement is less than that in fine tails.

The bitumen in the fine tails might also account for the transient behavior. Although considered as a solid in calculations, the bitumen is not totally rigid and can deform under stress. This deformable quality of the bitumen could allow it to move to block the pore throats while being subject to a seepage stress. At low hydraulic gradients, the effect of bitumen content on the measured permeability is negligible, as explained later under the effect of bitumen content. This suggests that the transient behavior cannot completely be attributed to the bitumen.

The time to reach the steady state for fine tails varies between 30 min to 15 hr. The time increases with an increase in high void ratio or a decrease in hydraulic gradient. The repeatability of this transient behavior suggests that it only takes about 5 to 10 min to return to the original state. The permeability tests were performed with an upward flow. The upward seepage stress is much less than the downward gravity force of the particles and bitumen (1/10 of the mass of the particles and 1/4 of the mass of the bitumen) so that it takes a long time for them to move under seepage stress and little time under gravity to return to their original position. Therefore, the transient behavior must be due to time dependent movement of fine particles and, to a lesser extent, bitumen.

Since there is continuous upward flow in the tailings ponds, due to consolidation, fine particle and bitumen blocking of pore throats would be occurring and the tailings pond field permeability will be similar to laboratory steady state permeability. Therefore, the steady state flow velocity was used to determine the permeability. Olsen et al. (1985) reports also, that for slurries, the steady state flow obeys Darcy's law with respect to a linear gradient - velocity relationship. The time to reach the steady state will vary with material and void ratio. For fine tails, it varied up to 15 hours and less for fine tails-sand mixes and nonsegregating tailings. Therefore, to obtain representative results, the flow velocity should be measured with time to determine the steady state velocity to use in permeability calculations.

3.7.2 Effect of Hydraulic Gradient

The effect of hydraulic gradient was studied by performing permeability tests at different hydraulic gradients at each void ratio after each load increment (Figure 3.10). As the gradient increases, the permeability of fine tails decreases at any given void ratio. However, when tested under hydraulic gradients less than 0.2, all permeability tests gave similar values for hydraulic conductivity. The effect of hydraulic gradient is small for void ratios less than 1. It can be concluded that the permeability of the fine tails depends on the hydraulic gradient, as opposed to Darcy's law.

The influence of hydraulic gradient may be due to: i) deformation of bitumen into pore throats within the soil skeleton in response to the flow of water; or ii) fines migration under the applied gradient into pore throats. It was noted that the permeability at low gradients was the same before and after a higher gradient was applied on the sample. This indicates a recoverable mechanism in flow through fine tails. The bitumen can deform when the hydraulic gradient is applied and when the gradient is removed it can relax back to the original position. If fines collect in the pore throats, the recoverable mechanism will not occur.

The deformation of bitumen should be directly related to the applied hydraulic gradient. The higher the gradient, the larger the deformation of the bitumen and the smaller the permeability. At small hydraulic gradients ($i < 0.2$), the deformation of bitumen should be small and the effect of hydraulic gradient on the permeability measurements should be negligible. At small void ratios ($e < 1$), the effect of hydraulic gradient is minimal because the bitumen does not have much space to deform. Hydraulic conductivity of fine tails is about 2 to 3 times lower when the hydraulic gradient is increased from 0.2 to 1. Due to the influence of hydraulic gradient on permeability, it is necessary to perform permeability tests at the field hydraulic gradient (measured or expected) in order to obtain reliable permeability values for field predictions.

3.7.3 Effect of Bitumen Content

Slurry consolidometer tests were performed on bitumen removed fine tails in order to evaluate the effect of the bitumen on the consolidation properties. Bitumen was removed by treating the sample with hydrogen peroxide. Figure 3.11 shows that the permeability of the bitumen removed fine tails is similar to the permeability of fine tails at a low hydraulic gradient. This reinforces the argument that the bitumen is causing the difference in hydraulic conductivity with different gradients.

The permeability of the bitumen removed fine tails is an upper bound for the permeability of the fine tails and is independent of the hydraulic gradient. Figure 3.11 suggests that the permeability of fine tails decreases with an increase in bitumen content. When the hydraulic gradient is less than 0.2, bitumen has little influence on the hydraulic conductivity. When the void ratio approaches 1, the effect of bitumen becomes less. The bitumen removed fine tails showed the highest permeability values at any given void ratio which suggests that removing the bitumen from the fine tails will increase the hydraulic conductivity and, hence, increase the rate of consolidation.

3.7.4 Effect of Initial Solids Content and Clay Content

The permeability of fine tails is shown to be dependent on the bitumen content and hydraulic gradient. Figure 3.12 shows the permeability - void ratio relationship of different fine tails samples with different initial solids contents from this study, Bromwell Engineering Inc. (1983), and Pollock (1988). All tests used a hydraulic gradient of approximately 0.2 to model the hydraulic gradient in the tailings pond which is 0.2 or less. The initial solids content did not affect the permeability and although the clay content of the fine tails varied between 47% and 55% it also did not have an effect. The permeability decreased by about four orders of magnitude when the void ratio decreased from 8 to 1. Several authors have used an e - log k relationship to describes the k variation for natural clays. However, for fine tails, the following power law describes the relationship.

$$k = 6.16 \times 10^{-9} \bullet e^{4.468} \tag{3.3}$$

where k is hydraulic conductivity in cm/s and e is the void ratio. This relationship can be used in the prediction or analysis of the consolidation behavior of fine tails.

3.7.5 Hydraulic Conductivity of Fine Tails - Sand Mixes

The effect of sand content on the fine tails was investigated by using three different mixes (Pollock [25]) with different solids contents and fine contents (Table 1) (Figure 3.13). Permeability trends appear from one mix to the next with respect to the sand - fines content (Figure 3.13). The permeability varies by several orders of magnitude at a given void ratio decreasing with increasing fines content. This suggests that the concentration of fines, not the sand, governs the permeability.

Figure 3.14 shows the variation of permeability in terms of fines void ratio. The fines void ratio (e_f) is defined as

$$e_f = (e / f)\ (G_f / G_s) \tag{3.4}$$

where, G_s is specific gravity of solids, G_f is specific gravity of fines, f is fines content, and e is void ratio. As all the data fall in the same range, this confirms the dependence of the permeability on the fines content. The influence of the sand on the permeability appears to be only as a filler, which decreases the fines concentration for a given volume. For this reason, permeability increases with increasing sand content in terms of total void ratio (Figure 3.13).

3.8 Hydraulic Conductivity of Nonsegregating Tailings (NST)

Hydraulic conductivity of nonsegregating tailings were analyzed to study the influence of gel structure in fine tails on permeability. Nonsegregating tailings is a mixture of fine tails and tailings sand combined with a chemical additive used to make the mix nonsegregating. The addition of different chemicals appears to produce different structures in NST. The additives used in this study were sulfuric acid, quick lime, fly ash and gypsum alone or in combination. Suncor nonsegregating tailings were formed using acid with quick lime or fly ash or gypsum alone, while Syncrude nonsegregating tailings were formed using quick lime. Suncor acid - lime/fly ash nonsegregating tailings show similar hydraulic conductivity values as the fine tails in terms of fines void ratio (Figure 3.15). Even with the addition of coagulants, the permeability is controlled by the fines, hence, by the fines void ratio. For Suncor gypsum nonsegregating tailings the permeability is only about one-third of the permeability of the fine tails (Figure 3.16). For Syncrude lime nonsegregating tailings, the permeability is only about one-fifth of that of fine tails (Figure 3.17).

The differences in permeability for the various NST mixes indicates that the different chemical additives result in different gel structures and that the gel structure controls the hydraulic conductivity of the fines. This finding is important in the analyses of the permeability of fine tails. Modeling has shown that the permeability of the fine tails in the laboratory is different than the permeability of the fine tails in the field tailings pond. This difference in permeability would appear to be caused by a difference in the fine tails gel structure in the laboratory and field.

The permeability's dependence on the additives can be explained with help of coagulation chemistry as applied to water and waste water treatment. In fine tails the solids are much more concentrated than those in water or waste water. However, the chemical principles involved may not differ. The predominant mechanisms of aggregation or coagulation are: i) charge neutralization where the soluble hydrolysis species interact with the fine particles and, ii) sweep coagulation where neutral species dominate and precipitate as solids to enhance the aggregation. Sweep coagulation is favored in high pH values when alum is used as coagulant (Amirtharajah and O'Melia 1990). For fine tails which contain many chemical constituents the chemistry of coagulation is very complex. However, the addition of acid in conjunction with quick lime lowers the pH and may reduce sweep coagulation in favor of the charge neutralization mechanism. With the absence of acid in Syncrude lime nonsegregating tailings, sweep coagulation may be predominant. The sweep-flocs formed will lead to a reduced number of flow channels, resulting in lower permeability. Addition of gypsum does not introduce H^+ or OH^- ions into nonsegregating tailings to alter the pH, however, with the OH^- ions, Ca^{2+} ions can associate to slightly lower the pH. This would lead to a coagulation state which lies between the lime added nonsegregating tailings and the acid added nonsegregating tailings. In contrast, the addition of acid in Suncor acid- lime/fly ash nonsegregating tailings may maintain a balance between the two coagulation mechanisms to keep the permeability similar in terms of fines void ratio. However, further experiments are required to validate this principle.

3.9 Conclusions

In general, the void ratio - permeability relationship of oil sands fine tails is influenced by hydraulic gradient and bitumen content. A transient state exists in the flow through the fine tails in the laboratory during permeability testing which requires some time to decrease to a steady state. This drop in flow velocity during the transient state decreases with decreasing void ratio. This phenomena can be attributed to the reorientation of fine particles due to the seepage force, which is found to be reversible. The presence of bitumen also can cause such transient behavior but to lesser degree. For appropriate measurement of permeability in the laboratory, the flow velocity should be measured with time and the steady state velocity must be used.

Deformation of bitumen from seepage forces makes the permeability of fine tails dependent on the hydraulic gradient. The permeability of fine tails, therefore does not conform to Darcy's law. Measuring permeability at field hydraulic gradients of less than 0.2 will lead to reliable use in predictions. It was determined that the permeability of the fine tails can be expressed in a power law form in terms of void ratio.

In fine tails-sand mixes, the fines content controls the permeability. The effect of sand content results in reduced fines content and, hence, alters the permeability. The addition of chemicals may have a significant effect on the permeability. The pH of the nonsegregating tailings may play an important role in coagulation or aggregation mechanisms and, hence, have an influence on the permeability.

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Table 3.1 Properties of Fine Tails - Sand Slurries Used in Consolidation - Permeability Tests

Test No.	Chemical Additives Type	Chemical Additives Concentration g/m^3	Initial Solids Content (%)	Fines Content (%)	Initial Void Ratio	Initial Fines Void Ratio
		Syncrude Fine Tails - Sand mixes				
1	none	-	48	54	2.72	5.27
2	calicum cholorite	375	52	27	2.39	9.42
3	none	-	70	20	1.22	6.63
		Syncrude NST				
1	quick lime	1250	55	12	2.12	16.00
2	"	1500	56	12	2.04	15.40
3	"	1200	54	17	2.21	11.77
4	"	1250	52	19	2.40	11.44
5	"	1250	52	26	2.40	8.36
		Suncor NST				
1	sulfuric acid/ lime	750 / 450	56	29	2.08	6.50
2	"	1200 / 400	60	26	1.75	6.10
3	"	1150 / 350	61	20	1.58	7.15
4	"	1200 / 400	64	17	1.48	7.88
5	"	700 / 450	65	14	1.40	9.06
6	sulfuric acid/fly ash	600 / 2000	40	38	4.40	10.49
7	"	600 / 2000	40	33	4.71	12.93
8	"	600 / 2000	59	22	1.81	7.45
9	"	700 / 2000	61	20	1.62	7.34
10	"	750 / 2500	64	17	1.47	7.83
11	"	750 / 2500	65	15	1.42	8.57
12	gypsum	1500	59	18	1.82	9.16
13	"	1500	57	22	2.00	8.23
14	"	900	56	21	2.05	8.84

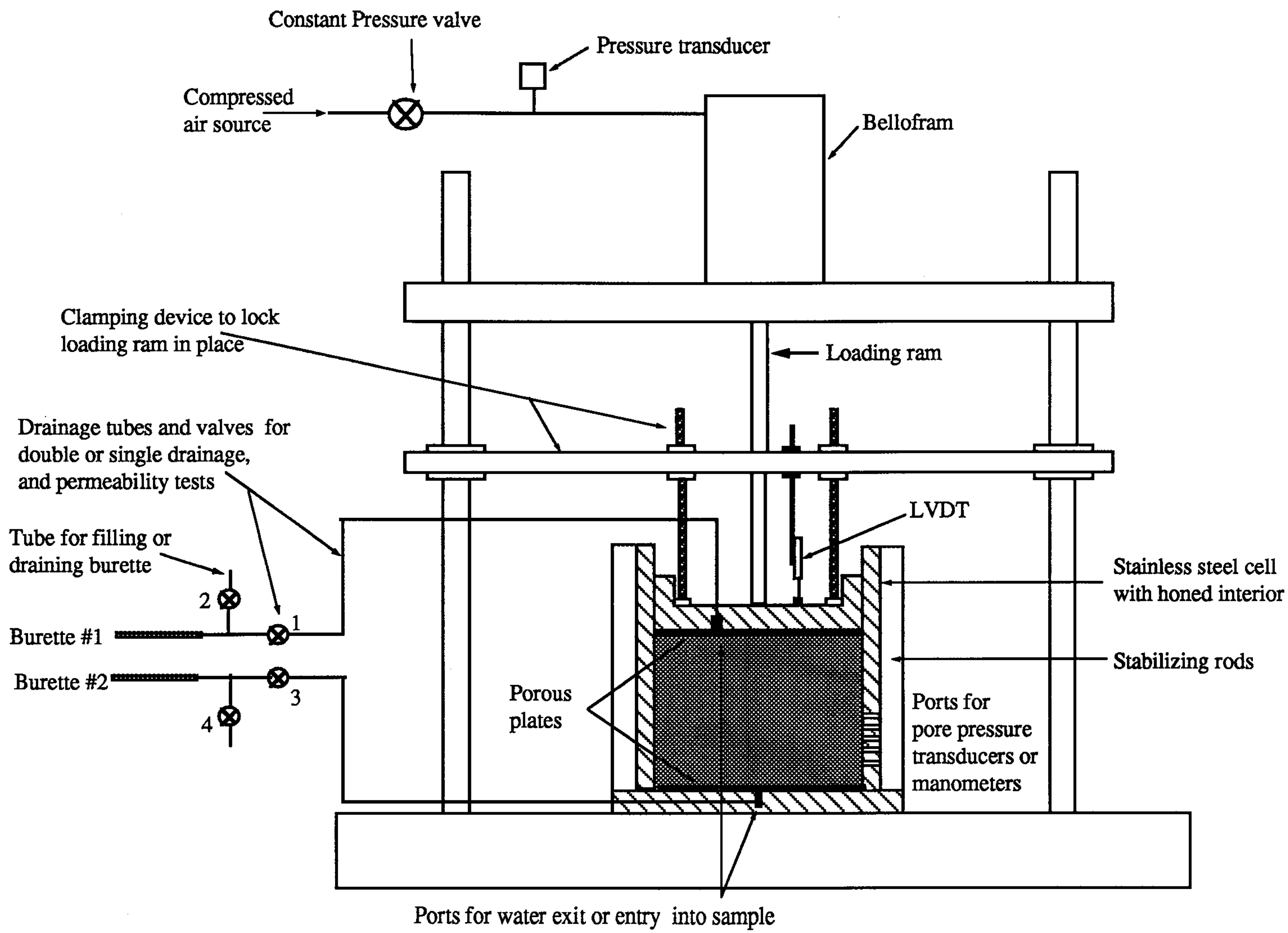

Figure 3.1 Slurry Consolidometer

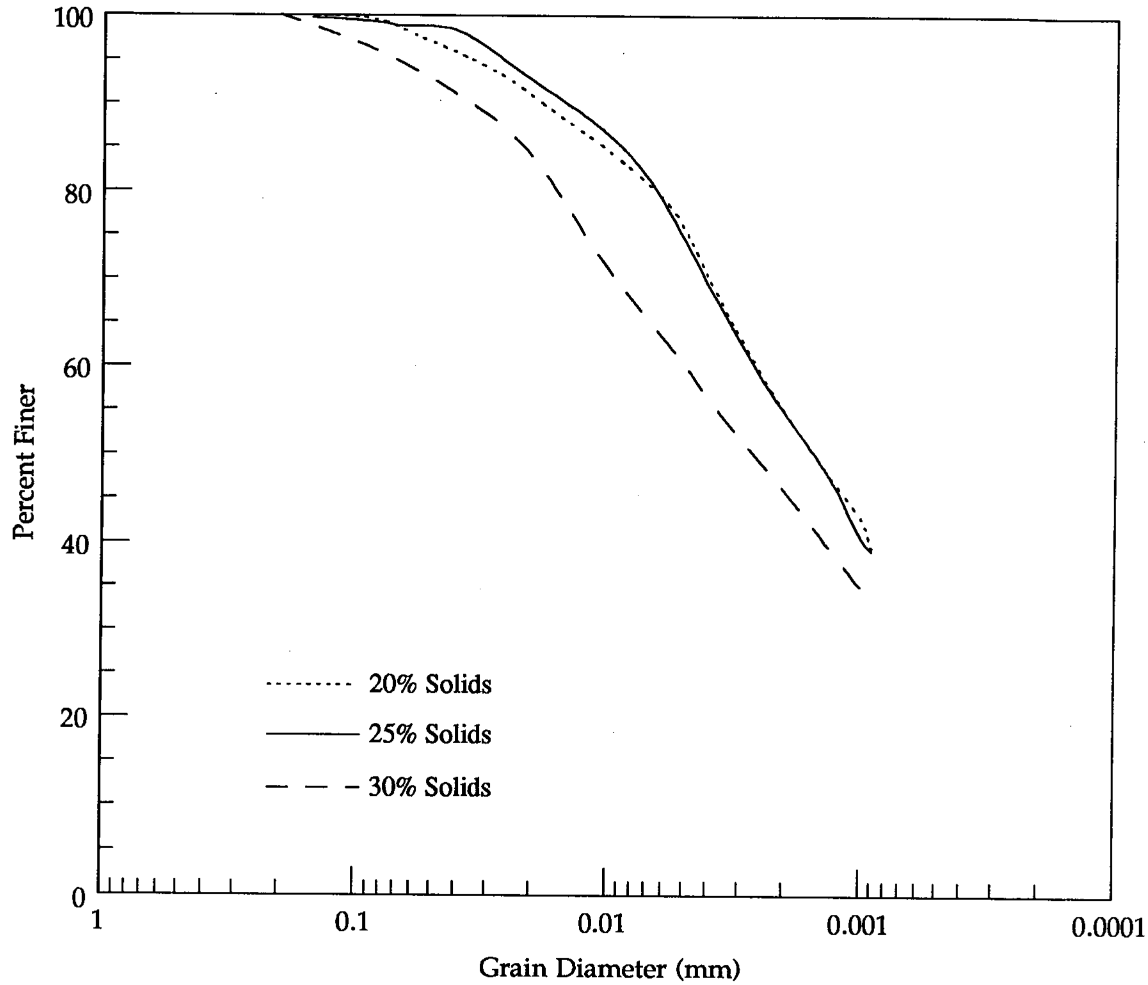

Figure 3.2 Grain Size Distribution of Fine Tails

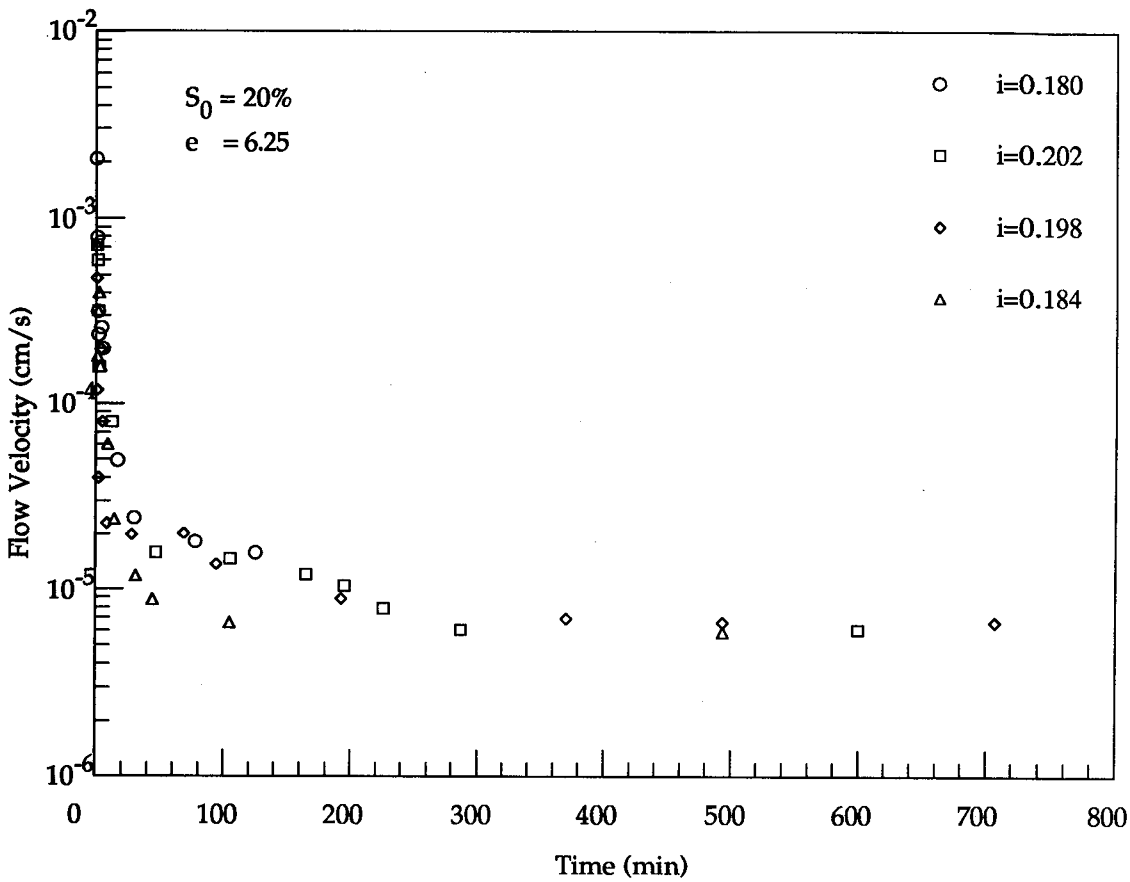

Figure 3.3 Variation of Flow Velocity with Time at a High Void Ratio

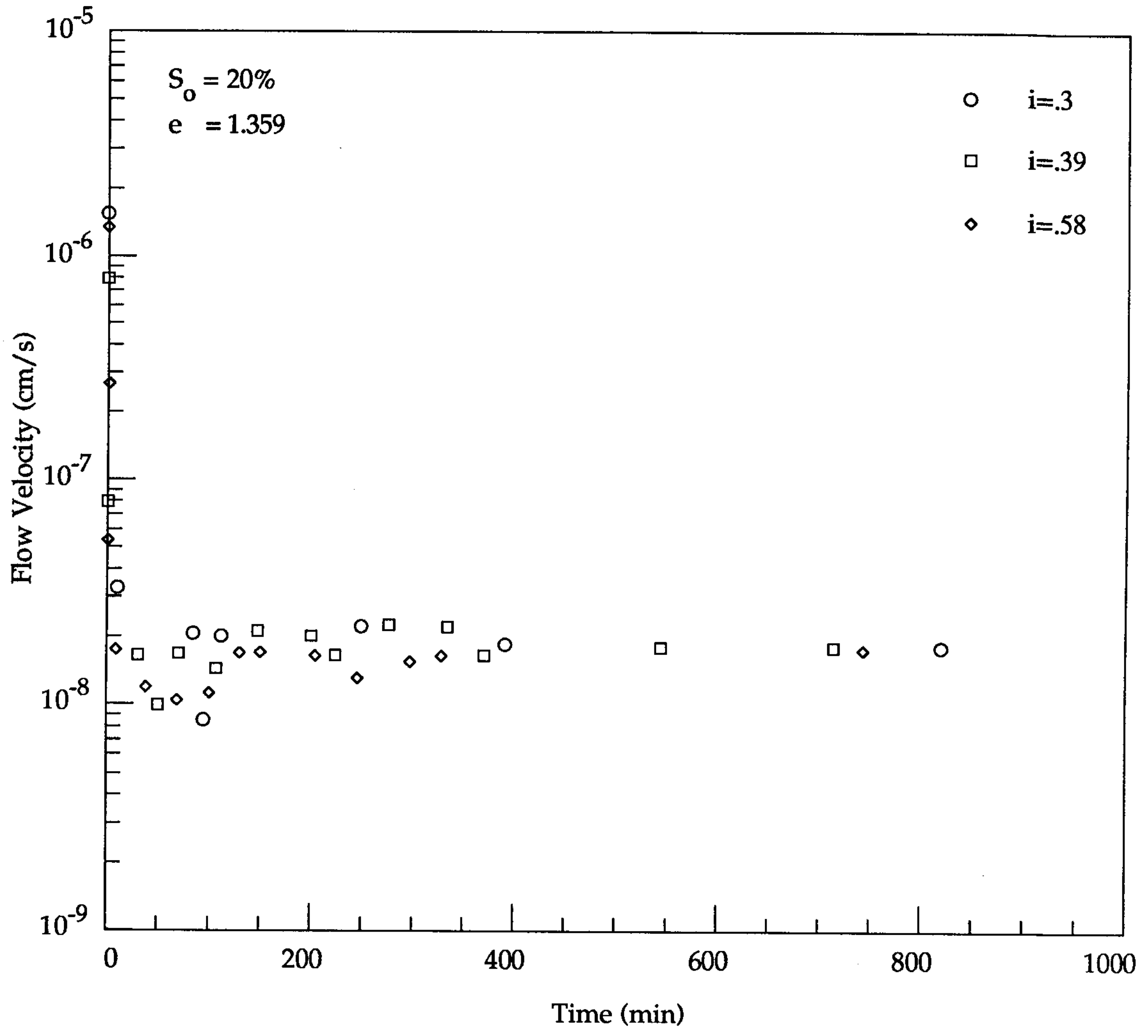

Figure 3.4 Variation of Flow Velocity with Time at a Low Void Ratio

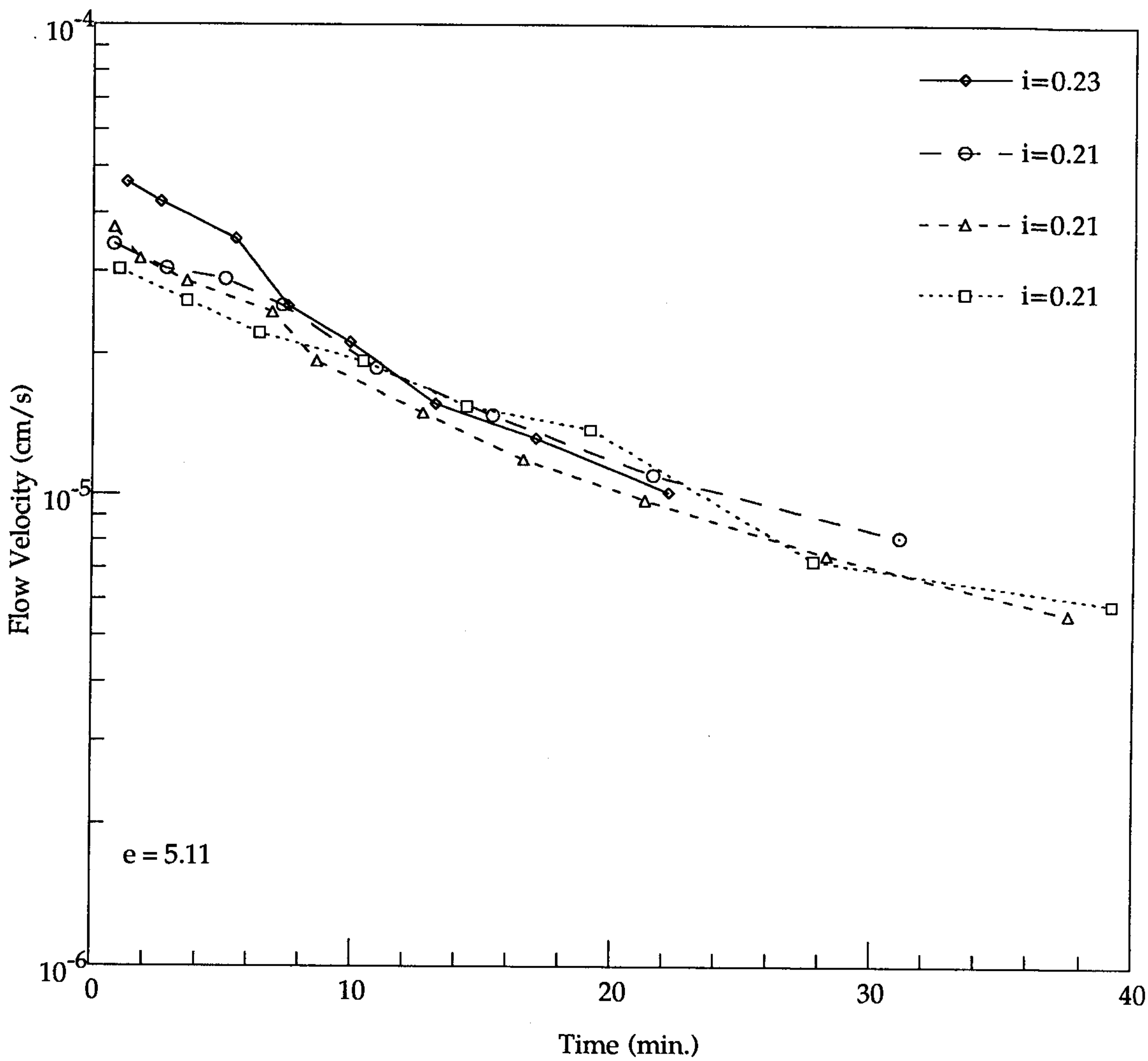

Figure 3.5 Flow Velocity with Time in Repeated Tests

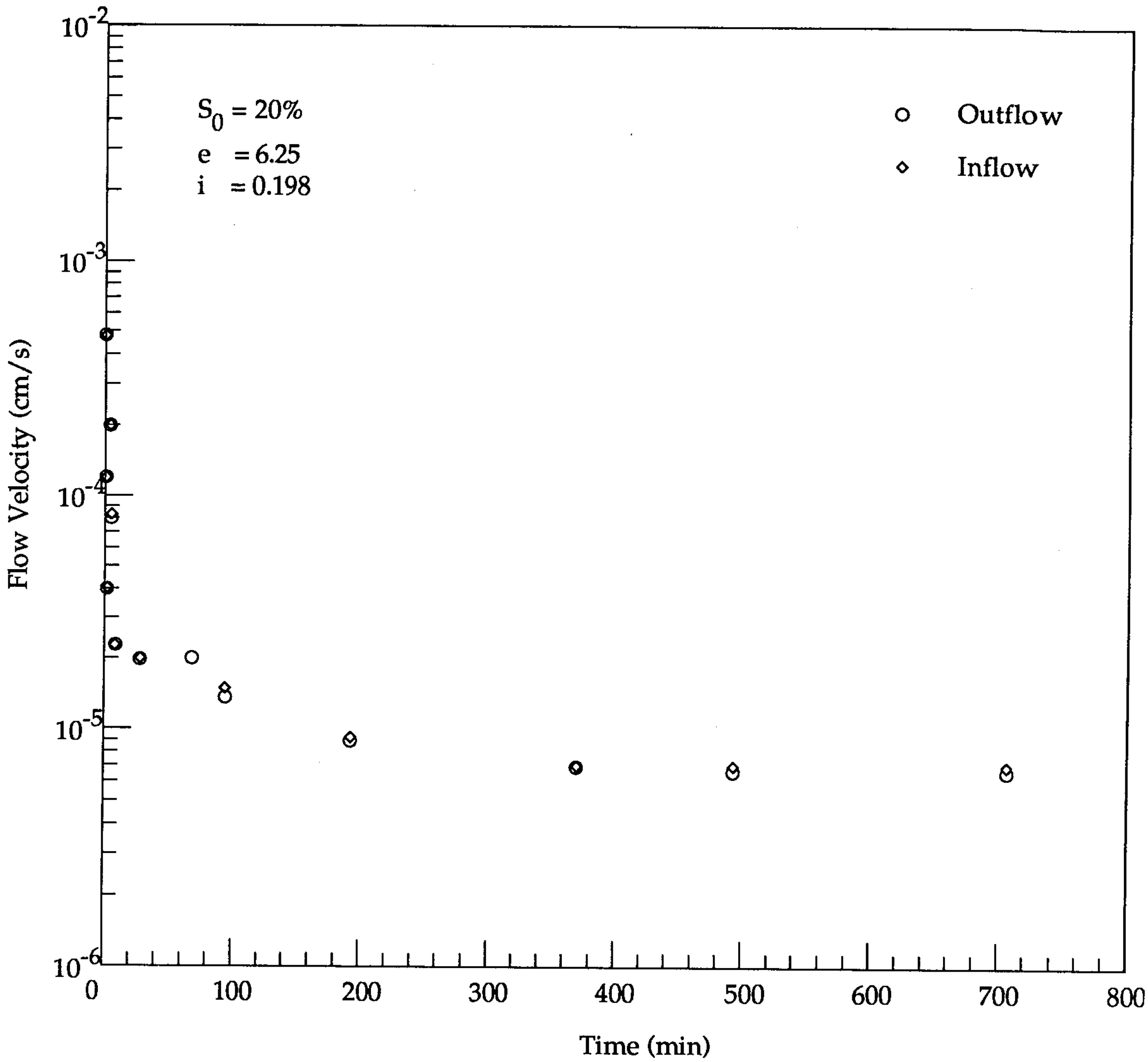

Figure 3.6 Variation of Inflow and Outflow with Time at a High Void Ratio

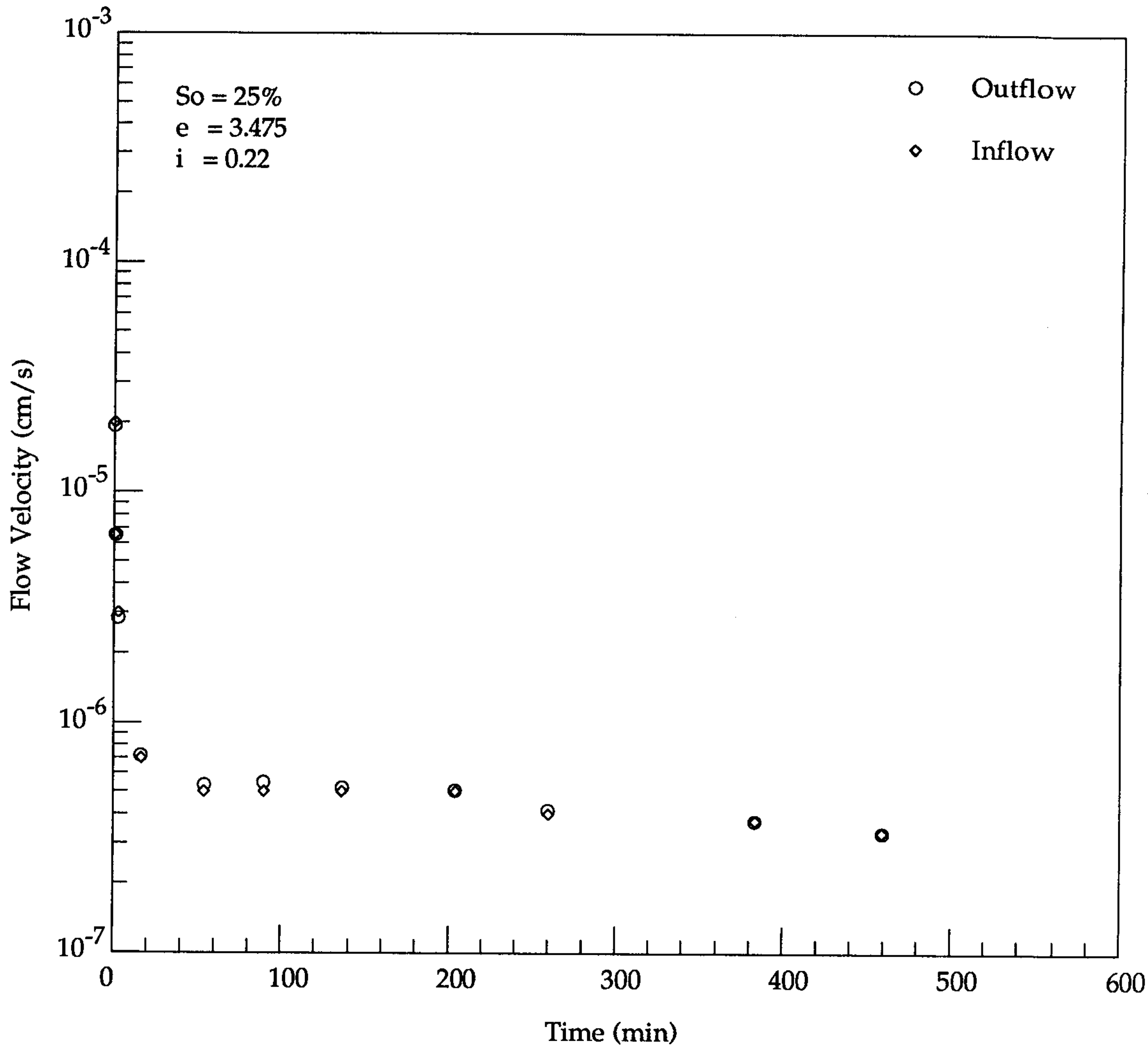

Figure 3.7 Variation of Inflow and Outflow with Time at a Low Void Ratio

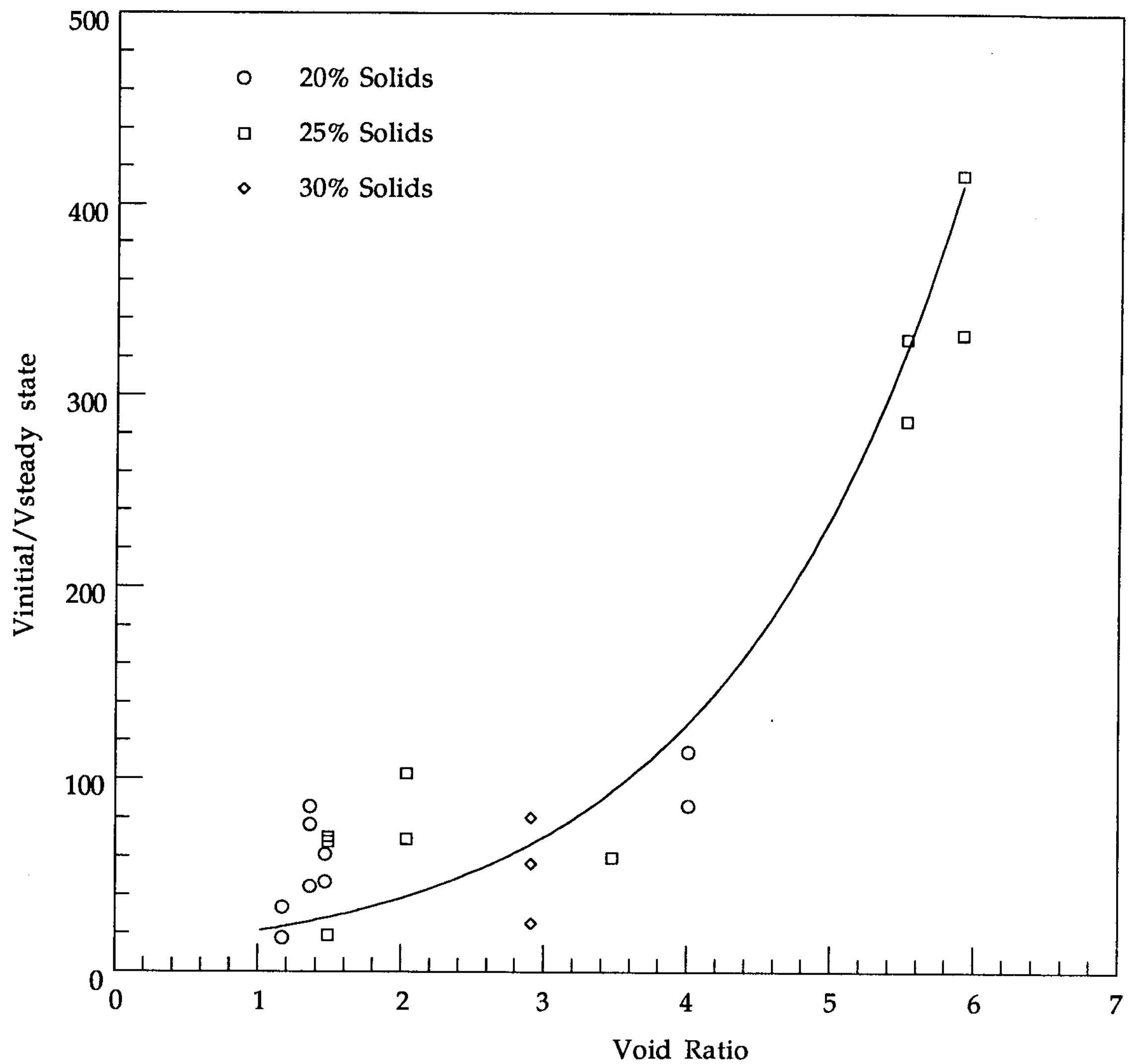

Figure 3.8 Ratio of Initial Flow Velocity to Steady State Flow Velocity for Fine Tails

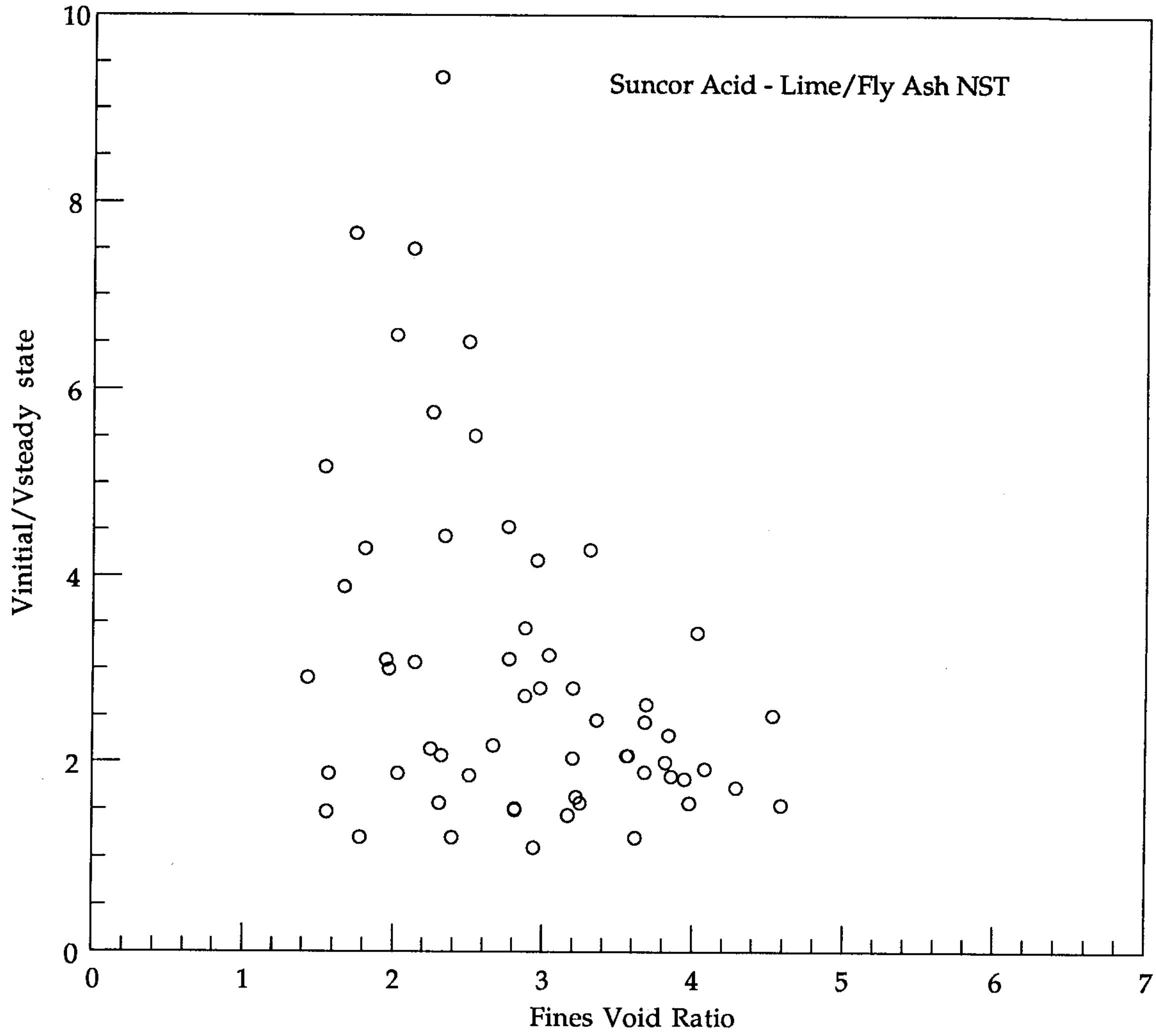

Figure 3.9 Ratio of Initial Flow Velocity to Steady State Flow Velocity for Nonsegregating Tailings

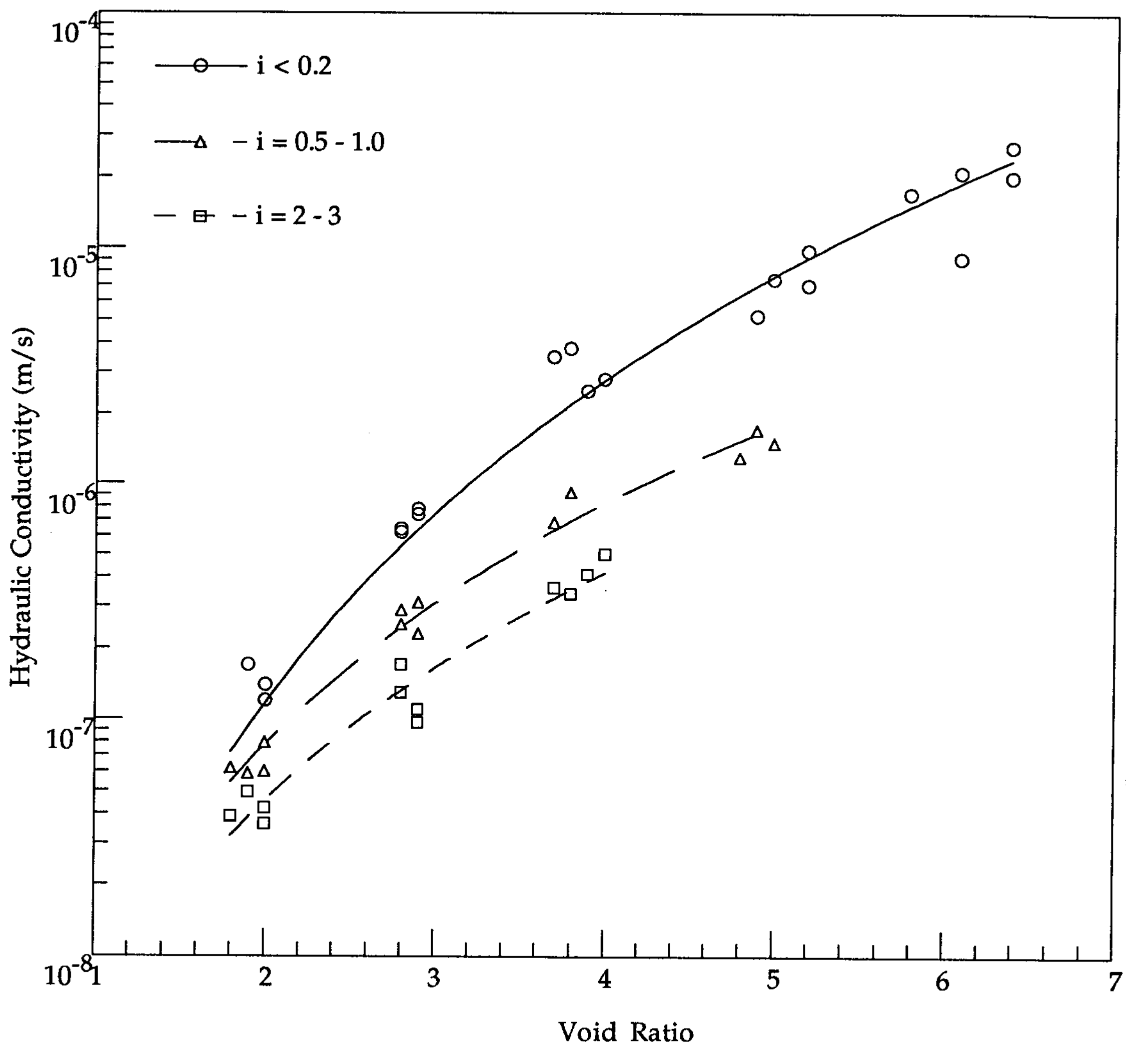

Figure 3.10 Variation of Permeability with Hydraulic Gradient
(Initial Solids Content = 20%)

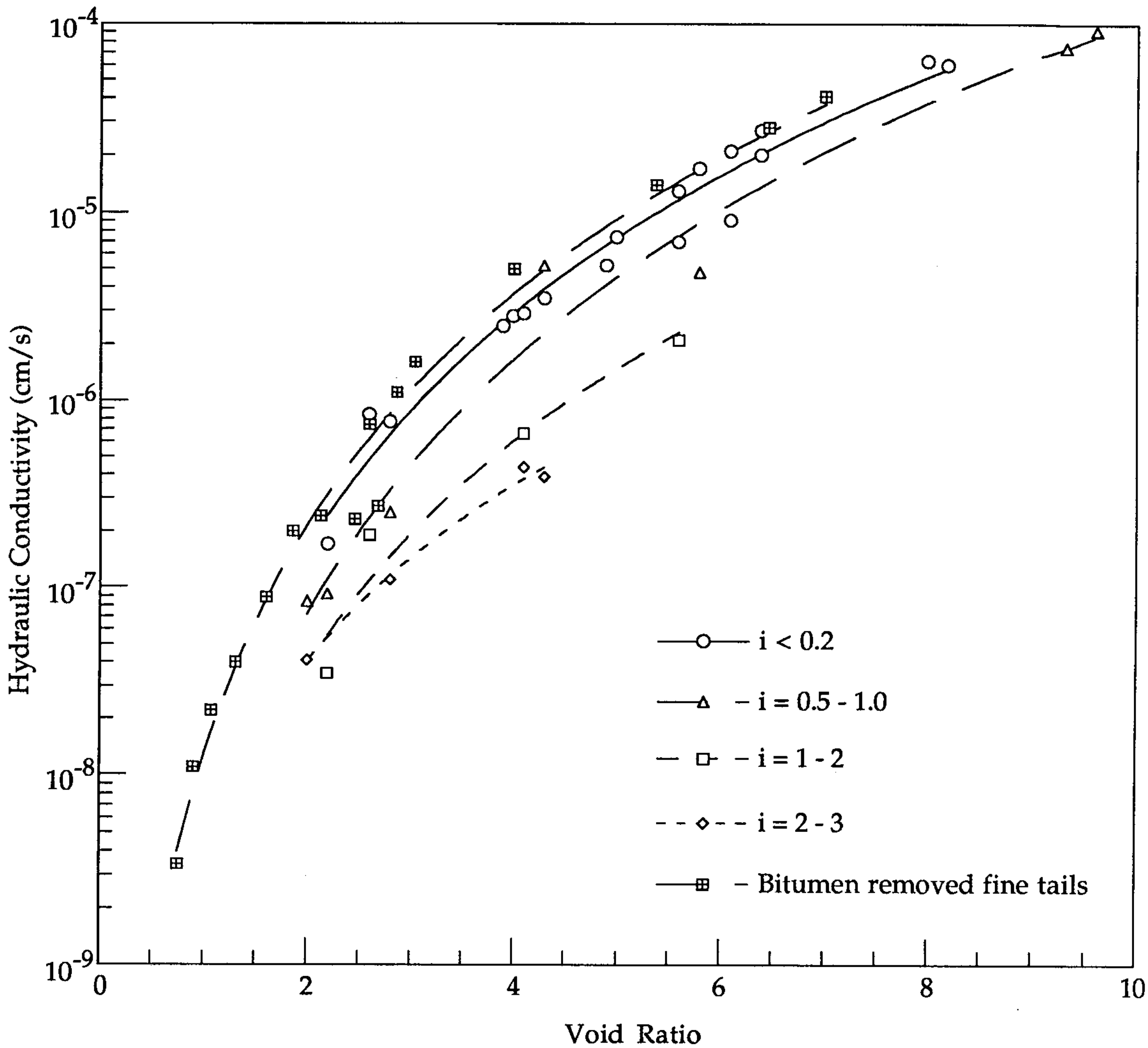

Figure 3.11 Variation of Permeability with Bitumen Content and Hydraulic Gradient
(Initial Solids Content = 10%)

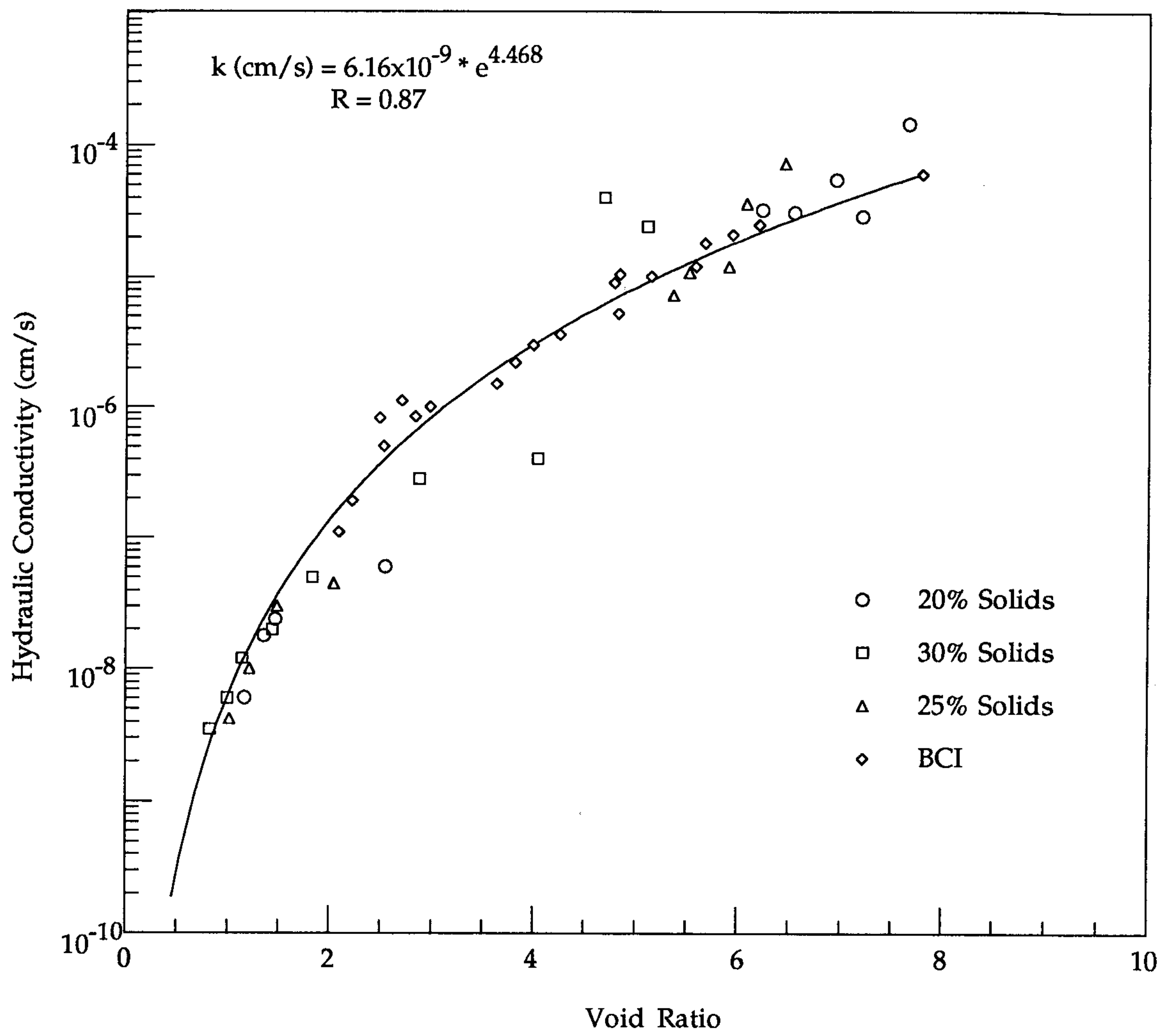

Figure 3.12 Permeability of Fine Tails

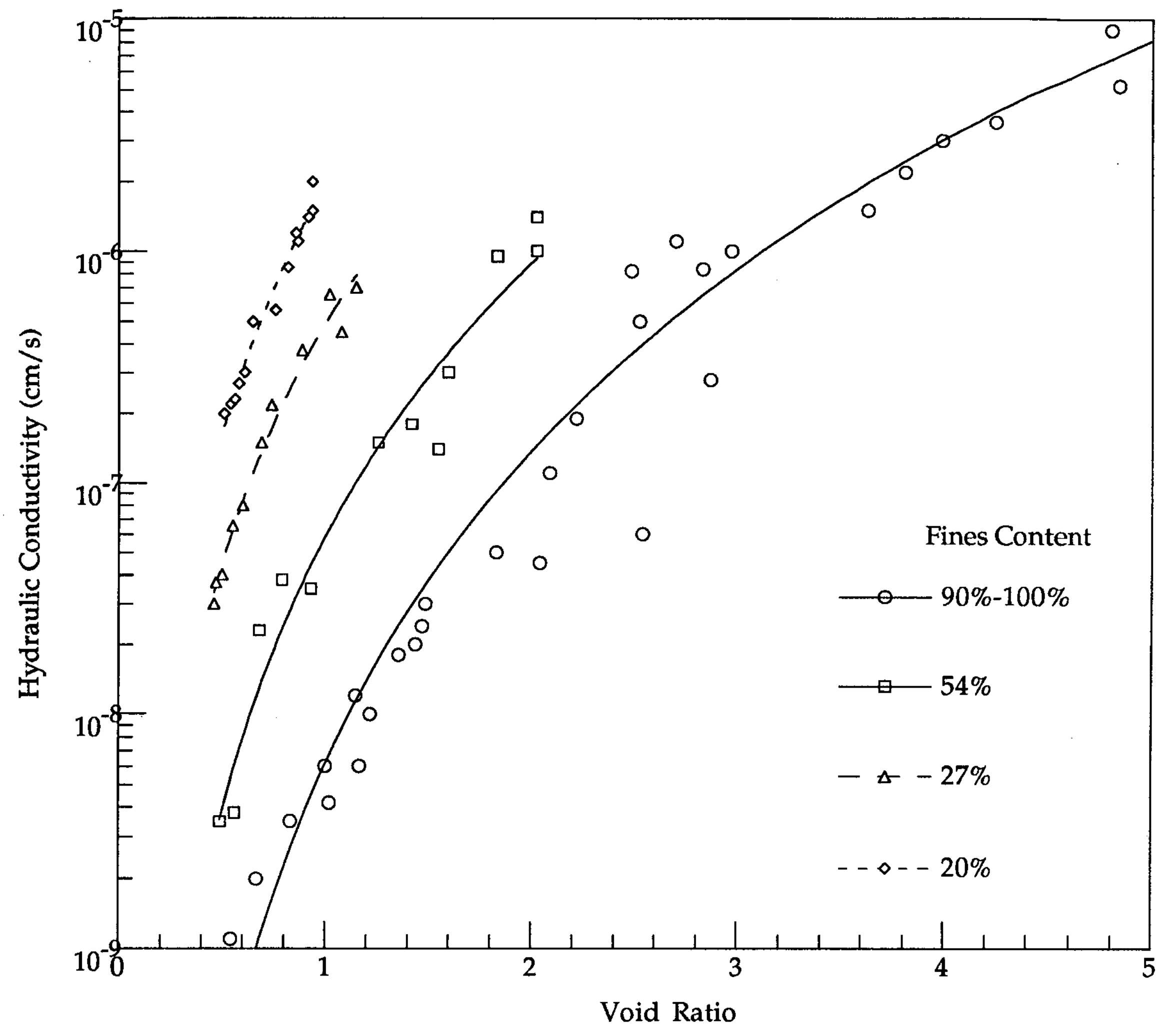

Figure 3.13 Permeability of Fine Tails - Sand Mixes

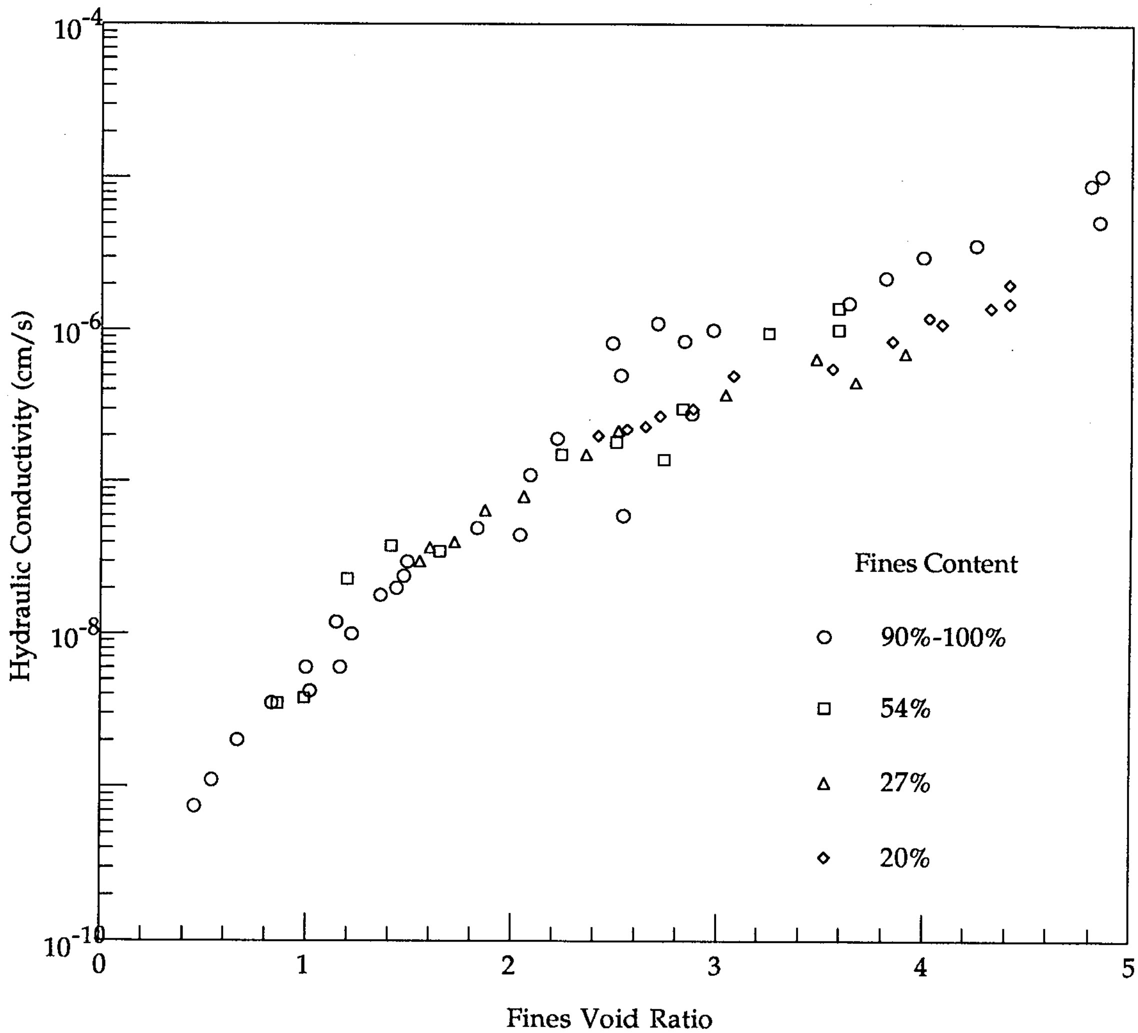

Figure 3.14 Comparison of Permeability of Fine Tails - Sand Mixes with Fine Tails

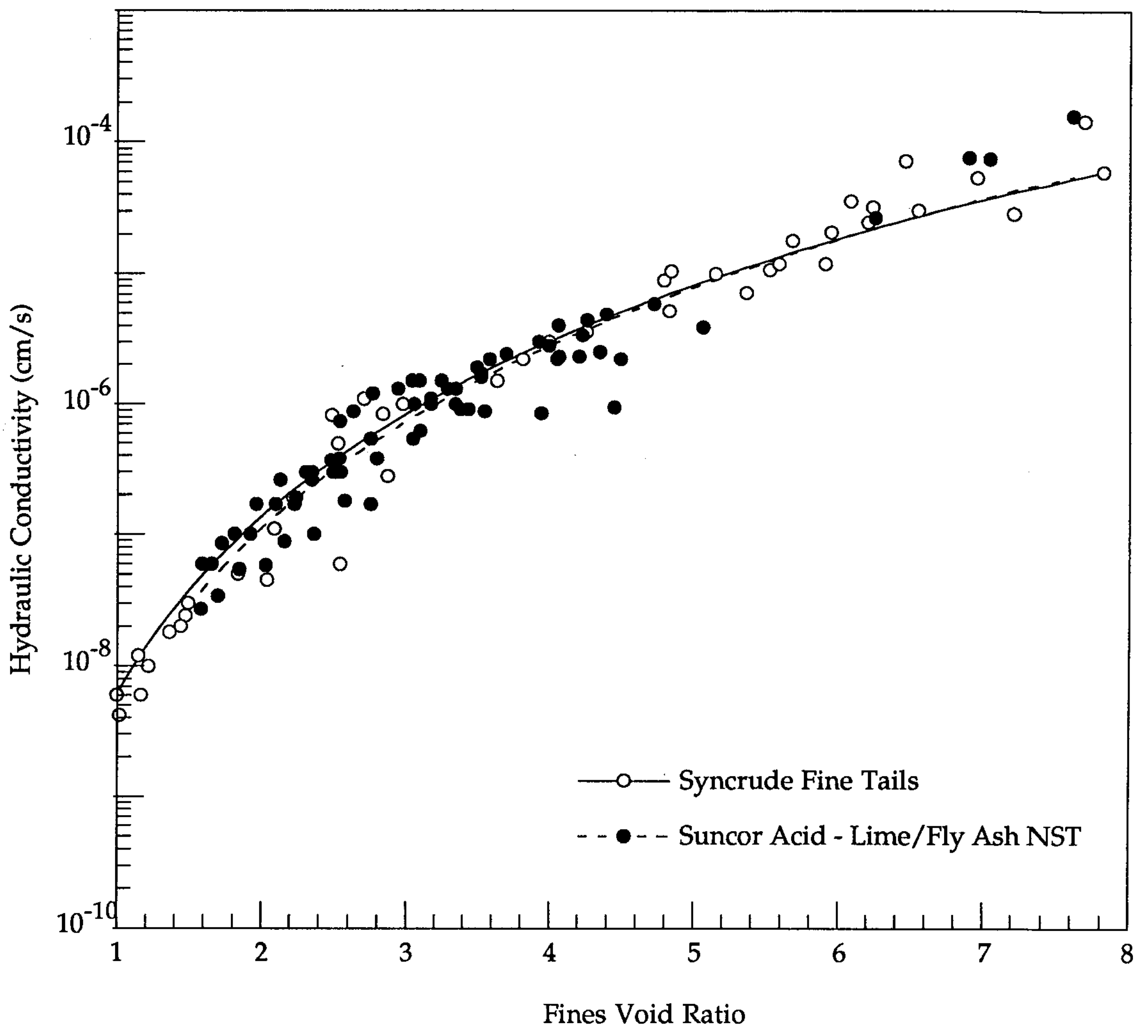

Figure 3.15 Permeability of Suncor Acid - Lime / Fly Ash Nonsegregating Tailings

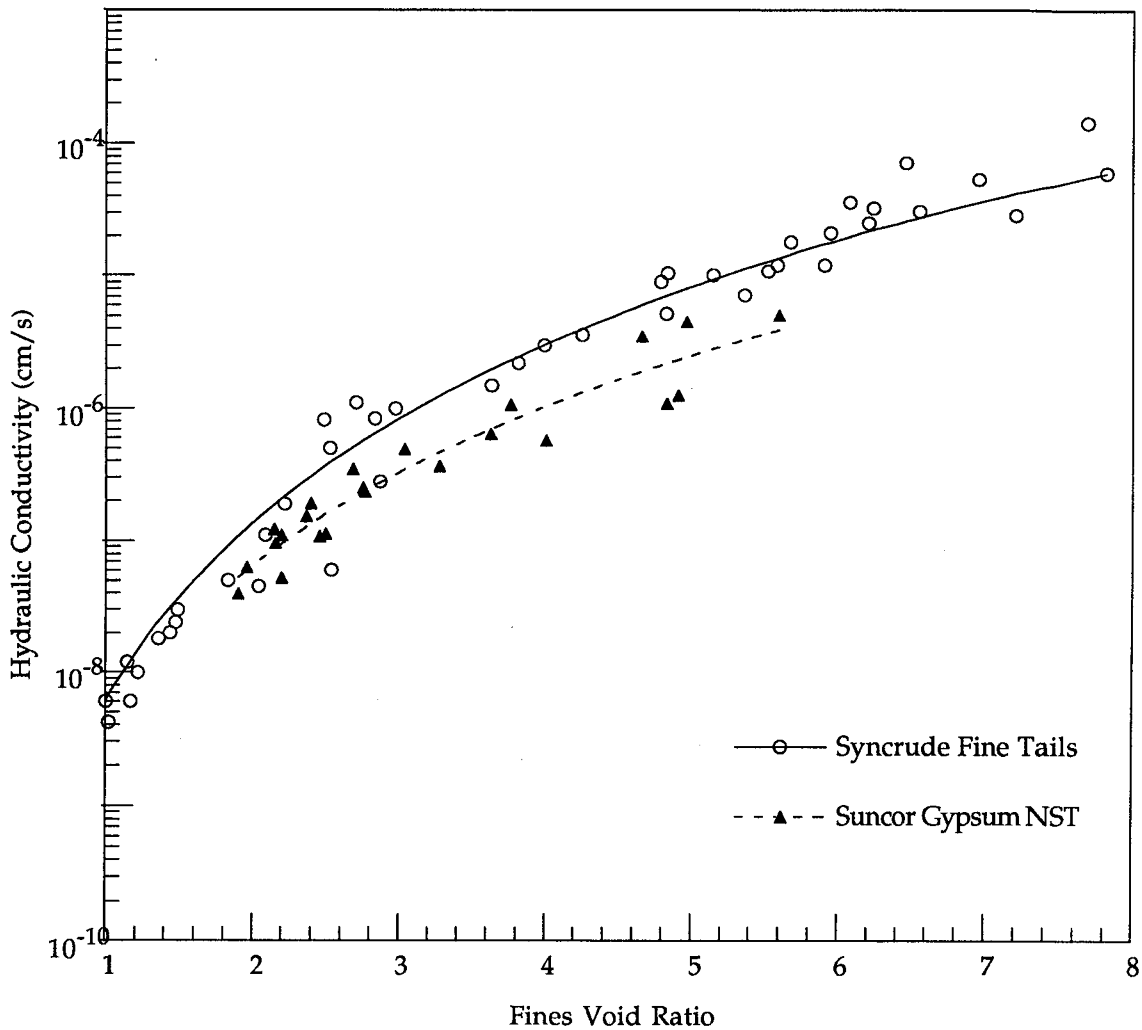

Figure 3.16 Permeability of Suncor Gypsum Nonsegregating Tailings

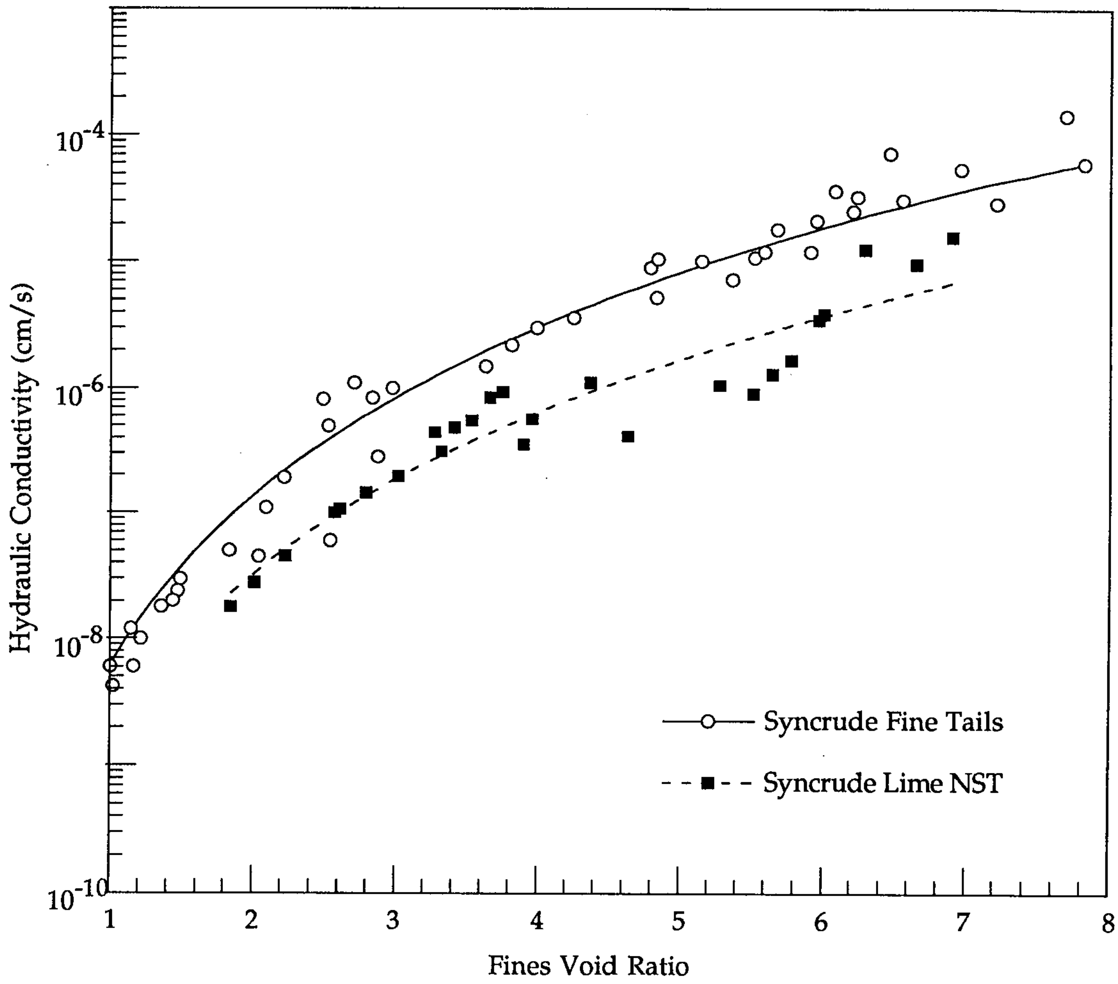

Figure 3.17 Permeability of Syncrude Lime Nonsegregating Tailings

4. CREEP BEHAVIOR OF OIL SAND FINE TAILS[1]

4.1 Introduction

In Alberta, Canada, 450,000 tonnes of oil sands are mined and processed per day in oil sand extraction plants to recover synthetic crude oil. The waste tailings stream from the extraction process is generally composed of 85% sand and 15% fines by weight. Tailings pond dykes and beaches are constructed with the sand. Approximately one-half to two-thirds of the fines and most of the water flow into these containment ponds to form fine tails deposits. The fine tails stream is a slurry containing silt and clay particles with traces of bitumen which enters a tailings pond at a low solids content. After sedimentation in a pond the fine tails reach a void ratio of 14 where consolidation begins. After approximately two years of fairly rapid consolidation, the fine tails reach a void ratio of 6 to 7 and then the consolidation slows down dramatically. At this void ratio, the fine tails is accumulating at a rate of 45,000 m^3 per day and approximately 325 million m^3 is presently held in the tailings ponds. The material, by economic necessity, must rely on self weight consolidation for its future densification and extensive research is being conducted to study its long term consolidation behavior.

Though the fine tails appears to consolidate in the tailings ponds, the measured effective stresses in the tailings ponds are very small; below 2 kPa. This suggests that much of the decrease in void ratio may be due to a creep mechanism. The main objective of the work presented in this paper is to establish the creep properties of fine tails and to compare the creep rates with a large scale, long term settlement test of fine tails. The creep rates are also compared with those of other clayey soils. As well, the creep properties of

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[1] A version of this chapter has been published. Suthaker N.N. and Scott J.D.,Proceedings of the Internaional Symposium on Compression and Consolidation of Clayey Soils, IS-Hiroshima'95, Hiroshima, Japan, May 10-12, 1995, pp. 195-200.

mixes of fine tails and sand have also been measured. The paper also describes the fine tails material, testing equipment and test results. Analysis of the long term behavior of fine tails incorporating the measured creep properties will enhance the design of eventual disposal and reclamation procedures.

4.2 Creep Mechanisms

A number of authors have considered the creep phenomenon. Taylor (1942) defined creep to be due to the gradual readjustment of the soil grains into a more stable configuration following the rupture of the soil structure that occurs during the primary consolidation stage. Wahls (1962) assumed that creep is caused by viscous yielding of the grain structure and the consequent reorientation of grains, and that this occurred so slowly that the pore pressures produced were negligible. Gray (1963) attributed creep to the inability of the mineral skeleton to reach elastic equilibrium with the applied load.

Sridharan and Rao (1982) have suggested that creep is related directly to the strength at a particle level which is governed by a modified effective stress concept. Murakami (1988) suggested that creep is due to the viscous shear resistance of adsorbed water films against the relative movement at contact points of soil particles.

Mitchell (1993) defined creep as the time dependent volumetric strains and/or shear strains that develop at a rate controlled by the viscous resistance of the soil structure. Secondary compression or secondary consolidation is the specific case of the creep that follows primary consolidation and is also controlled by the viscous resistance.

The previous works described above indicate that the creep mechanism of soil can be attributed to several physical phenomena. It appears that the form and magnitude of the interparticle forces are the basic governing factors of creep behavior.

4.3 Factors Influencing Creep Behavior

The relationship between the creep index (C_α) and time cannot be definitely established by means of laboratory tests even though the tests extend over several years. Besides the time limit, the value of some long term laboratory tests may be limited by factors producing physico-chemical changes in the soil sample (Mesri, 1973).

The relationship between creep and effective stress is not clear for several reasons. One is that the final effective stress, under which the creep index is measured, is reached by many different ways such as incremental loading with a load increment ratio of unity versus continuous loading, or single increment loading with a load increment ratio greater than unity. Sridharan and Rao (1982) showed that higher load increments cause higher shear stresses and greater creep. As the load increment increases, the creep index increases.

There is a general agreement that preconsolidation can reduce the magnitude of creep (Mesri, 1973) and that the degree of reduction is a function of the degree of overconsolidation.

Murakami (1980) concluded from the tests by Aboshi (1973) that the vertical strain during primary consolidation increases with sample thickness. This occurs because when the sample thickness is large, primary consolidation takes a long time to complete and, therefore, the creep during primary consolidation is higher due to the increased time available.

For oil sand fine tails, the significant factors which appear to affect the creep behavior are physico-chemical changes that occur with time and the considerable depth of the fine tails in the tailings ponds. As the depth of the fine tails is from 40 to 60 metres and the ponds were filled fairly rapidly, there will be a significant amount of creep during primary consolidation.

4.4 Creep During Primary Consolidation

It has been observed that one dimensional consolidation progresses more rapidly in the field than that predicted by Terzaghi's theory using laboratory measurements (Aboshi, 1973). This tendency is more remarkable as the thickness of a deposit becomes greater. The higher rate of consolidation found in the field cannot be explained by differences in hydraulic gradients or strain rates from those occurring in laboratory consolidation tests.

One of the reasons to explain this phenomena is creep which occurs during primary consolidation. When the thickness of a clay deposit or the maximum distance to a drainage boundary is small, a significant amount of creep appears only after the excess pore pressure has been dissipated. On the other hand, when the thickness of the clay deposit is large, primary consolidation takes a long time. As a result, creep is significant during the stage of excess pore water dissipation and takes place at the same time as primary consolidation.

There are numerous research papers that deal with creep characteristics which occurs after primary consolidation is completed. But very few papers are concerned with creep which arises during primary consolidation. Murakami (1988) showed the influence of the thickness of a clay layer on creep and that creep is generated during primary consolidation only when the effective stress is greater than the preconsolidation pressure. Little creep may arise during primary consolidation when clay is in an apparently overconsolidated state.

4.5 Laboratory Investigation

In order to study creep behavior of fine tails, slurry consolidometer tests have been performed on fine tails (Suthaker and Scott, 1994a). The creep index (C_α) is defined as $\Delta e / \Delta \log t$; where e is void ratio, and t is time. This is same as the definition for secondary compression index, coefficient of secondary consolidation and coefficient of secondary compression defined by various authors.

Oil sand fine tails with three different initial solids contents (20%, 25% and 30%) (initial void ratios of 9.11, 7.36, 6.46) were tested in the slurry consolidometers. Liquid and plastic limits of the fine tails are approximately 59% and 23%, respectively. The bitumen content of the fine tails is 3% by mass of the mineral solids. Fine tails-sand mixes were also tested for creep properties (Pollock, 1988). The properties of the mixes are shown in Table 4.1. The fines void ratio (e_f) is calculated by

$$e_f = (e / f) \ (G_f / G_s) \tag{4.1}$$

Where: G_s = specific gravity of solids, G_f = specific gravity of fines, f = fines content

The initial height of the specimens in the slurry consolidometers was 26 cm. The applied stress increments were equal to the previously applied stress. The consolidation tests lasted for up to two years. The sample was consolidated under a load until the primary consolidation was complete, and then sufficient time was allowed to measure the creep. The duration of each load increment was about 2 months.

4.6 Results and Discussion

Typical void ratio - log time plots obtained from this study are depicted in Figures 4.1 and 4.2 for low void ratio and high void ratio load increments respectively. Figure 4.1 shows a classical void ratio-time measurement. The creep index (C_α) was calculated using the slope of the void ratio - log time curve after primary consolidation was complete. However, there is no definite inflection point, which is required to define the end of primary consolidation in Figure 4.2. To overcome such material behavior, a rectangular hyperbolic technique (Sridharan et al, 1987) was used to determine the end of primary consolidation. This technique recognizes the relationship between the time factor and degree of consolidation as a rectangular hyperbola over the consolidation range between 60 to 90 percent. Given the end of primary consolidation, the creep index was assessed from the e - log time plot.

Figure 4.3 shows the variation of creep index (C_α) with void ratio at the end of primary consolidation (e_p). Variation in initial solids content in the range of 20% to 30% did not seem to affect the creep behavior of the fine tails. As expected, C_α was found to increase with an increase in e_p. The creep index can be estimated from $C_\alpha(\%) = 3.689\ e_p$. Previous study indicated that the primary consolidation at low stress levels was affected by initial void ratio (Suthaker and Scott, 1994a). Therefore the value of e_p is governed by several parameters, especially the stress level and the initial void ratio during primary consolidation, and hence influences C_α. At high effective stress levels the effects of such parameters become insignificant.

One of the disposal options for the oil sand tailings is to form a nonsegregating tailings which would be a mixture of fine tails and sand. Therefore, it is of interest to study the creep behavior of such fine tails-sand mixes. Figure 4.4 depicts the creep properties of three mixes. The value of C_α increased rapidly with a decrease in void ratio to a maximum value and then decreased. Similar behavior was obtained during the creep of undisturbed field samples of organic silt, where it was attributed to the overconsolidated and normally consolidated stages (Wahls, 1962). However, the fine tails-sand mixes are not overconsolidated and the mixes showed an apparent preconsolidation effect from the sand content. Figure 4.4 also suggests the existence of an optimum e_f to maximize the creep. The optimum e_f may represent the void ratio at an apparent preconsolidation stress.

Figure 4.5 shows the variation of C_α with C_c for the fine tails and for the high fines mix. This plot clearly indicates that the creep behavior of the fine tails and the high fines mix are similar. The ratio of C_α to C_c for these materials is 0.085. The behavior of the low fines mixes were different from the fine tails and have a C_α to C_c ratio of 0.019 (Figure 4.6). The above results show that for fine tails-sand mixes, there is a limiting fines content beyond which the nonsegregating tailings behaves as fine tails during creep.

Figure 4.7 compares the variation of C_α with e_p for fine tails, Chicago Blue Clay (Mesri, 1973) and organic silt (Wahls, 1962). The creep of fine tails is similar to that of organic silt. The fine tails has a very high rate of creep compared to Chicago Blue Clay. Figure 4.8 depicts the effect of C_c on C_α for various materials. The fine tails show the highest C_α to C_c ratio. The ratio C_α to C_c is a key parameter in describing creep and Table 4.2 shows values for a number of soils. The fine tails which contain organic matter in the form of a hydrocarbon show a similar ratio as other organic soils.

4.7 Settlement in a 10 m Deep Deposit of Fine Tails

A large scale self weight consolidation test in a 10 m high 1 m diameter column has been conducted from 1982 for 12 years (Suthaker and Scott, 1994b). The column was filled with 31% solids content fine tails. The column has pore pressure ports and sampling ports at 1 m intervals to allow the calculation of total stress and effective stress. The creep settlement was calculated using the laboratory measured creep properties and compared with the measured total settlement.

Figure 4.9 shows the effective stress distribution and the surface of fine tails after 10 years of settlement. Initially the measured pore pressures were equal to the total stress. After 10 years of settlement, the fine tails - water interface has settled 2.17 m. The corresponding increase in effective stress is about 0.5 kPa to 1.0 kPa throughout the top 7 m depth of fine tails. The effective stress increase in the bottom 1 m of the column appears to be due to sand settling through the fine tails, in addition to some consolidation. The effective stress increase calculated from finite strain consolidation theory is higher at the bottom and very small at the top of the deposit. This suggests that creep is increasing the rate of settlement in the top half of the column which then decreases the rate of consolidation in the bottom half.

The fine tails in the column had an induction time of one month and the starting time of creep was taken as 1 month. The final time was taken as

10 years. The calculated creep settlement over this period is 0.642 m which is about 30% of the total settlement.

4.8 Conclusions

From this study, the following conclusions can be drawn.

1. The creep rate of fine tails increases linearly with void ratio as with other soils.

2. For fine tails-sand mixes, there exists an optimum e_f where the C_α was a maximum. The optimum e_f increased with an increase in the sand content. The optimum e_f might represent the void ratio at an apparent preconsolidation stress caused by the sand content.

3. There exists a limiting fines content beyond which the creep behavior of fine tails-sand mixes is similar to that of fine tails. This occurs because at a high fines content the sand grains are not in contact and the fines control the geotechnical behavior.

4. Fine tails shows a large creep index when compared to other clayey materials and is similar to that of organic silt.

5. C_α increases linearly with C_c and the ratio of C_α to C_c for fine tails is found to be 0.085 which is high and similar to other organic soils.

6. In the 10 m deep deposit, the measured effective stresses indicated that creep is occurring in the column and the calculated settlement using the laboratory measured creep index showed that 30% of the total settlement at the end of 10 years can be attributed to creep.

7. In the tailings ponds, the primary consolidation of the fine tails will take a long time to complete because of its low permeability and large depth. The very small effective stresses which have developed over 15 to 20 years

indicate that the volume decrease of the fine tails has a creep component which is taking place during primary consolidation. At its present void ratio of about 6, the fine tails in the pond has a very high creep index of about 22%. Finite strain consolidation analysis and creep analysis of a 50 m deep pond indicates that over a 20 year period about 27% of the total settlement would be due to creep.

4.9 References

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Table 4.1 Properties of Fine Tails-Sand Mixes

Mix	Solids Content (%)	Fines Content (%)	Initial Void Ratio	Initial Fines Void Ratio
1	48	54	2.72	5.27
2	52	27	2.39	9.42
3	70	20	1.22	6.63

Table 4.2 C_α/C_c Values for Soils

Soil	C_α/C_c
Norfolk organic silt[1]	0.05
Berthville clay[2]	0.045
Whangamarino clay[3]	0.03
Calcareous organic silt[1]	0.035-0.06
Amorphous and fibrous peat[4]	0.035-0.083
Canadian muskeg[1]	0.09-0.10
Leda clay[1]	0.025-0.06
Peat[1]	0.05-0.085
Post-glacial organic clay[1]	0.05-0.07
Soft blue clay[1]	0.026
Organic clay and silts[1]	0.04-0.06
Sensitive clay, Portland[1]	0.025-0.055
San Francisco Bay mud[1]	0.04-0.06
New Liskeard varved clay[1]	0.03-0.06
Silty clay C[1]	0.032
Nearshore clays and silts[1]	0.055-0.075
Fibrous peat[1]	0.06-0.085
Mexico City clay[1]	0.03-0.035
Hudson River silt[1]	0.03-0.06
New Heaven organic clay silt[1]	0.04-0.075
Tar Sand[2]	0.035
Bearpaw shale[2]	0.029
Oil sand fine tails[5]	0.085
Fine tails-sand mixes[5]	0.019

1 - Mesri and Godlewski (1977)
2 - Mesri and Castro (1987)
3 - Newland and Allely (1960)
4 - Wahls (1960)
5 - This study

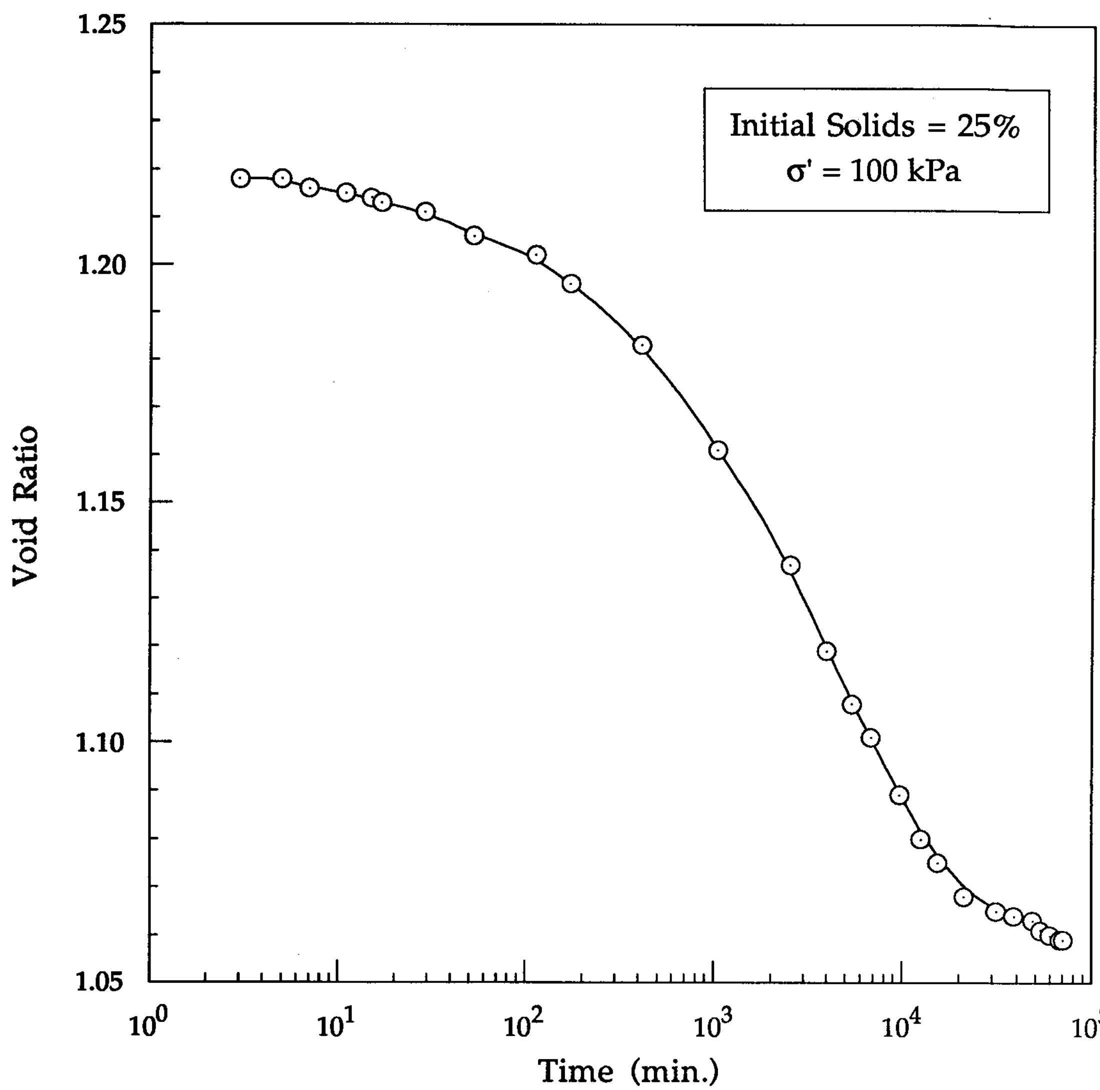


Figure 4.1 Classical Time - Compression Curve

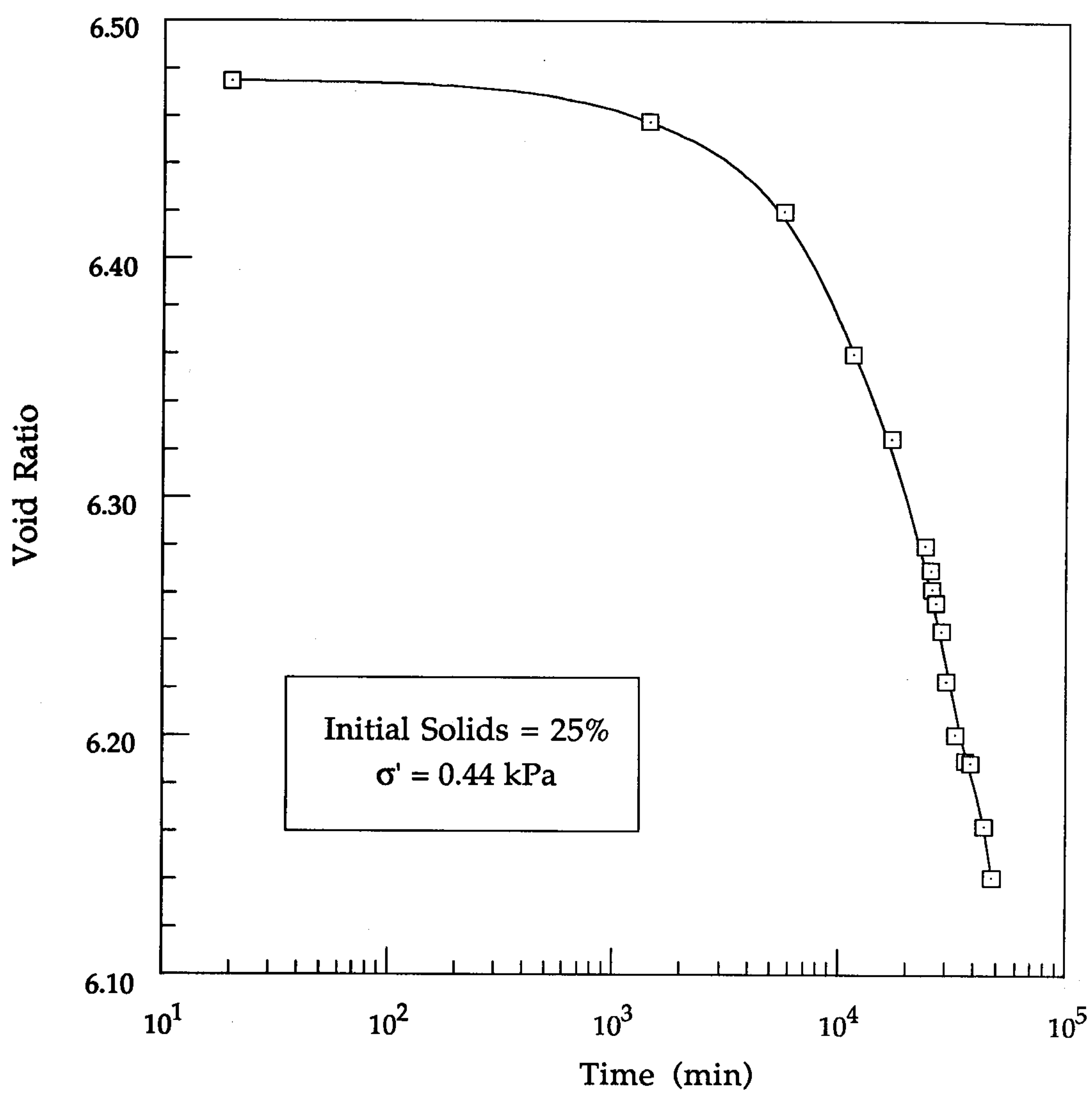


Figure 4.2 High Void Ratio Time - Compression Curve

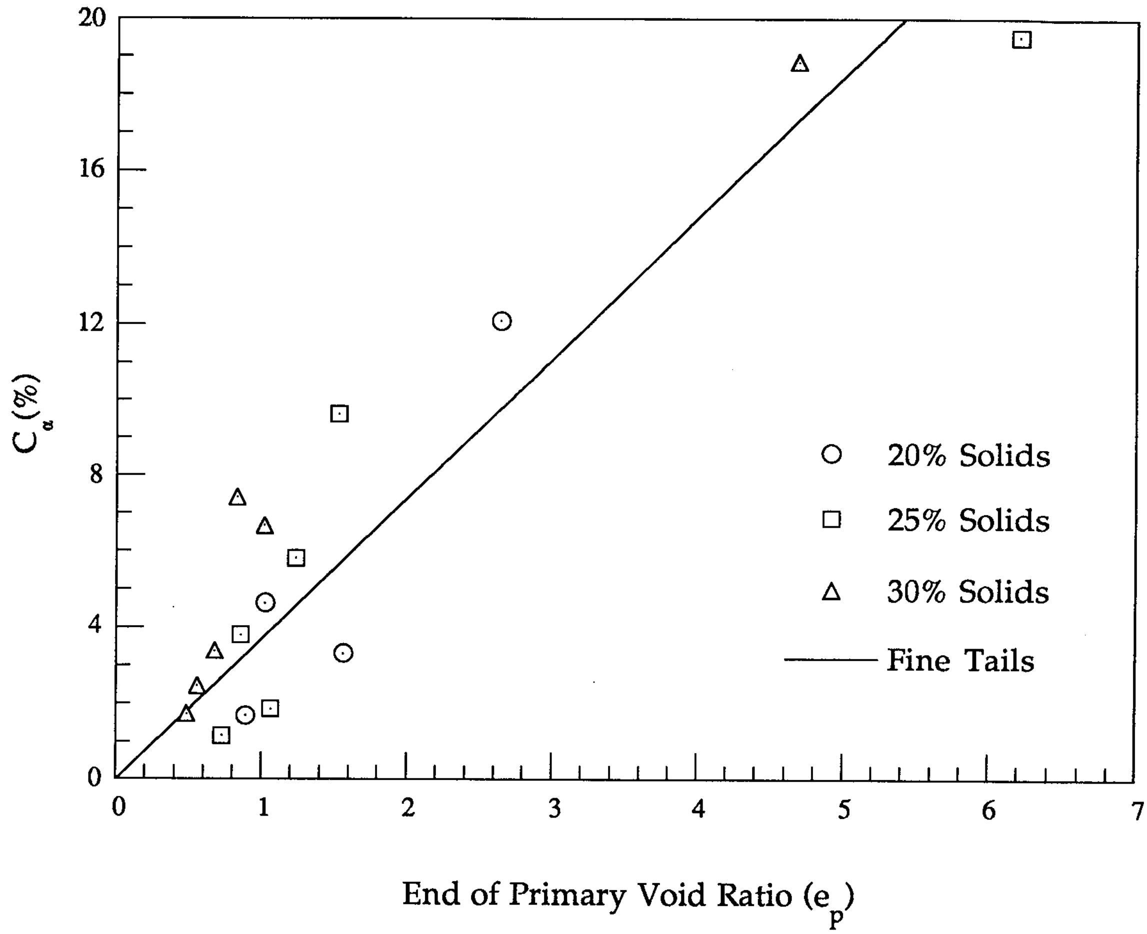


Figure 4.3 Creep Index for Fine Tails

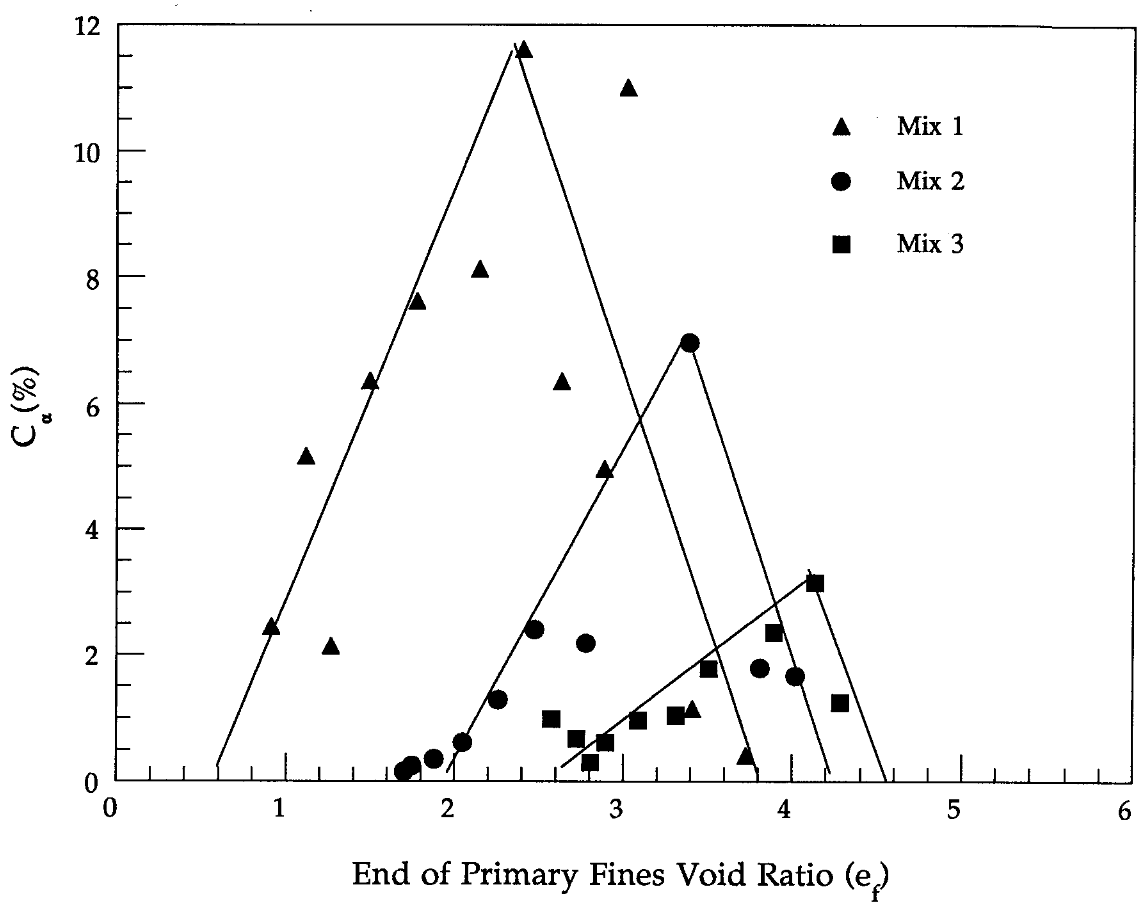


Figure 4.4 Creep Index for Fine Tails - Sand Mixes

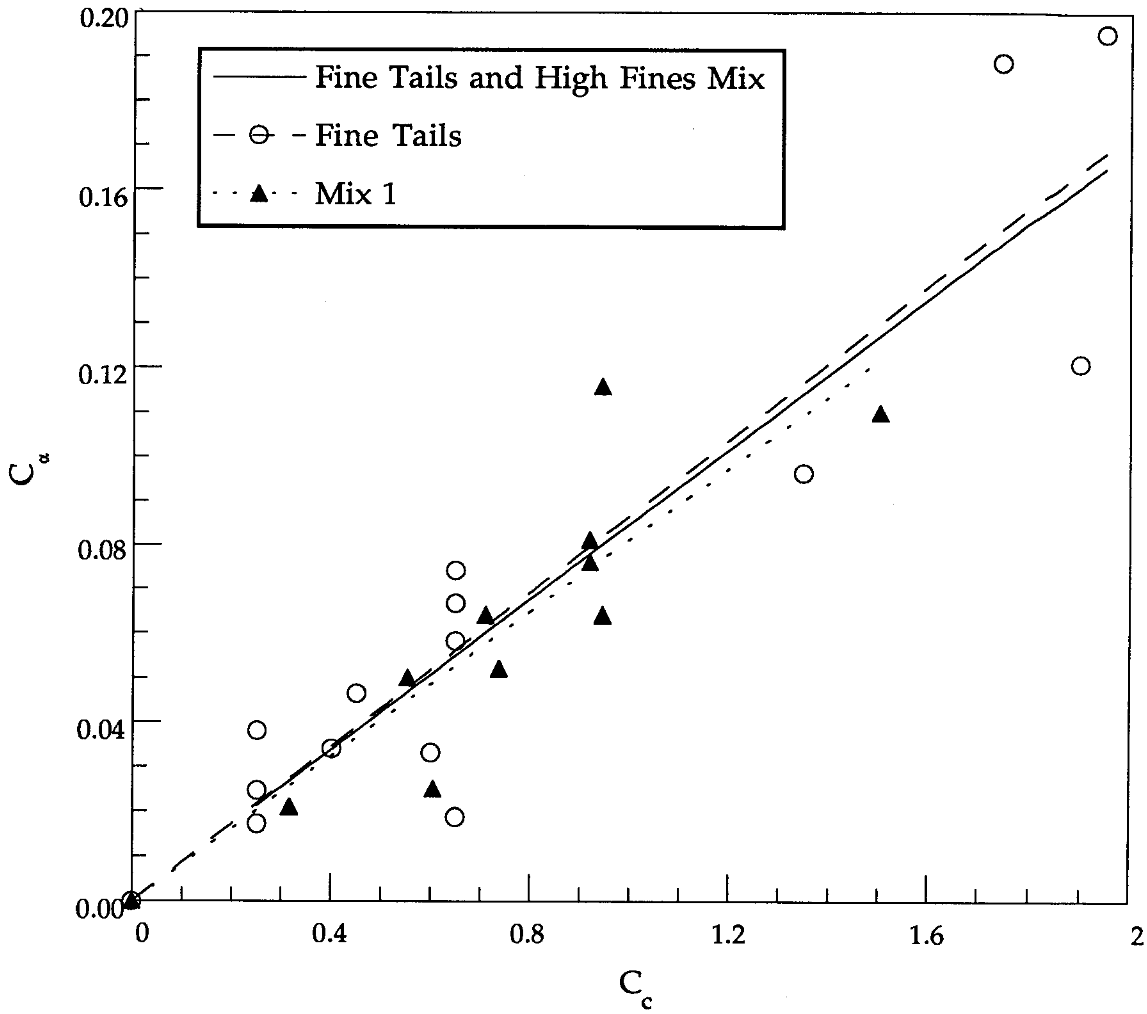


Figure 4.5 C_α - C_c Relationship for Fine Tails and High Fines Mix

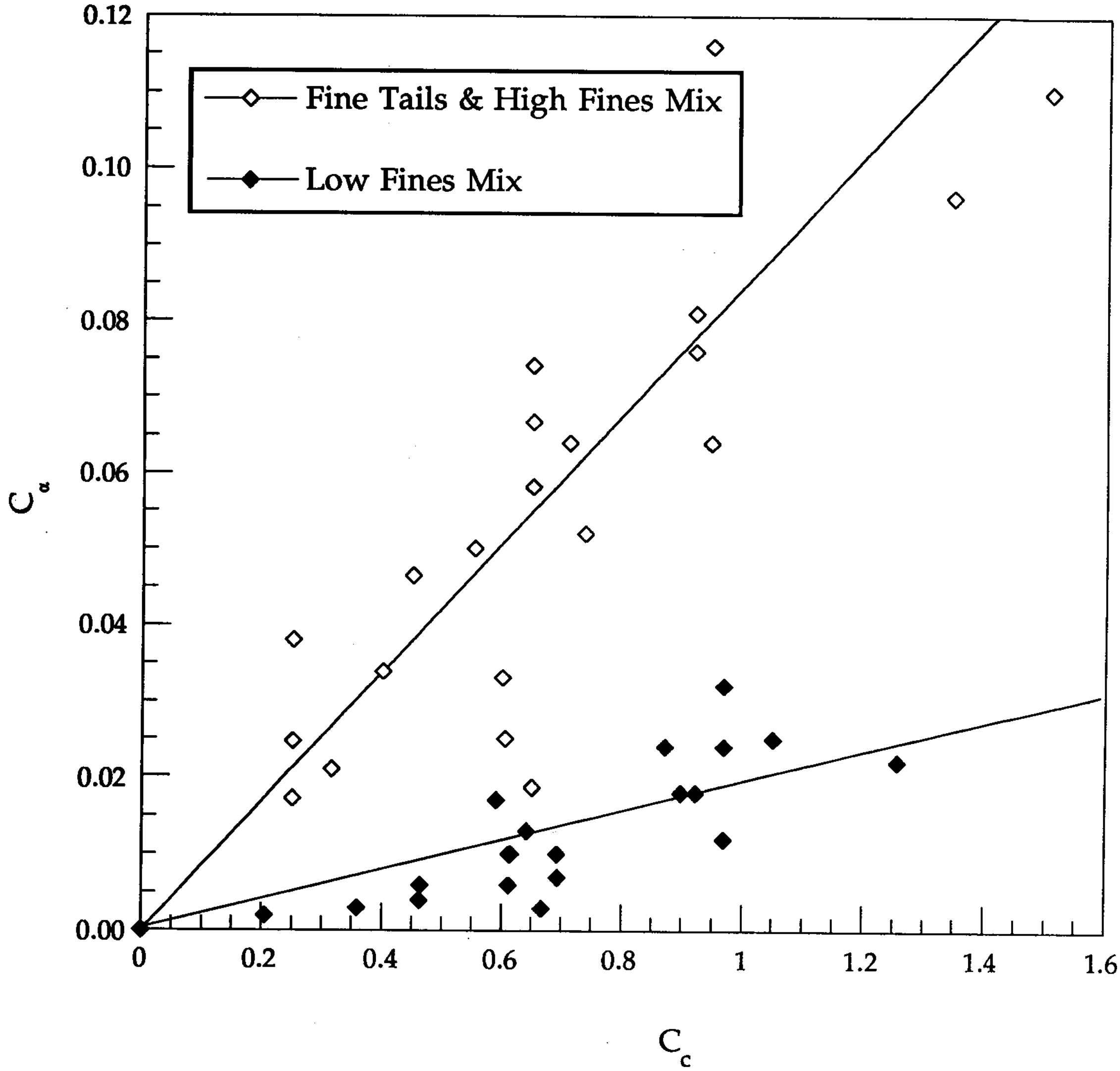


Figure 4.6 C_α - C_c Relationship for Fine Tails and High Fines Mix and Low Fines Mix

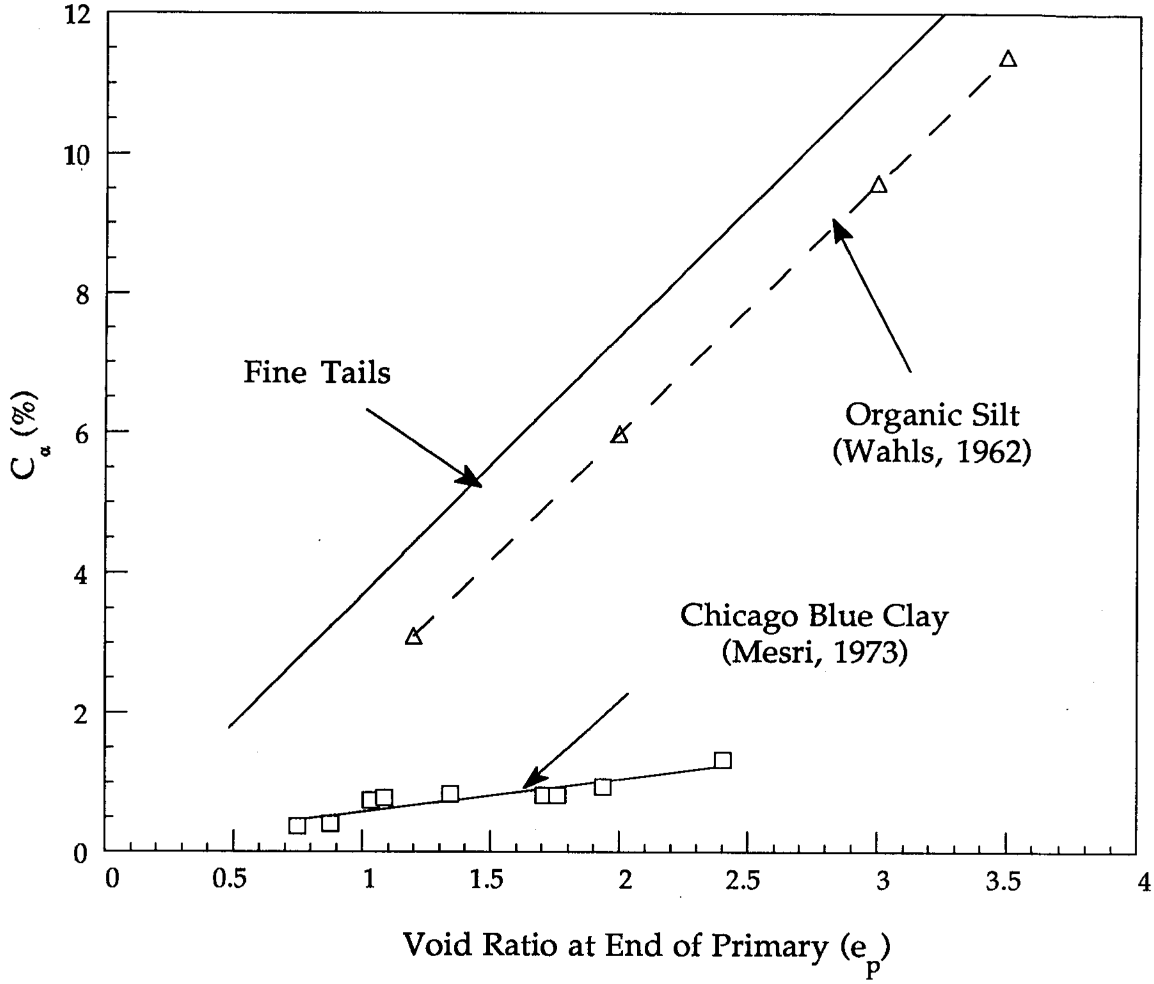
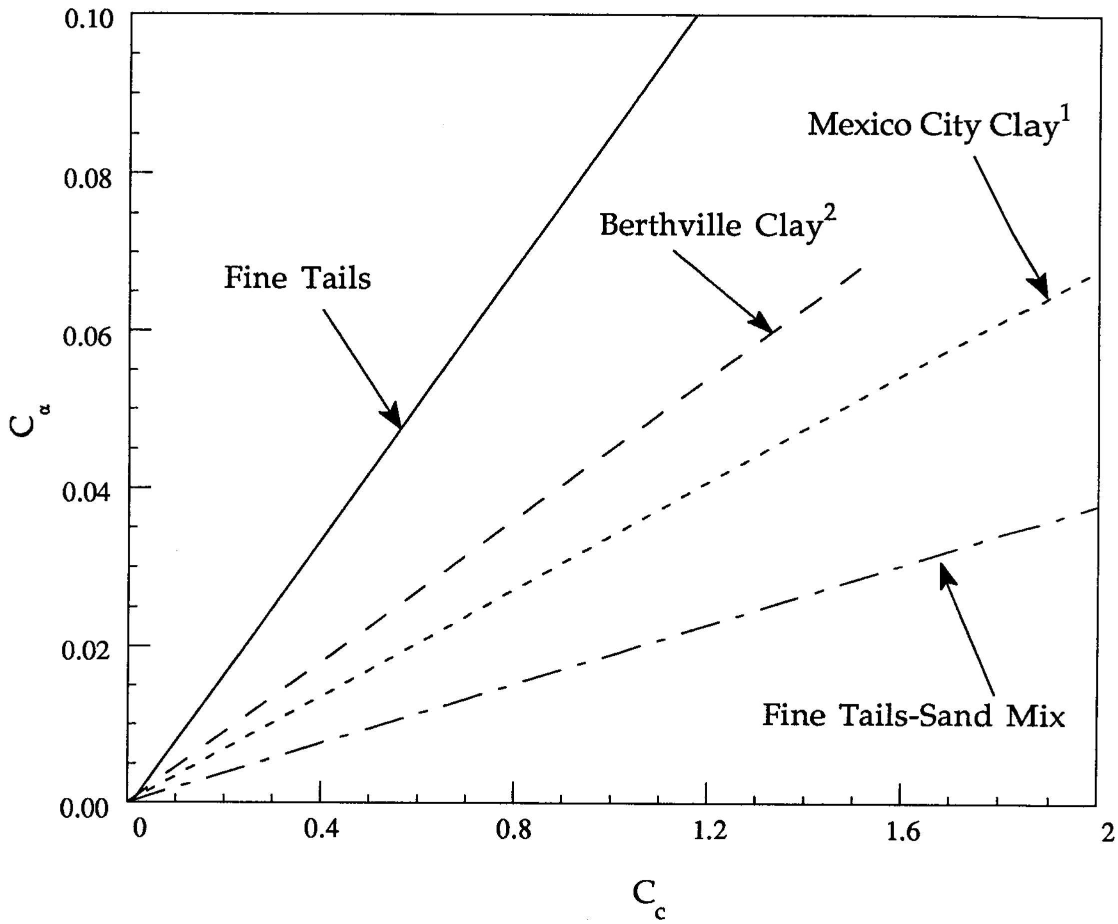


Figure 4.7 Comparison of Creep Index



1- Mesri and Godlewski (1977)
2- Mesri and Castro (1987)]

Figure 4.8 Comparison of C_α - C_c Relationship

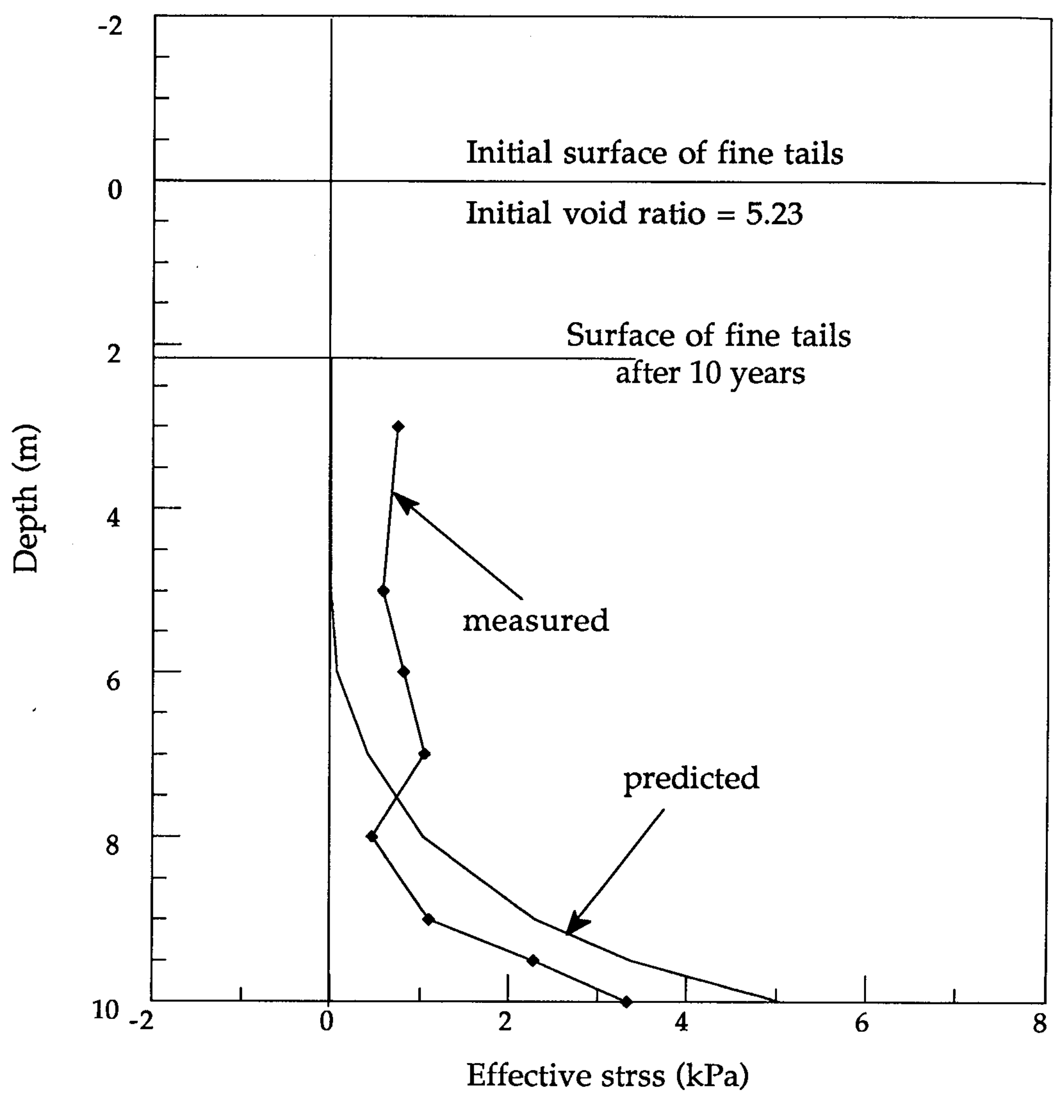


Figure 4.9 Effective Stress in 10 m Deep Deposit of Fine Tails

5. THIXOTROPIC STRENGTH MEASUREMENT OF OIL SAND FINE TAILS[1]

5.1 Introduction

Increasing awareness of environmental issues has brought stricter waste control regulations to industrial operations. The mining industry has traditionally produced vast amounts of waste which cause disposal problems. In surface mining operations the resulting tailings waste is generally in the form of a slurry. The conventional method of disposal of tailings is to contain the tailings in ponds for sedimentation, consolidation, and land reclamation.

In northern Alberta, open pit mining is carried out at two large scale oil sand plants, Syncrude Canada Ltd. and Suncor Inc., to produce synthetic crude oil from oil sands. The waste tailings stream is generally composed of about 85% sand and 15% fines at solids contents from 40% to 60%. The tailings pond dykes and beaches are formed from the sand and some fines. Approximately one-half to two-thirds of the fines and most of the water flow into the ponds to form fine tails deposits. Fine tails is a mixture of silt and clay particles and water with traces of bitumen. Approximately 400 million cubic metres of fine tails are presently held in the tailings ponds. Sedimentation and self weight consolidation are the mechanisms for solid-liquid separation in the tailings ponds. Sedimentation is completed in a short period of time but consolidation is slow which results in a continuous growth of the tailings ponds. Though many factors dictate the consolidation mechanism, one of the unusual factors that appears to affect the rate and magnitude of consolidation of oil sands fine tails is the rapid and large thixotropic strength gain of the material.

This chapter explains the thixotropic phenomenon and the factors affecting thixotropic behavior. The theory, and the test apparatus and procedures for measuring thixotropic strength, such as vane shear tests, viscometer tests and cavity expansion tests, are described. The evaluation and

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[1] A version of this chapter has been submitted for publication inCanadian Geotechnical Journal

discussion of the thixotropic behavior of the fine tails and its implication on fine tails consolidation is the main objective of this work.

5.2 Thixotropy

The term thixotropy was originally introduced to describe the well-known phenomenon of isothermal, reversible gel-sol (solid-liquid) transformation in colloidal suspensions due to mechanical agitation (Mewis, 1979). Thixotropy is also classified as a rheological process. From this perspective it is described as a continuous decrease in apparent viscosity with time under shear, and subsequent recovery of viscosity after cessation of flow (Mewis, 1979). Thixotropy is of interest to the geotechnical engineer as studies suggest that this phenomenon generally occurs in the majority of clay-water systems. From a geotechnical point of view, thixotropy can be defined as a process of softening caused by remolding, followed by a time-dependent return to the original harder state at a constant water content and constant porosity (Mitchell, 1960). Other authors (Locat et al., 1985, Bentley, 1979) discussed thixotropy in conjunction with sensitivity of clays. Thixotropic strength gain is measured as a ratio of the strength after an elapsed time to the strength immediately after remolding (or compaction) and is called the thixotropic strength ratio. Generally, the ratio is determined in terms of undrained shear strength not effective stress shear strength.

The most significant data available to demonstrate the fundamentals of thixotropy is reported by Days (1954). The effect of shear on the forces of cohesion between clay particles was studied through measurements of the intensity of the water adsorptive forces. Days's test results indicated that tension falls to a minimum value after stirring and then gradually returns nearly to the initial value, at which point the process could be repeated. Although no strength values were obtained during these tests, it was noted that the stirring action produced a decrease in viscosity and that the gel slowly returned to its original viscous state as the tension was restored. The phenomenon of decreased tension (or increased pore pressure) occurred in a variety of different clay materials, and appeared to be of quite general occurrence. Aging of material leads to increased water tension (or decreased pore pressure), which increases the effective stress and leads to an increase in strength.

There are two main approaches to the subject, microscopic and macroscopic. Osipov et al. (1984) gave an in-depth view of microstructural changes of clay soils during thixotropic strength loss and restoration. Scanning electron microscopy is used as the main tool in this type of research. Although important in understanding the phenomenon, this approach is of greater interest to colloidal physicists. For this reason any detailed discussion of the microstructure of clays associated with thixotropy is beyond the scope of this study. The macroscopic approach to the subject is considered more relevant and has been investigated herein.

5.2.1 Factors Affecting Thixotropic Behavior

Several factors such as the mineralogy of the clay, water content, and rate of loading have been reported to directly affect thixotropy. A brief discussion of each factor is presented below.

5.2.1.1 Clay Mineralogy.

Skempton and Northey (1952) tested three clay minerals: kaolinite, illite and bentonite. In their findings kaolinite exhibited almost no thixotropy and illite showed a moderate regain of its strength. In contrast, bentonite showed a remarkable regain of strength at very short time intervals and continued regaining strength at a high rate even after a year. This suggests that clays with a high bentonite content should exhibit high thixotropy while clays with a high kaolinite content will generally lack the strength regain. However, tests at the University of California have shown that even kaolinite may be made very thixotropic by the addition of a dispersing agent in order to reduce the degree of flocculation present in the natural material (Mitchell, 1960). Though fine tails clay particles consist mainly of kaolinite (55% to 65%) and illite (30% to 40%) and only a trace of montmorillonite, because of the addition of sodium hydroxide, which is used in oil sand extraction processes, and the presence of organic matter in the form of bitumen, fine tails are thixotropic in nature.

5.2.1.2 Water Content

Water content appears to be of primary importance in thixotropic behavior. Generally, thixotropic effects have been found to increase with increasing water content. Seed and Chan (1957) drew the lower boundary at the plastic limit and found that soils with water contents approaching the plastic limit showed very little or no thixotropy. However, Mitchell (1960) proved that thixotropy can be significant even at low water contents near the plastic limit. The evidence is conflicting at water contents greater than the liquid limit. Skempton and Northey (1952) found that some of the samples were slightly less thixotropic at such water contents while others showed a significant increase in thixotropy. No explanation of this phenomenon has been found in the literature. Seed and Chan (1959) presented evidence that the magnitude of thixotropic effects was not related directly to the Atterberg limits. The liquidity index was the only parameter that was found to be relevant to thixotropic behavior of soil. To some extent thixotropy also depends on the initial soil structure as well as initial water content. In the tailings ponds, however, the soil structure results from sedimentation and consolidation processes and is not a variable. The effect of water content is important in fine tails. The water content of the fine tails decreases with depth in the tailings ponds which then influences the effect of thixotropic strength on the consolidation behavior with depth.

5.2.1.3 Rate of Loading.

The strength of saturated clays generally has been found to be reduced with a decrease in the rate of loading. However, Seed and Chan (1959) suggested that for a specimen whose strength is strongly influenced by thixotropy, the longer the test duration the greater the thixotropic strength acquired by the specimen. Therefore, the total effect of an increase in time of loading would be a combination of two factors: a tendency for the strength to decrease because of the increased time available for creep deformation; and a tendency for the strength to increase because of the increased time available for the development of thixotropic effects.

5.2.2 Thixotropy and Sensitivity

Thixotropy is usually associated with sensitivity of clays. Skempton and Northey (1952) concluded from their work that in clays with medium sensitivity it is possible that sensitivity may be due entirely to thixotropy. This could not be the case in sensitive or extra-sensitive clays where other factors such as leaching or cementation are mainly responsible for high sensitivity. Newland and Allely (1957) found that sensitivity was developed in a highly plastic clay during consolidation under a given load. After thorough remolding of the consolidated clay and reapplication of the same consolidation pressure, a further decrease in void ratio occurred because of the sensitivity.

5.2.3 Thixotropy and Consolidation

Thixotropy can also affect consolidation by altering the compressibility of the soil. Mitchell (1960) suggested that thixotropic effects during consolidation lead to a smaller compression index. However, he recommended more detailed investigations on this subject. In contrast, Berger and Gnaedinger (1949) performed consolidation tests on a plastic clay and noted no significant differences in compression curves for samples tested at 0, 6, and 12 months after remolding.

5.3 Experimental Program

Spherical cavity analysis is used to model aspects of pile and cone penetration where expansion occurs from zero initial radius. Cylindrical analysis is used to interpret the pressure meter test with expansion from a finite radius, determined by borehole size. Elder (1985) developed a laboratory testing procedure for a cavity expansion test to measure undrained strength in marine clays. Since marine clays are also soft clays, the cavity expansion test can be used in the strength measurement of fine tails.

Viscosity is a measure of the internal friction resistance of the movement of each layer of material as it moves past an adjacent layer. Such frictional strength can be correlated to the viscosity of fine tails and can be used as indirect measurement of thixotropy.

The vane shear test has been used to measure the undrained shear strength of soft clays both in situ and in the laboratory. Though it has been criticized by several investigators for its uncertainties and variabilities, vane shear testing remains the most widely used method in soils which are too soft to sample for triaxial testing. Therefore, a vane shear test can be considered as a viable shear strength measurement for fine tails. In this study, the thixotropic strength obtained from the cavity expansion test will be compared to that obtained by viscometer and vane shear tests.

5.3.1 Cavity Expansion Tests (CET)

Spherical cavity expansion analyses are formulated in spherical coordinates (r,θ) where symmetry applies. The self weight of soil is assumed to be negligible. The equation of equilibrium of a soil element at a distance r from the center of the cavity reduces to,

$$\frac{\partial \sigma_r}{\partial r} + \frac{2}{r}(\sigma_r - \sigma_\theta) = 0 \qquad (5.1)$$

where, σ_r is the radial stress and σ_θ the circumferential stress assuming a homogeneous and isotropic soil mass. The cavity is surrounded by a plastic region extending from the current cavity radius R_c (initial value R_{co}) to a radial distance R_p, beyond which linear elastic behavior occurs. The solution of the problem requires use of a relationship describing a yield behavior suitable for soils, the Mohr-Coulomb criterion,

$$\sigma_r - \sigma_\theta = (\sigma_r+\sigma_\theta)\sin\phi + 2c\cos\phi \quad (5.2)$$

Bishop et al. (1945) solved equation 2 for the case of a frictionless (ϕ=0) incompressible medium where it reduces to the Tresca criterion. Gibson (1950) analyzed the cohesionless case (c=0) for a purely frictional material, also assuming incompressible conditions. The most general analysis considers only the conditions at an ultimate cavity pressure and uses the Mohr-Coulomb criterion (Vesic, 1972), which is not valid for ϕ=0, where the Bishop et al. solution applies.

For undrained conditions, the undrained soil moduli are related by G = E/3 and the cohesion, c, which is equivalent to the undrained strength (S_u) (Elder, 1985). This is similar to the conditions analyzed by Bishop et al. (1945) but retains several terms, due to their importance in very soft soils, which had been neglected in the original analysis. The assumptions of isotropy and homogeneity apply and the initial total stress state is assumed to be P_o everywhere in the soil ($r > R_c$). The effect is considered for an increase in cavity pressure from P_o by an amount Δp, causing a radial displacement u(r) and stress changes $\Delta\sigma_r$, $\Delta\sigma_\theta$ at a radial distance r. Yield will occur when,

$$\Delta\sigma_r - \Delta\sigma_\theta = 2S_u \quad (5.3)$$

and that occurs when

$$\Delta p = 4S_u/3 \quad (5.4)$$

If a spherical cavity exists in an isotropic soil mass with initial stress P_o everywhere, the cavity will begin to expand when the cavity pressure P exceeds P_o. The soil behavior will initially be elastic everywhere until the yield criterion (equation 5.1) is satisfied at the cavity surface. The cavity pressure when the first yield occurs will be,

$$P = P_o + \frac{4}{3}c \quad (5.5)$$

In this study, cavity pressure with time was measured. Δp is the change in pressure at yield and was derived from a graph of cavity pressure vs. time. The undrained strength was calculated using the measured Δp at yield from equation (5.4).

5.3.1.1 Experimental Apparatus

Figure 5.1 shows the experimental equipment. A controlled pumping system (ISCO LC-5000 Syringe Pump) was used to create a fluid filled cavity volume at a controlled and preset rate of 15 to 400 ml/h. Thick walled 'zero volume change' nylon tubing was used to connect the pump to the steel tubing (1.6 mm) which had a transducer attached. Two differential pressure transducers with different capacities of 3.5 and 0.85 kPa were used. The first transducer (3.5 kPa) was used for the experiments with 150% and 100% water content fine tails. The second transducer (0.85 kPa) was used in all other tests. The Lab View® software system was used to record cavity pressure at very small time increments (5 data per second) in order to accurately determine the yield pressure.

Initial tests were done with silicone fluid as the injection medium to ensure that there was no fluid flow into the soil surrounding the expanding cavity and a subsequent effect on the pore pressure. The strength of the fine tails was found to be very low compared to the surface tension between silicone fluid and water. Therefore, it was difficult to measure the strength of the material accurately. As an alternative to the silicone fluid, water was later used as a cavity medium which eliminated the surface tension effects. The rate of water injection was evaluated by injecting at different rates up to 40 ml/h to ensure that water leak-off was not significant. When the soil was sampled, the cavities were found intact and filled with fluid. This observation was taken as sufficient evidence that, at least for small cavity volumes, the fluid filled cavity method was suitable.

5.3.1.2 Test Procedure

An initial test procedure established by Banas (1991) based on Elder's (1985) work required further refining. The injecting part of the steel tubing was positioned vertically and water was expelled from the tip to form a drop preventing air from entering the tubing. The tip was then lowered slowly to a depth of 25 mm (1") below the fine tails surface and left there to allow excess pore pressure to dissipate as indicated by steady transducer readings. The cavity was then expanded at a constant rate of volume increase with continuous recording of time and cavity pressure. Limiting pressure was reached within the first 5 seconds, although it depended on the injection rate. The cavity pressure at the tip of the needle was calculated from that measured at the transducer taking elevation heads into account. The strength of the material is calculated from equation 5.4.

Figure 5.2 shows a typical variation of cavity pressure with time during a CET (data shown are for a fine tails sample: initial water content = 233%; age of sample = 30 days; and injection rate = 4 ml/h). The initial constant pressure line denotes the initial effective stress in the fine tails sample. As the cavity pressure increases, the stress in the sample around the cavity also increases and the sample collapse at the yield condition is indicated by the sudden drop in the cavity pressure. The peak pressure corresponds to the undrained shear strength of the fine tails sample tested. A similar trend was observed for all the other samples tested and the strength was calculated accordingly.

5.3.2 Viscometer Tests (VT)

The flow properties of Newtonian fluids are characterized by one parameter known as viscosity (η), which is the ratio of shear stress (τ) to shear strain (γ) ($\eta=\tau/\gamma$). The assumption in this type of flow is that, at a given temperature, viscosity is a constant. Soft clays and slurries display non-Newtonian flow behavior which are both shear and time dependent. The most generalized model is the Herchel-Bulkley model (Banas, 1991) which is represented by

$$\tau = \tau_y + k\gamma^n \tag{5.6}$$

where τ_y is the yield stress, n is the power law index and k is the fluid consistency parameter. The Bingham model $[(\tau - \tau_y)=h_{PL}\gamma]$ and Power Law fluid model $[\tau=k\gamma^n]$ are special cases of this model. Another special case of the Herchel-Bulkley model is the Casson equation $[\tau^{1/2} = \tau_y + k\gamma^{1/2}]$ (Rosen and Foster, 1978).

A Brookfield Synchro-Lectric Viscometer, model RVT was selected because of its simplicity and ease of operation (Banas, 1991). It employs the principle of rotational viscometry and measures the viscosity by sensing the torque required to rotate a spindle at constant speed. The torque is proportional to the viscous drag on the spindle and, thus, to the viscosity. However, using the conventional spindle would have imposed restrictions in this study. It would be required to leave the spindle in the fine tails sample throughout the test which lasted up to 700 days. As well, approximately 200 viscosity measurements were to be made which would require a large number of spindles. In order to avoid these complications, a T-bar spindle was used. The viscometer had a helipath stand which combined with the T-bar spindle resulted in negligible disturbance to the material during insertion of the spindle compared to normal spindles. The T-bar spindle has been calibrated to determine viscosity of Newtonian fluids. However, no mathematical models are available for calculating viscosity of non-Newtonian fluids with T-bar spindles as the shear rate cannot be determined. Therefore, measurements conducted with this spindle were not treated as viscosity values but rather as index values. Each viscosity test consisted of four sets of measurements which were performed at four different rotational speeds 1, 2.5, 5 and 10 rpm. Two groups of readings, peak and residual values were taken in each test.

5.3.3 Vane Shear Tests (VST)

Cadling and Odenstad (1950) developed the relationship between shear strength and torque as

$$S_u = \frac{2T}{\pi D^2\left(H + \frac{D}{3}\right)} \tag{5.7}$$

where S_u is undrained shear strength, T is torque, D is vane diameter and H is vane height.

A modified Wykeham-Farrance miniature vane shear apparatus was used in this study. The hand operated drive mechanism of the conventional vane shear apparatus was replaced with a motor. The motor was connected with the drive mechanism through two belt-driven gears. This arrangement allowed a constant vane rotational speed any where between 0 and 120 rpm. The torque measuring spring was replaced with a rigid transducer. Three custom made vanes were used for the strength testing. Two of them, with the height to diameter ratio of 2, had four blades each. The third one had a height to diameter ratio of 1 and had six blades. The testing procedure was in accordance with the ASTM D 4648-87. Vane rotational speeds were determined according to an angular velocity of 0.17 mm/s. The viscometer and vane shear tests were performed by Banas (1991) at the University of Alberta.

5.4 Testing Program

A set of index tests such as water content, Atterberg limits, grain size distribution, and bitumen content were done on representative specimens of fine tails from the Syncrude tailings pond to classify the material. The fine tails was received in two batches (168% and 270% water content). Five different water contents, 400%, 300%, 233%, 150%, and 100% were investigated. These particular water contents were chosen to represent layers of fine tails of different age during long term consolidation. The 400% water content (20% solids content) fine tails represents the thin slurry after completion of the sedimentation process. After reaching this solids content any increase in fine tails density arrives from the process of self weight consolidation (Scott and Dusseault, 1982). It takes about two years for the fine tails in the pond to consolidate to a solids content of 30% (233% water content). Fine tails at this point is considered mature and anything less than 30% solids is called immature fine tails. Therefore, in this research, two 'immature' water content fine tails (400% and 300%) and three 'mature' ones (233%, 150% and 100%) were investigated. The vane shear tests were also performed for samples at the liquid limit (water content of 47%) to compare with other researchers' work. Cavity expansion tests were done at four different injection rates (1.5 ml/h, 4 ml/h, 15 ml/h, and 40 ml/h) to study the effect of injection rate on the thixotropic strength measurement.

Age of the fine tails was perhaps the most important variable in the study of thixotropy. It was defined as the time from the moment when the process of remolding, or physical agitation ceased. Specimens were tested from age zero up to 680 days. Viscometer, vane shear and cavity expansion tests were conducted at each water content at different ages of the samples (0, 5 hrs, 1 day, 2 days, 5 days, 15 days, 30 days, 60 days, 100 days, 350 days, 450 days and 680 days).

5.5 Results and Discussion

The initial water content of the two batches of fine tails were significantly different. Batch 1 fine tails showed a 168% water content whereas the Batch 2 fine tails had a water content of 270%. This difference is explained by the history of the two fine tails. Batch 1 had been stored in a tank for several years prior to sampling and the was taken from the bottom of the tank without mixing of the contents. Thus, it had a low water content. Batch 2 was collected directly from the tailings pond shortly before it was received in the laboratory.

Atterberg limit tests on representative fine tails were performed. The purpose of these tests was to classify the material and to determine the differences between the two batches of the fine tails. Table 5.1 summarizes the Atterberg test results.

Scott et al. (1985) reported the range of liquid limit data for fine tails to be between 40 and 80%. The variations in liquid limit data reflect several factors other than clay mineralogy such as ionic concentration, bitumen content and total percentage clays in the mineral solids. In general, higher values reflect greater bitumen content and finer-grained fine tails. The results of the test materials fall within these boundaries.

The differences in the liquid limits and plastic limits between Batch 1 and Batch 2 are small and should not have any significant effect on strength test results. The bitumen content of samples were determined by toluene extraction and is defined as mass of bitumen over the mass of mineral grains. Batch 1 fine tails displayed a slightly higher bitumen content.

The Batch 1 and Batch 2 fine tails were mixed together and diluted with tailings pond water or air dried to make samples at different water contents. Figure 5.3 shows the grain size distribution of the different water content fine tails samples and they are consistent.

5.5.1 Thixotropy From Cavity Expansion Tests

5.5.1.1 Consolidation Effects

It is important to note that during the test period, the strength developed was not totally due to thixotropy, but also due to a decrease in water content from self weight consolidation. This is evident by the variation of water content with the age of fine tails (Figure 5.4). For the 100% initial water content sample, no change of water content was observed. Therefore, consolidation effects did not prevail at 100% water content. A constant water content was observed for 165 days, 21 days, 15 days and 15 days for the 150%, 233%, 300% and 400% fine tails water content samples respectively. Therefore, consolidation effects will start to show only after the time given above for the different initial water contents. In assessing the actual thixotropic strength, such effects caused by consolidation had to be determined.

Corrections for consolidation effects were done by interpolation between the water contents for each sample. Figure 5.5 shows the total strength of fine tails with a water content of 233%. The strength reported in Figure 5.5 includes both thixotropic and consolidation strengths. The differentiation of thixotropic strength from the strength due to consolidation is illustrated below for the initial fine tails water content sample of 233%. Until 500 h (21 days) (Figure 5.4), the consolidation effects are zero and no correction was required. At 8423 h (351 days), if no consolidation effects existed, the water content would be 233% (Point A, Figure 5.4). However, the water content of the sample decreased to 210% (Point B, Figure 5.4). Thus, it was required to determine the strength of the sample if the water content was unchanged. From the adjacent 300% water content plot its strength (Point C) was determined. The corrected strength (Point A) was then determined by interpolation between Points B and C. The corrected strength plot is also shown in Figure 5.5. Figure 5.6 shows the corrected thixotropic strengths with age for all the fine tails water contents.

5.5.1.2 Effects of Strain Rates in Thixotropic Strength.

Figure 5.5 depicts the effects of different strain rates tested. The strain rate is indicated by the fluid injection rate (ml/h). Although the strength is different for different injection rates, maximum strength is seen at different injection rates for different ages of fine tails. This may be due to small inhomogeneities in the sample. It can be concluded that the effect of strain rate was not significant over the 1.5 to 40 ml/h range.

5.5.1.2 Effect of Age of Fine Tails

A quadratic increase of thixotropic strength with age was seen which was still increasing after 450 days (Figure 5.6). This thixotropic strength development pattern for fine tails was different from that obtained for marine clay (Locat et al, 1985) where a rapid and short thixotropic strength recovery was followed by a slow and long recovery. It can also be observed in Figure 5.6 that the lower the water content, the higher the thixotropic strength. A substantial strength gain is seen when the water content decreases from 150% to 100%. The rate at which the thixotropic strength develops can be visualized from the slope of the curves. A much greater rate of thixotropic strength development was observed for the sample with an initial water content of 100%. Thixotropy development rates for the other samples tested were much closer to each other and smaller than that of the 100% initial water content sample. The thixotropic strength ratio can be used to characterize the strength gain during a period. In general, the thixotropic strength ratio was found to decrease with decreasing water content (Figure 5.7).

5.5.1.2 Effect of Water Content

The effect of water content on thixotropic strength at different ages of fine tails is shown in Figure 5.8. In general, thixotropic strength decreased with an increase in water content at any given age of fine tails. When the thixotropic strength ratio is considered (Figure 5.9) an interesting phenomenon, a minimum thixotropic strength ratio, was observed for the 150% water content sample. This phenomenon is clearly visible after 100 days. The results are similar to what was observed by Skempton and Northey (1952) and Mitchell (1960).

5.5.2 Comparison Of Thixotropy Obtained From Different Tests

5.5.2.1 Thixotropic Strength

Thixotropic strength of the fine tails measured by the vane shear tests are depicted in Figure 5.10. In general, the thixotropic strength measured by the vane shear test is lower than that obtained by the CET. The thixotropic strength obtained by the vane shear test followed a quadratic form as for the strengths obtained by the CET. However, unlike in the CET, the thixotropic strength declined after some time in the vane shear tests. This may be due to the fact that the sample heights (200 mm) used in the vane shear tests are 2 to 3 times that of the cavity expansion tests (75 mm to 100 mm). This larger height would lead to larger amounts of consolidation in the vane shear test samples. Although corrections were made for decreases in water content, the consolidation settlement also reduces the physico-chemical bonding produced by thixotropy. This reduction in thixotropic bonding will be higher in the long term vane shear test samples. Figure 5.11 shows the same trend for the vane tests as Figure 5.8 shows for the cavity expansion tests but with a difference in magnitude.

A low measurement of vane shear strength (Figure 5.11) also may be due to progressive failure in the sample. A vane shear test assumes a simultaneous mobilization of strength on a cylindrical failure surface in the soil. In this study the vane mostly used was 80 mm high by 80 mm in diameter and had six blades. In high water content samples the failure probably started to occur at the tip of the vane by local shear and progressed slowly. The potential cylindrical shear surface would be at different levels of strain and strength at the highest torque measurement which would lead to an underestimation of the strength.

5.5.2.2 Thixotropic Ratio

Thixotropic ratios for the CET, VST and VT tests are presented in Figures 5.12 and 5.13 at different ages for different water contents. Although the thixotropic strength was underestimated in the vane test, the ratio appears overestimated possibly due to the initial strength being underestimated by the vane. The initial strength will be more underestimated in the high water content and low strength samples. Figure 5.14 depicts the effect of water content on thixotropic ratio. Compared to that obtained in the CET (Figure 5.9), the thixotropic ratios of the VST were larger. It should also be noted that the trend of the curves in Figure 5.9 (CET) and Figure 5.14 (VST) are reversed. It would appear that the higher water content samples, which had substantial self weight consolidation, had long term strengths that were substantially reduced by the shearing stresses during consolidation.

Absolute strength measurements in the viscometer tests were not possible as no correlation has been established for the T-bar spindle used in this tests. However, the determination of thixotropic ratio is possible by using the test results as index values. Figures 5.12 and 5.13 show these thixotropic ratios. The viscometer tests were performed in the same samples as the vane tests and show similar test results. The higher water content values, however, would be similarly affected by consolidation as the vane tests. Figure 5.15 shows the effect of water content on the thixotropic ratio obtained in the viscometer tests.

Traditionally, thixotropic ratio is used to compare thixotropy from different test procedures. Thixotropic strength starts to develop instantaneously after remolding (Figure 5.5). The initial strength measurement, therefore, may affect the thixotropic ratio. This would reduce the accuracy of the comparison of thixotropic ratio obtained from different testing procedures having different amounts of error in the initial strength measurement. Most engineering design applications, however, require the measurement of absolute strength not the ratio. In view of this, accurate thixotropic strength measurements are important. Cavity expansion tests, introduce very little disturbance, are not affected by progressive failure, and hence, better measure the thixotropic strength.

5.5.3 Comparison with Other Thixotropic Soils

Moretto (1948) tested natural clays for thixotropic strength at water contents at or close to the liquid limit. Unconfined compression tests were conducted on the clay samples at different time intervals after remolding. Skempton and Northey (1952) reported the results for three different sensitive clays and three clay minerals. Figure 5.16 shows the comparison of the thixotropic strength at water contents at the liquid limits for fine tails and natural clays. The fine tails has the highest thixotropic strength ratio and only Beauharnois clay is similar. The other four natural clays show low thixotropic strength ratios below 2.

Figure 5.17 similarly depicts the thixotropic ratios for three clay minerals, kaolinite, illite and montmorillonite with the fine tails. Kaolinite shows relatively no strength gain and illite only shows a small strength gain. In contrast, montmorillonite shows a high rate of strength gain at very short time intervals, about 50 days, and after that the rate reduces significantly. Fine tails show a significantly higher thixotropic effect when compared to the clay minerals. This supports the results obtained by Mitchell (1960) which suggests kaolinite, which exhibits low thixotropy, can be made thixotropic by the addition of dispersing agents.

5.6 Summary and Conclusions

1. Cavity expansion tests can be successfully used to estimate the thixotropic strength of fine tails and other similar materials. The strain rate in cavity expansion tests did not significantly affect the strength.

2. Long term strength developed in slurries can be either thixotropic or a combination of thixotropic and consolidation strengths. At low water contents (< 100%), the effect of consolidation strength development did not exist as the water content remained unchanged. At higher water contents, water content changes were prominent indicating the existence of consolidation effects. Interpolation techniques, however, were used to isolate the thixotropic strength development.

3. Oil sands fine tails is a highly thixotropic soil. It exhibits higher thixotropic gain in strength than typical clays and clay minerals. The thixotropic strength in the fine tails developed rapidly and increased quadratically with age. Strength increases were still continuing at the end of test period of 450 days.

4. The thixotropy of fine tails is highly dependent on water content. Substantial thixotropic strength increases were seen for fine tails with water contents less than 150%. The lower the water content, the higher the thixotropic strength.

5. The thixotropic strength ratio for fine tails was at a minimum at 150% water content. Generally, the thixotropic strength ratio increased with an increase in water content.

6. The thixotropic strength may be affected by self weight consolidation. Particle shearing resulting from consolidation reduces physico-chemical bonding produced by thixotropy. The higher the rate of consolidation, the smaller the thixotropic strength gain.

7. The study of the physico-chemical properties of fine tails should follow the same time-consolidation strength history as that of the fine tails in the tailings ponds to model the combined effect of thixotropy and consolidation.

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Table 5.1 Index Properties of Test Materials

Sample	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Bitumen Content (%)
Batch 1	47	20	24	6
Batch 2	44	21	23	5

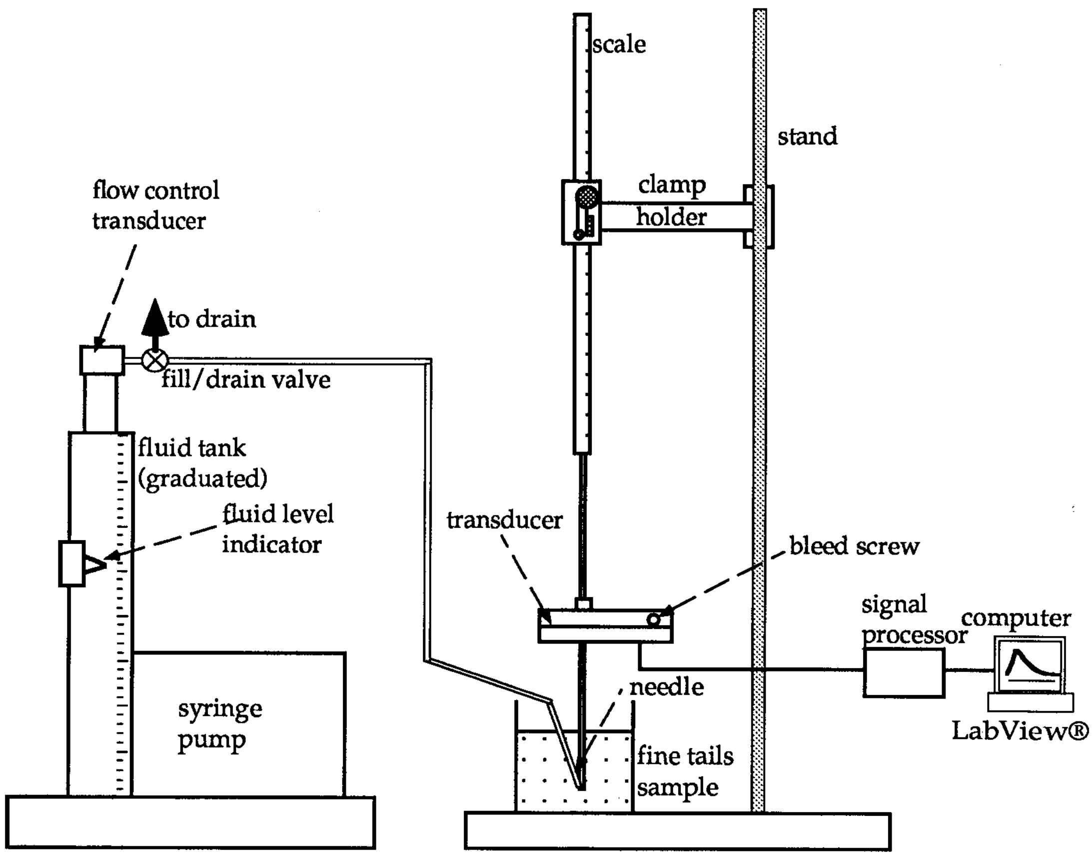

Figure 5.1 Cavity Expansion Test Apparatus

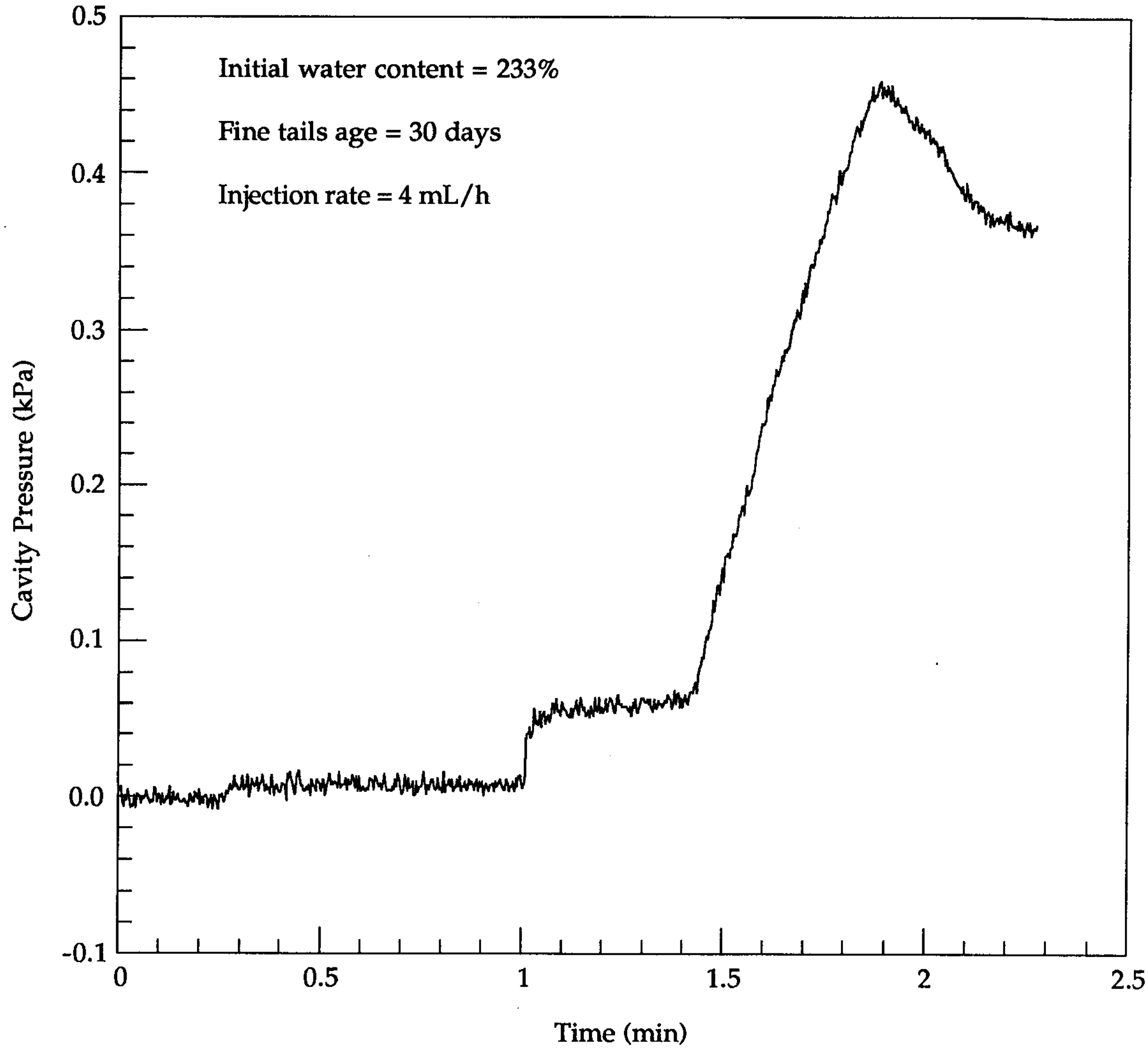

Figure 5.2 Variation of Cavity Pressure with Time

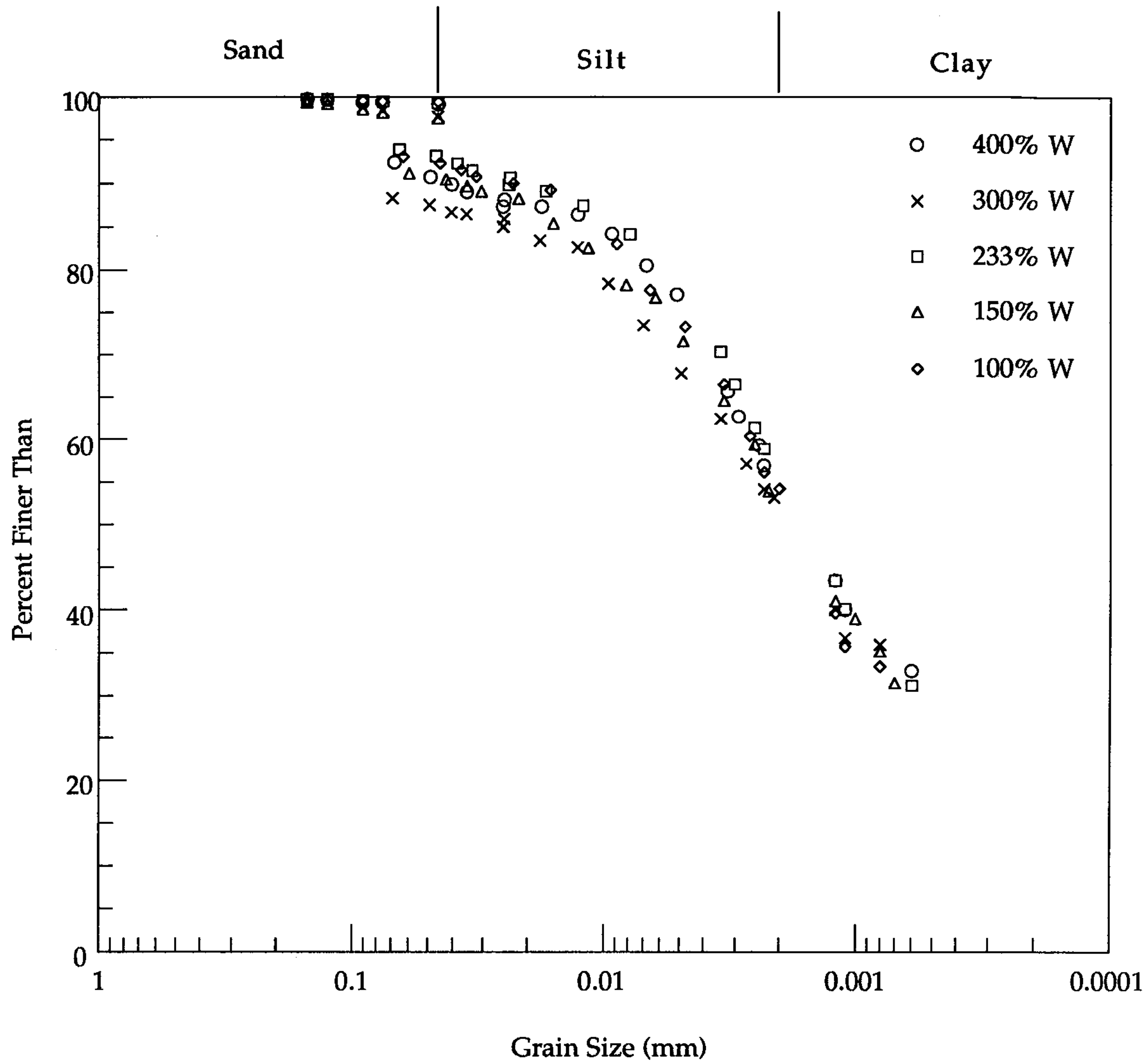

Figure 5.3 Grain Size Distribution of Fine Tails

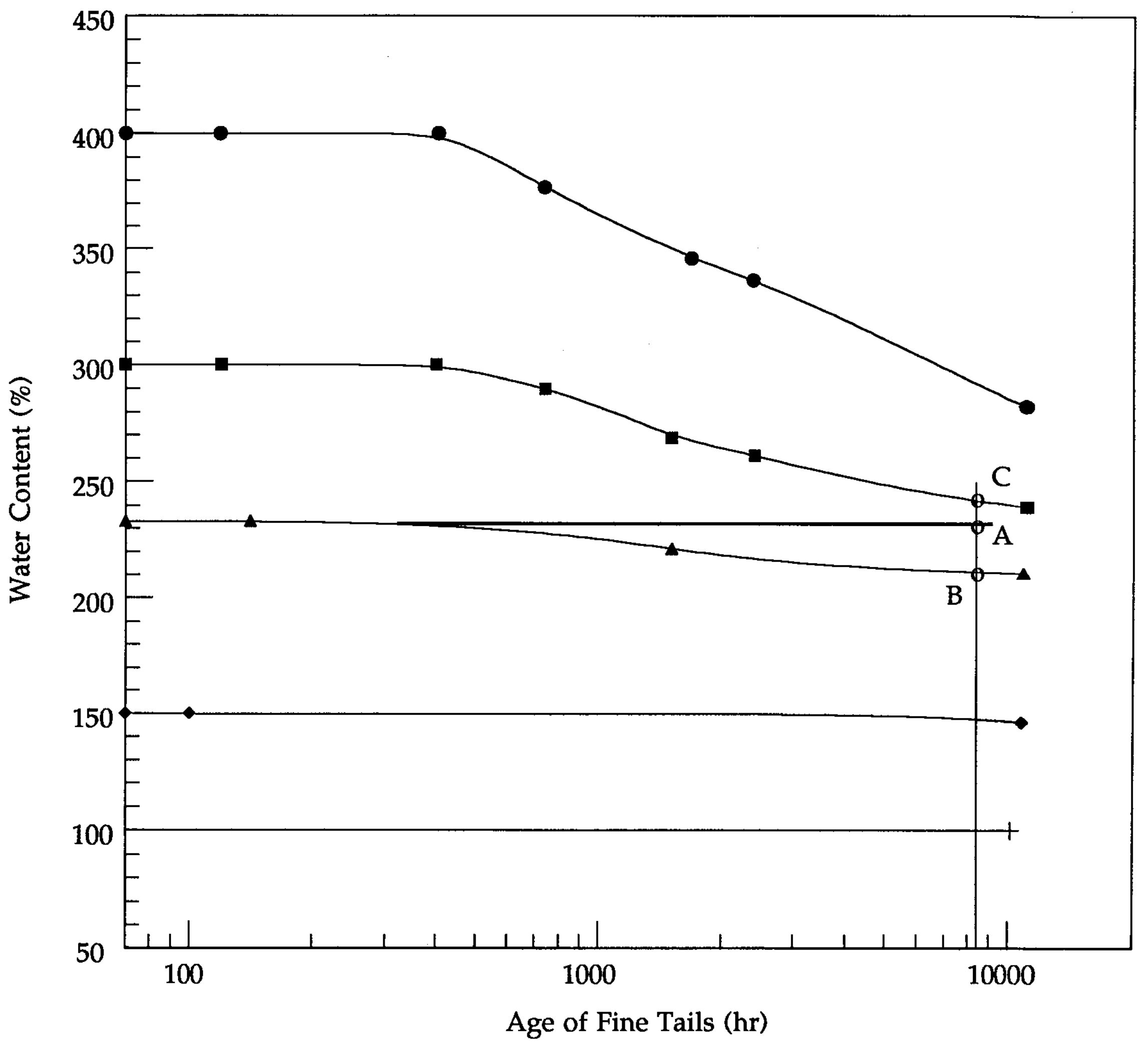

Figure 5.4 Self Weight Consolidation of Fine Tails in Cavity Expansion Test Samples

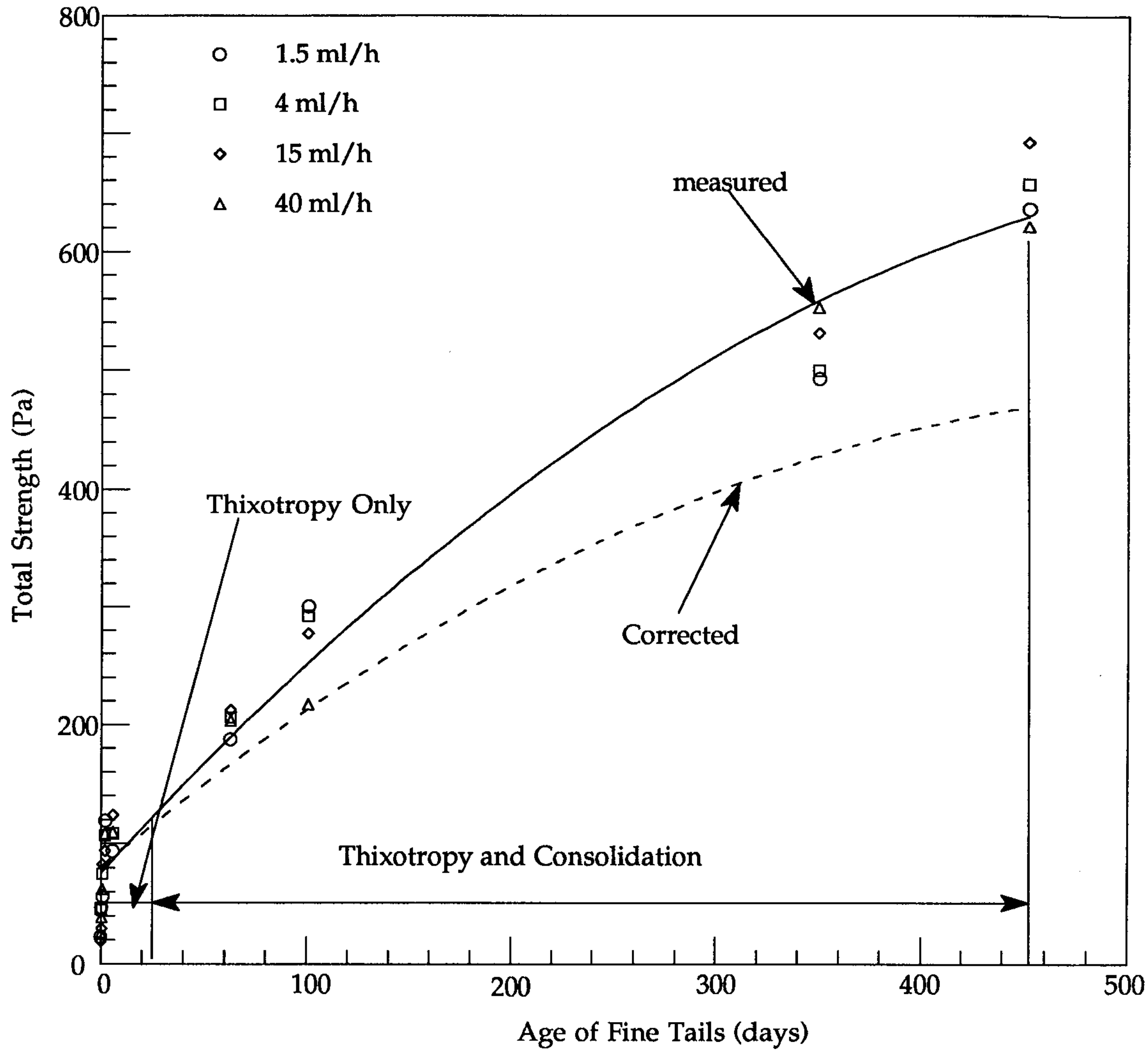

Figure 5.5 Thixotropic Strength with Time for 233% Initial Water Content Cavity Expansion Test Sample

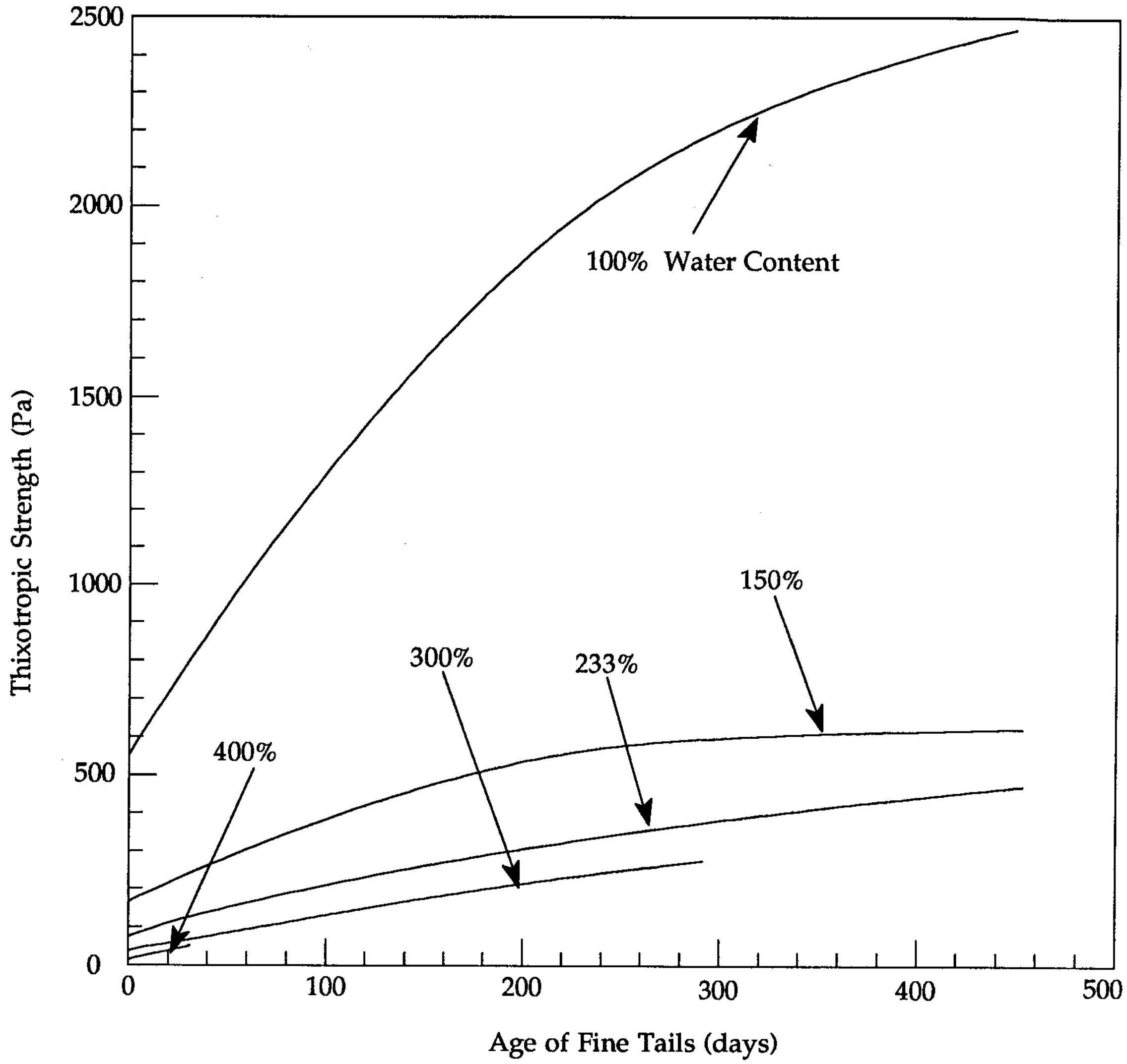

Figure 5.6 Thixotropic Strength With Time for Cavity Expansion Tests

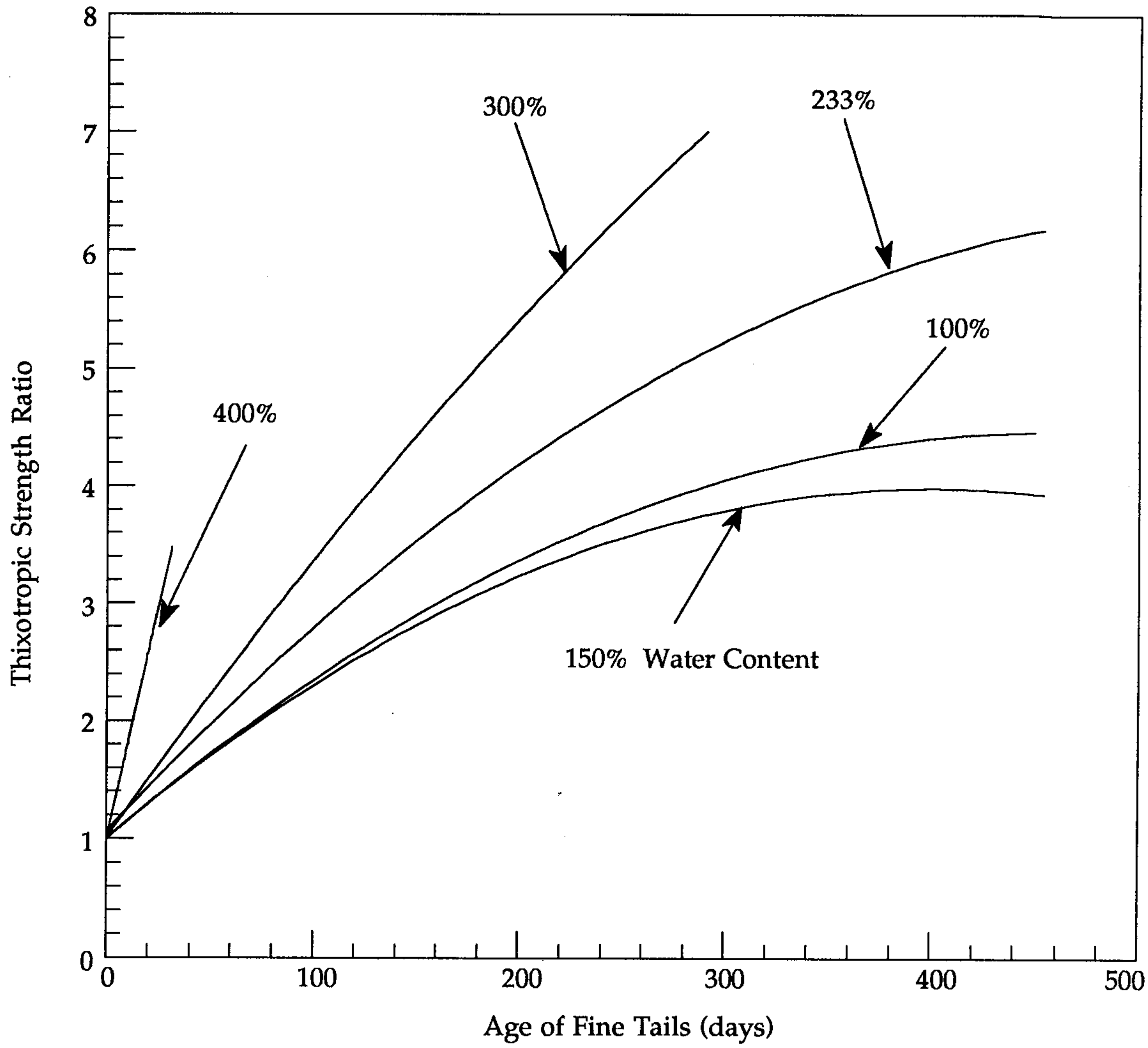

Figure 5.7 Thixotropic Strength Ratio with Time for Cavity Expansion Tests

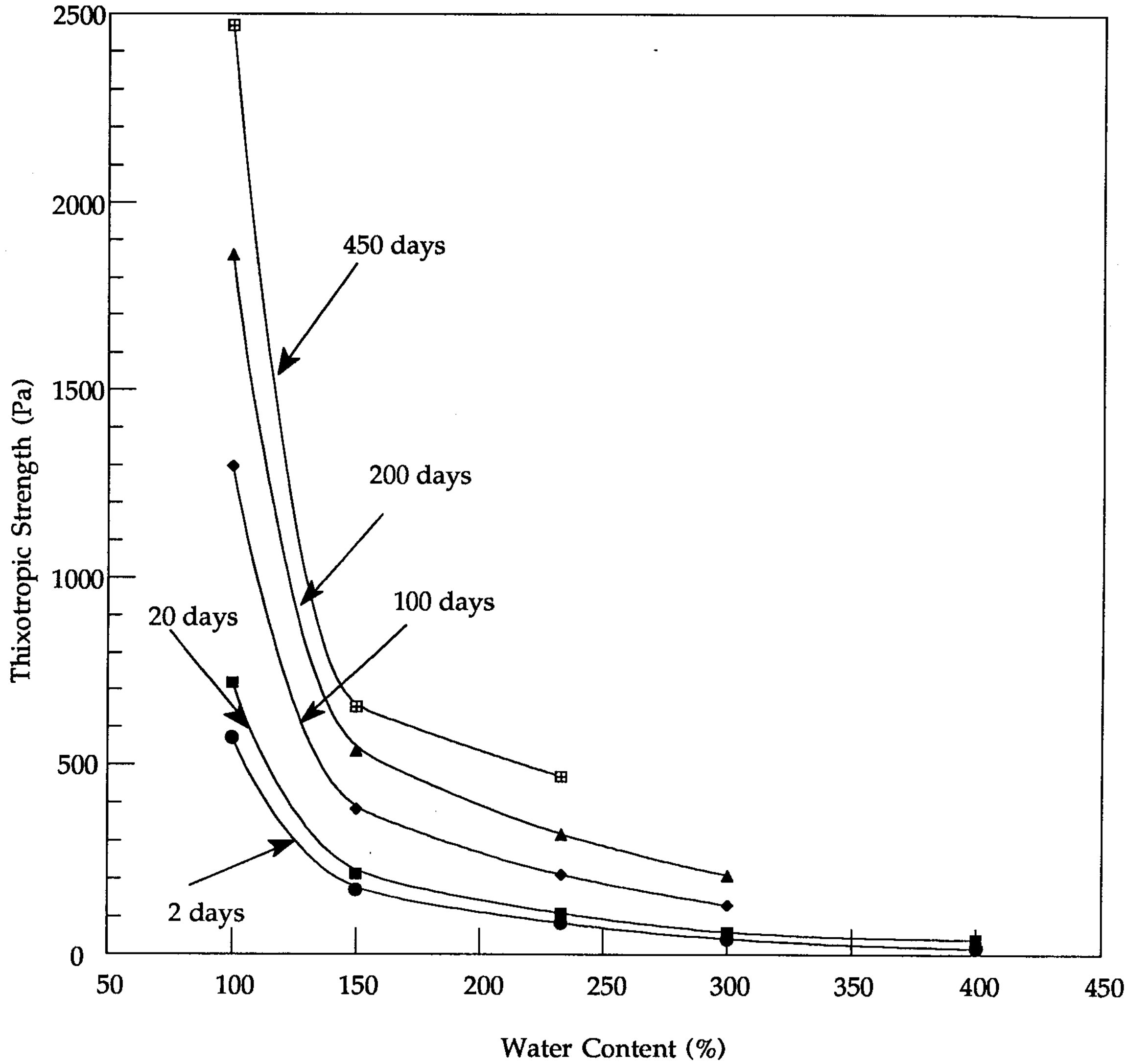

Figure 5.8 Thixotropic Strength for Cavity Expansion Tests

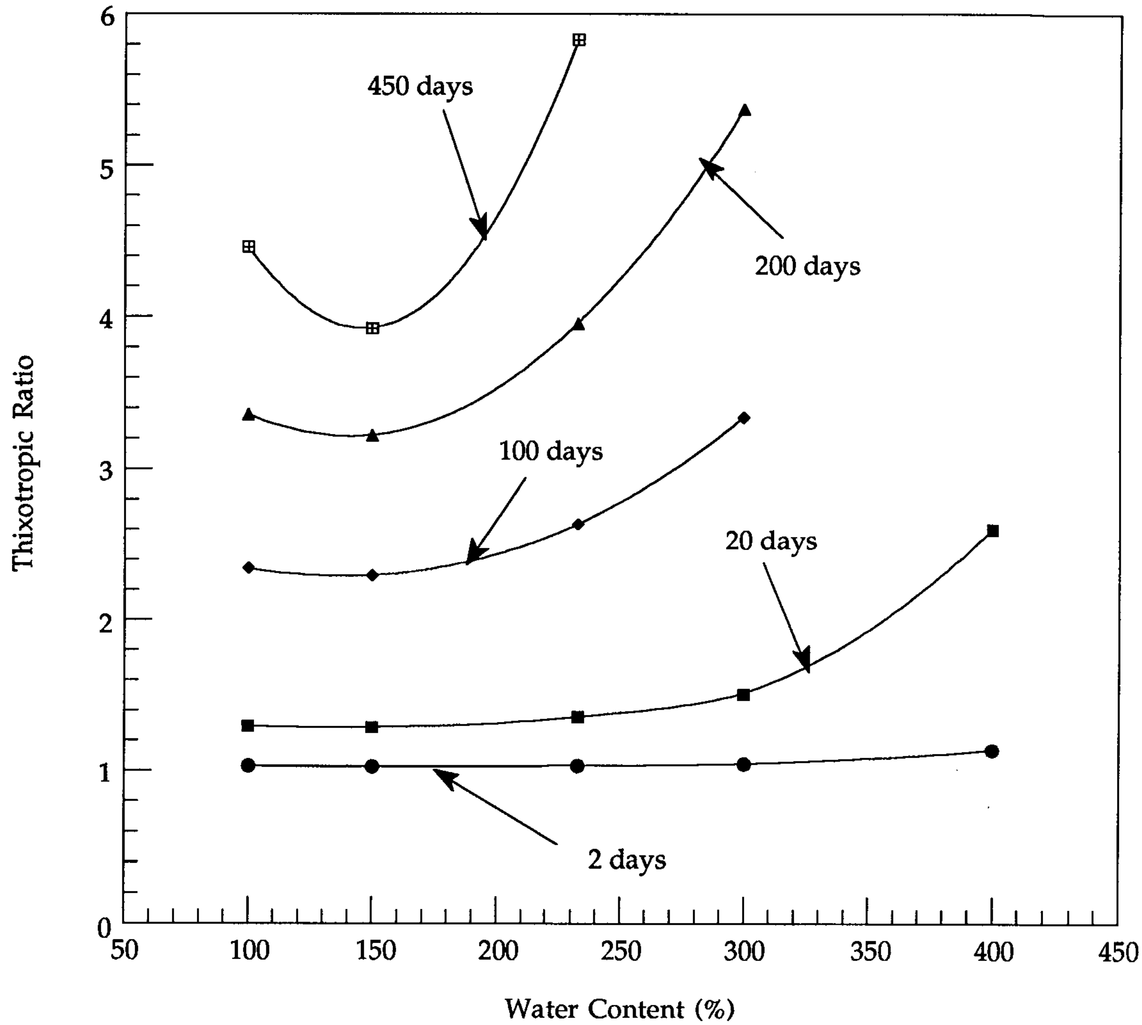

Figure 5.9 Thixotropic Ratio for Cavity Expansion Tests

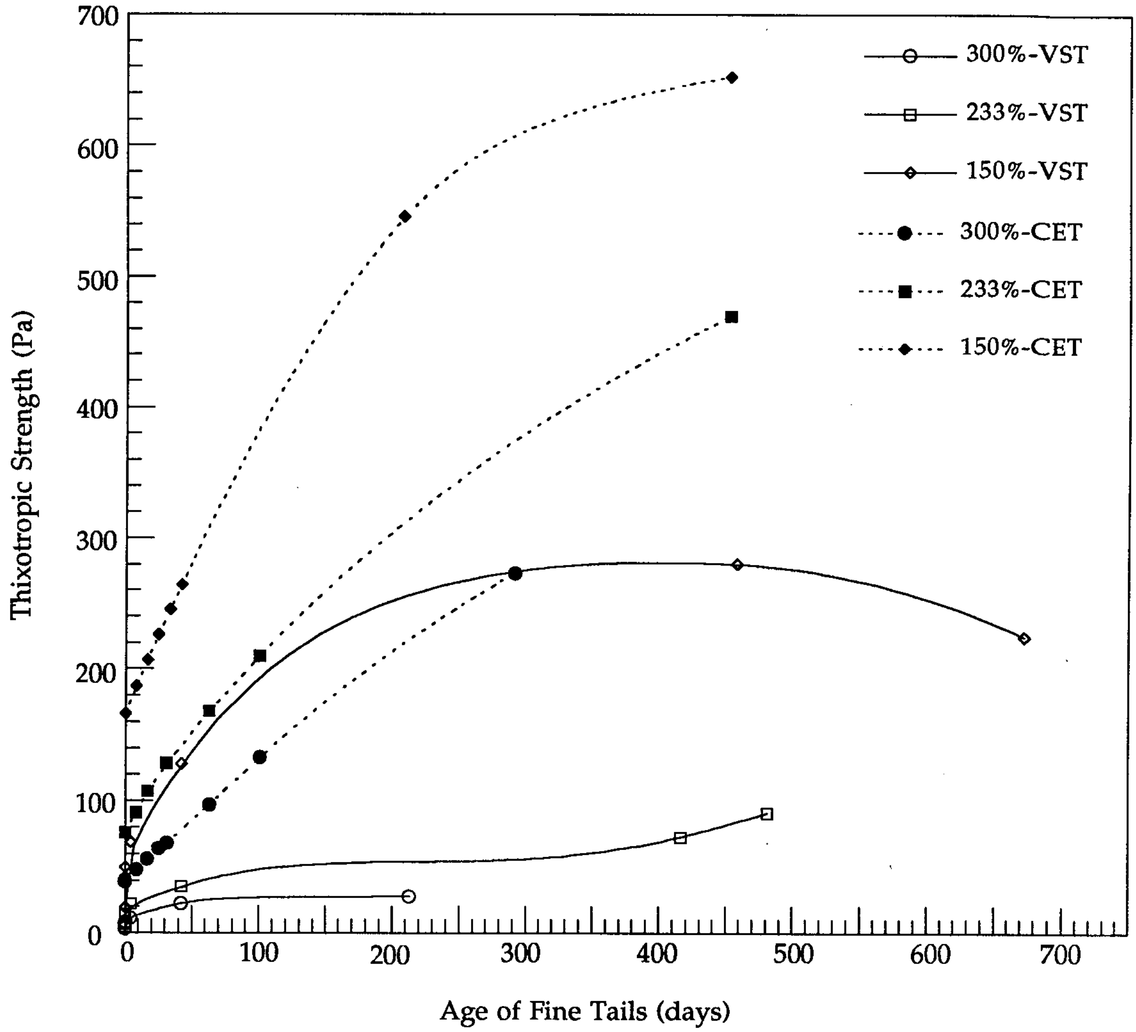

Figure 5.10 Comparison of Thixotropic Strength Measurements

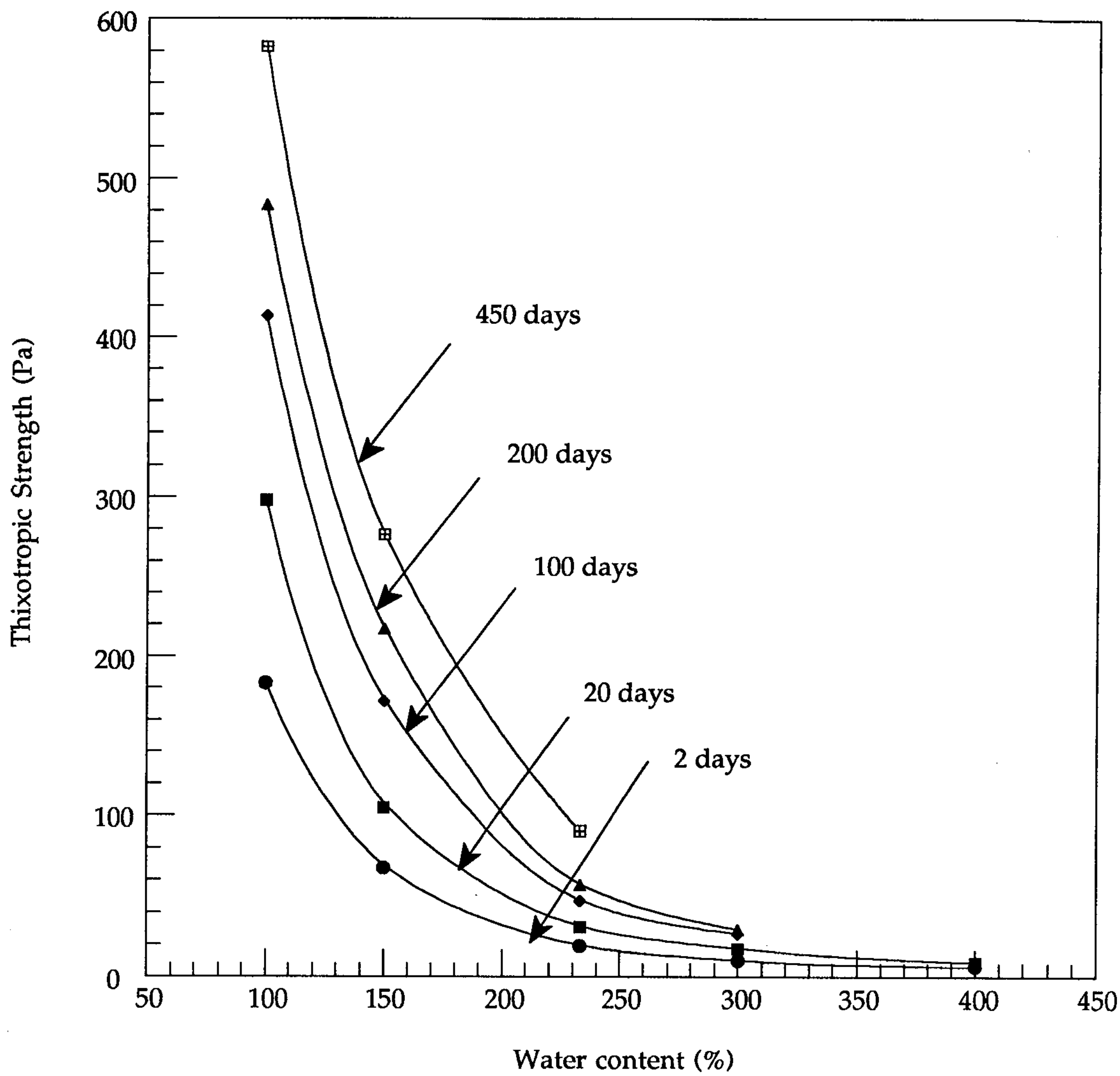

Figure 5.11 Thixotropic Strength for Vane Shear Tests

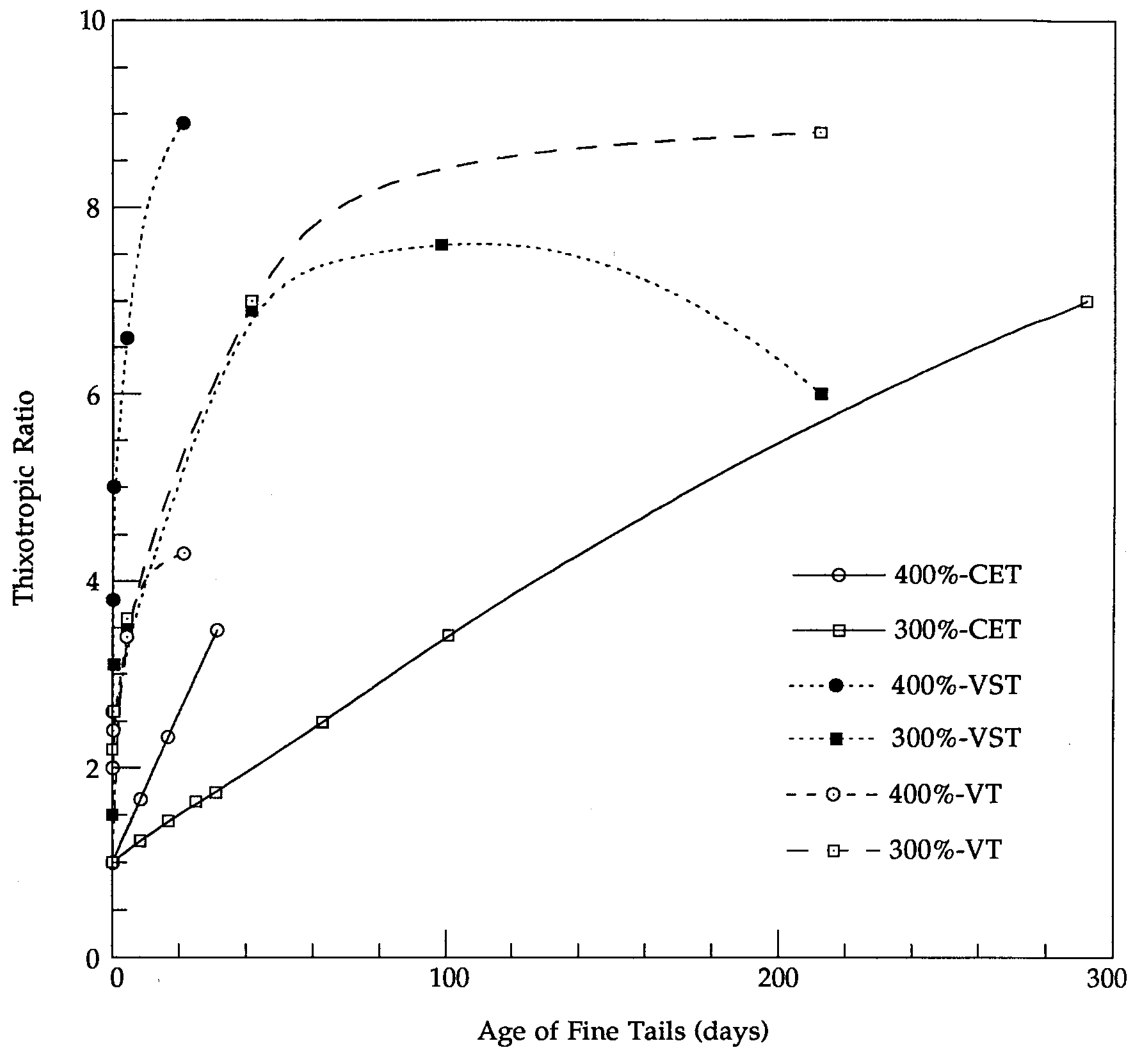

Figure 5.12 Thixotropic Ratio From Different Tests for Water Content Samples of 300% and 400%

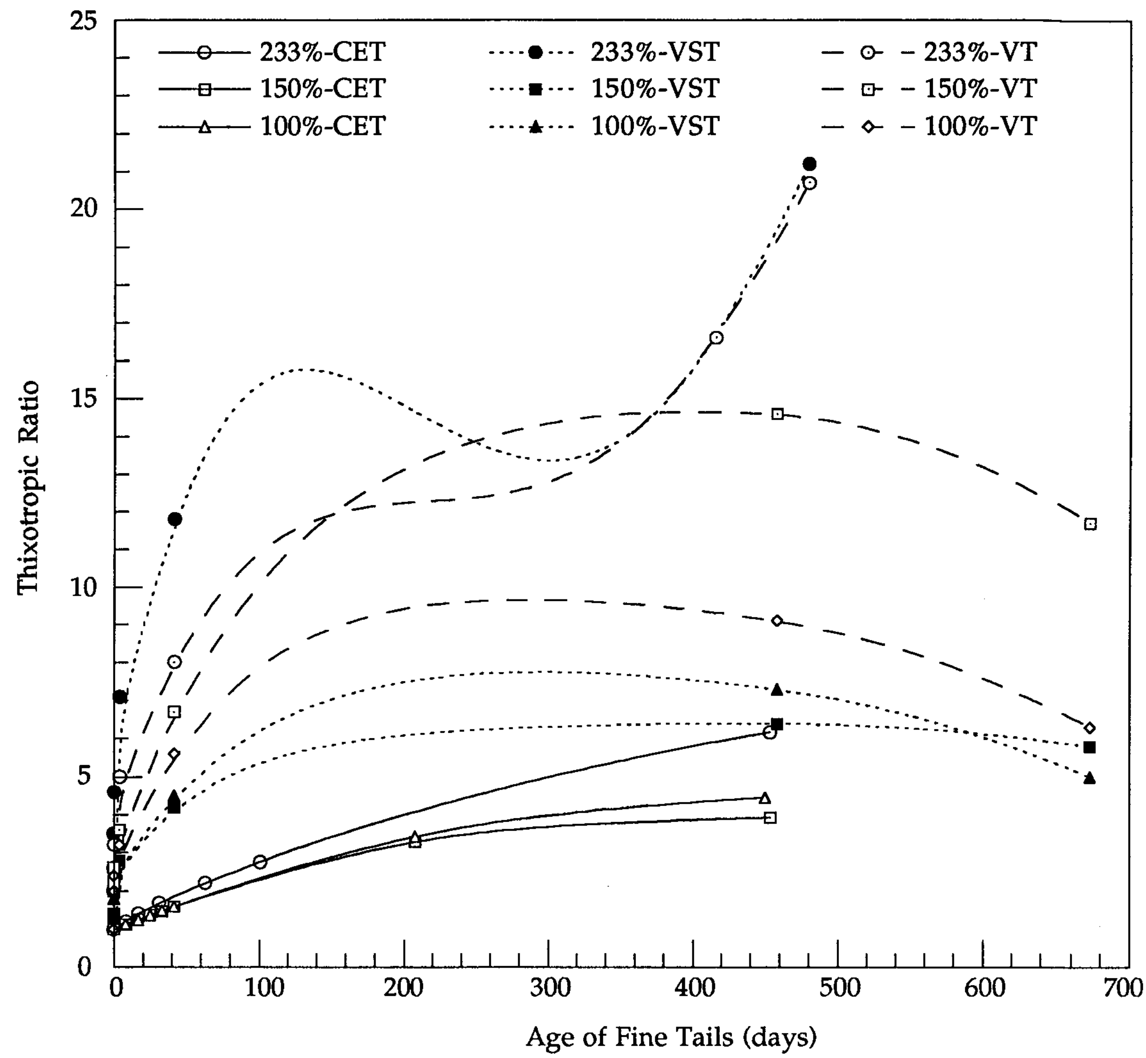

Figure 5.13 Thixotropic Ratio From Different Tests for Water Content Samples of 100%, 150% and 233%

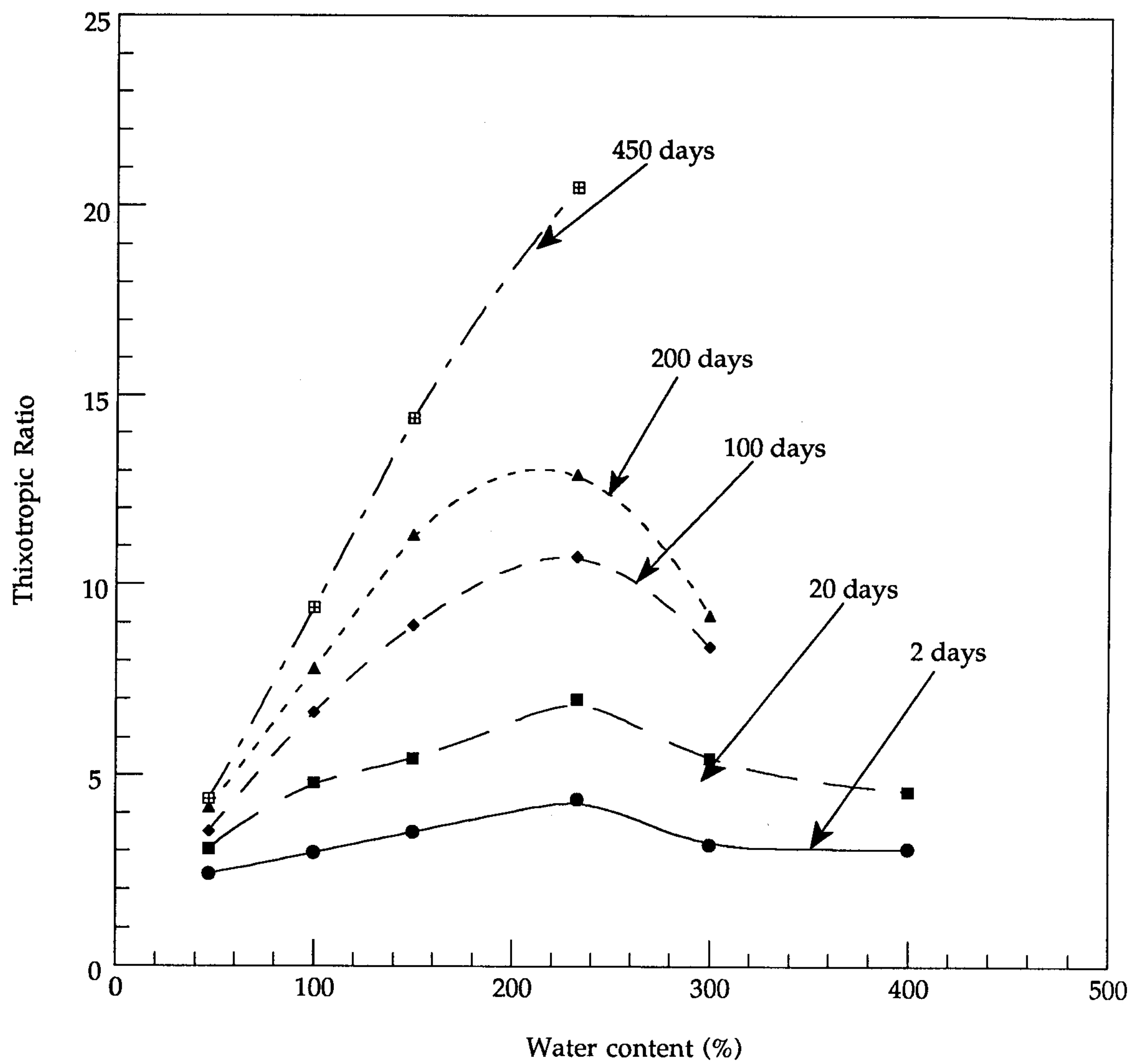

Figure 5.14 Thixotropic Ratio for Vane Shear Tests

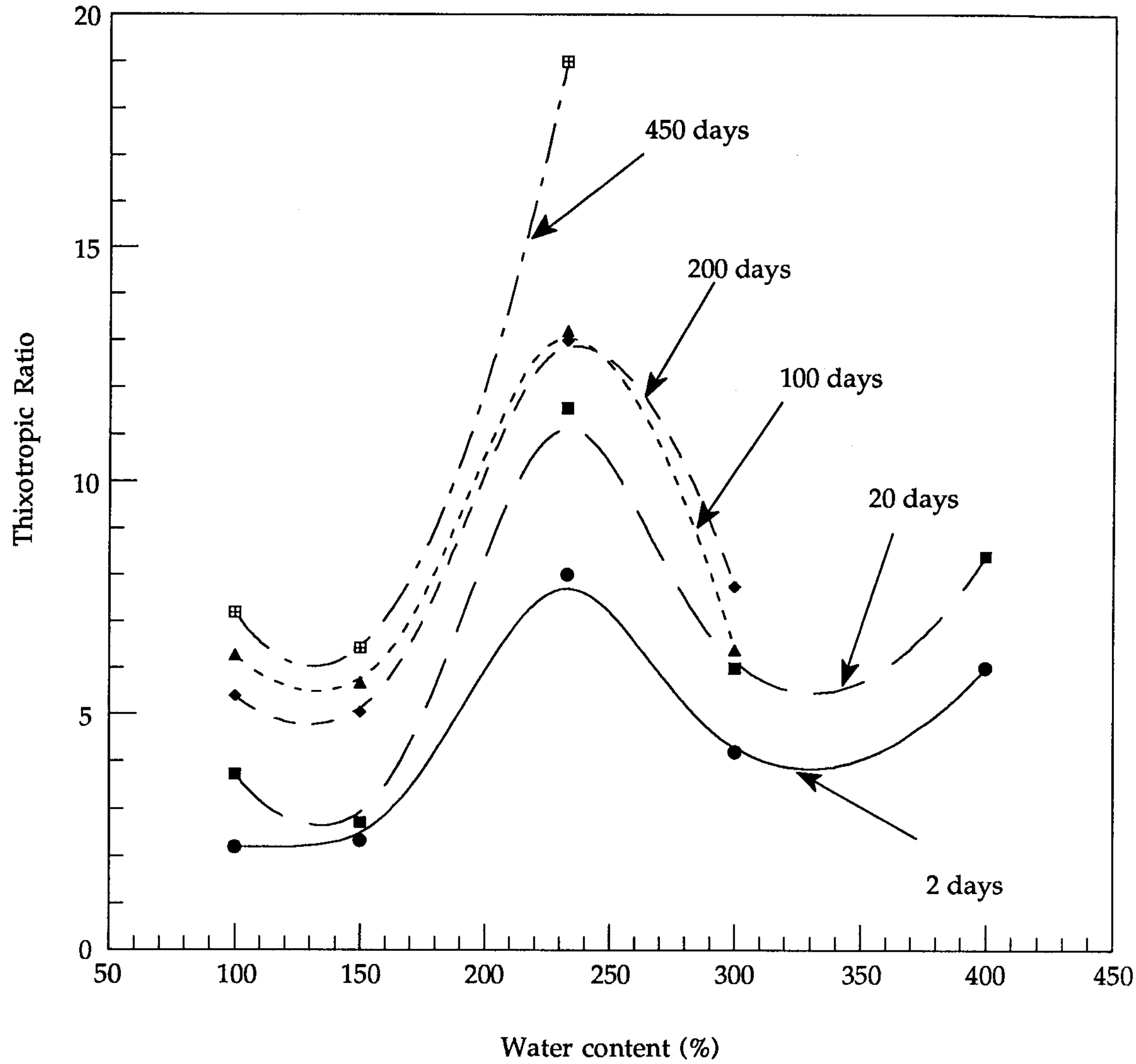

Figure 5.15 Thixotropic Ratio for Viscometer Tests

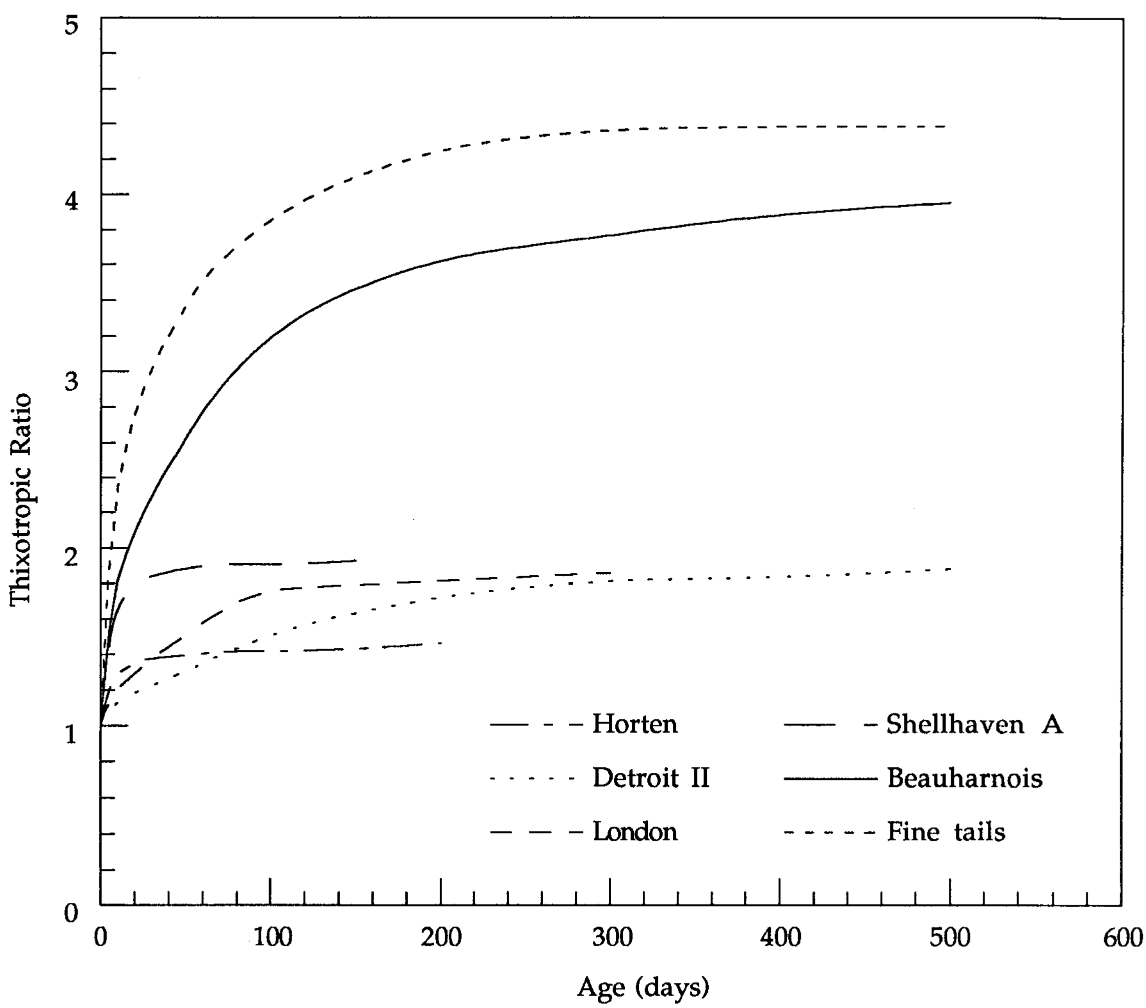

Figure 5.16 Comparison of Thixotropic Ratio of Different Clays at the Liquid Limit

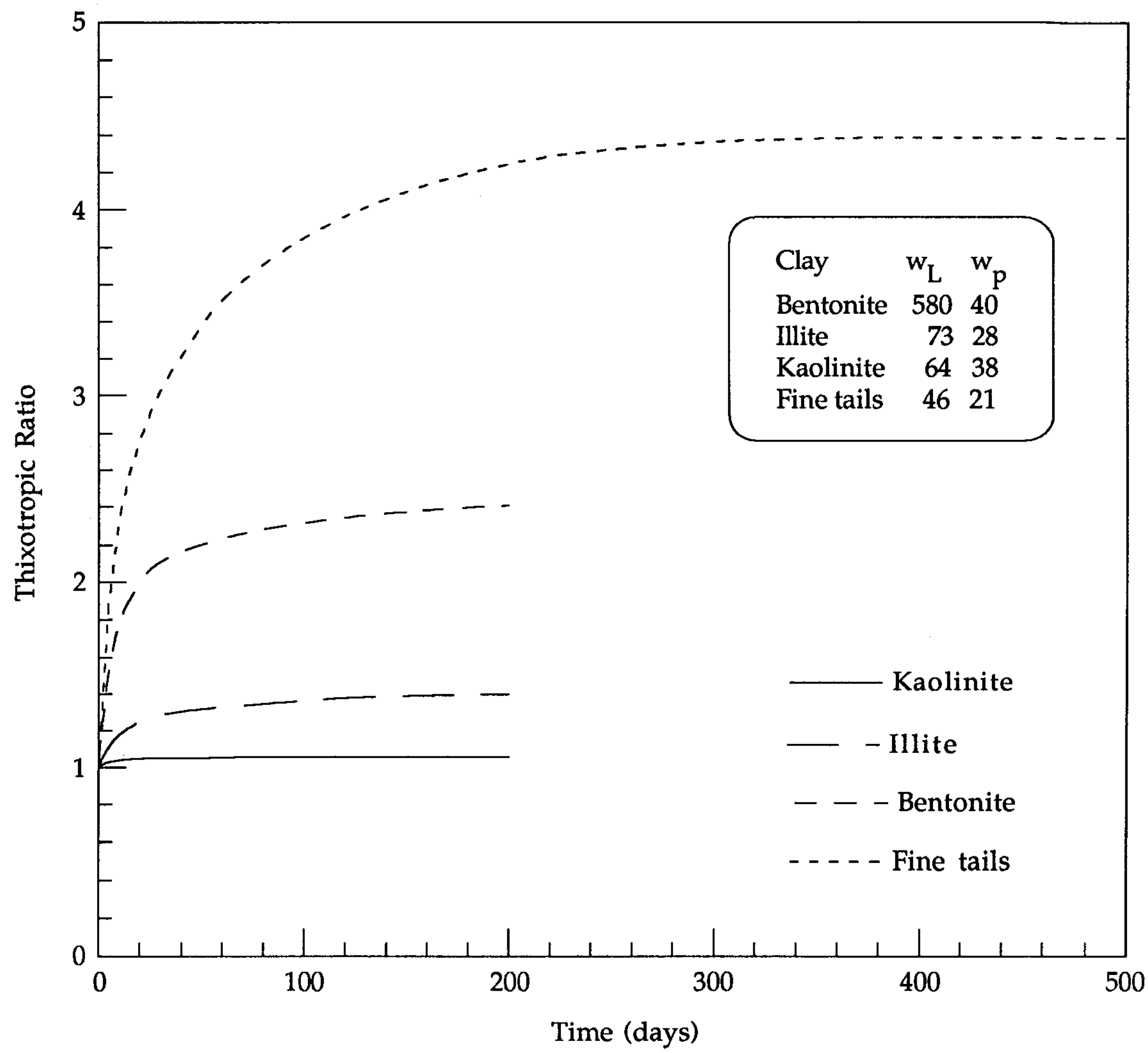

Figure 5.17 Comparison Of Thixotropic Ratio Of Different Minerals at the Liquid Limit

6. LARGE STRAIN CONSOLIDATION BEHAVIOR OF OIL SAND FINE TAILS[1]

6.1 Introduction

In northern Alberta, synthetic crude oil is produced from oil sand deposits by open pit mining in two oil sand plants, Syncrude Canada Ltd., and Suncor Inc. Syncrude and Suncor together produce about 650,000 tonnes of tailings daily which vary in solids content from 40% to 60%. The tailings stream is generally composed of about 85% sand and 15% fines by weight. The tailings sand and some fines form the pond dykes and beaches, which accumulate at a rate of 225,000 m^3 per day. Approximately one-half to two-thirds of the fines and most of the water flow into the pond to form a deposit which is called fine tails. Environmental concerns result in a zero discharge policy for the fine tails and it requires permanent storage. After sedimentation and some consolidation, the fine tails form a deposit at a void ratio of 6 to 7 which accumulates at a rate of 45,000 m^3 per day. Approximately 400,000,000 m^3 of this fine tails is presently held in the tailings ponds.

A wet landscape option is being considered as a long range disposal alternative for the oil sand fine tails by depositing them in the mined-out pits. The biologically poor quality fine tails will then be capped with a layer of water to preserve a natural habitat. Thus the reclaimed area would be a lake environment. The depth of water cap and the impact of fine tails on the quality of the water cap will depend on how much the fine tails are going to consolidate. Fine tails are expected to have an initial solids content of 30% in the wet landscape (Gulley and MacKinnon 1993). Predictions of the behavior of fine tails in the wet landscape disposal pond are important for the engineering design aspects and for environmental considerations.

The consolidation behavior of oil sand fine tails is controlled by its compressibility and permeability properties. Large strain consolidation tests can be used to establish the effective stress - void ratio and the permeability - void ratio relationships (Suthaker and Scott 1994a) which are needed to

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predict the field consolidation behavior. In general, the solids content of the fine tails varies with time and with the depth in the tailings pond. Therefore, a field prediction based on one specific initial solids content may not accurately represent the field conditions. Field permeability is greatly influenced by the grain size distribution and bitumen content of the fine tails, and by the hydraulic gradient in the tailings pond. As well, some deviations of the permeability relationships obtained from the laboratory results are possible depending on the test equipment and test procedures. When predicting the long term field consolidation behavior, the influence of the above deviations may play a substantial role and the predictions may vary accordingly.

This chapter reviews the slurry consolidometer testing program, which determined the properties of the fine tails, and the finite strain consolidation program used to predict the consolidation behavior of the fine tails. The test results of 2 m high and 10 m high self weight consolidation standpipe tests are presented and compared to their predicted behavior. The finite strain consolidation model is then used to evaluate the influence on field consolidation behavior of the variations in the laboratory measured compressibility and permeability properties. The importance of such variations in predicting the fine tails consolidation in the wet landscape disposal option is discussed.

6.2 Oil Sand Fine Tails

Oil sand fine tails is a mixture of clay and silt size particles with unrecovered bitumen. Grain size analyses of the fine tails show a composition of about 5% fine grained sand, 30% silt size and 65% clay size (Scott and Dusseault, 1982). The clay mineralogy of the fine tails reflects the average clay mineralogy of the McMurray formation in northern Alberta. Kaolinite and illite clays dominate, smectite is present as a minor ingredient. Vermiculite in small amounts and a trace of chlorite and mixed layer clays is also present (Scott et al., 1985). Dereniwski and Mimura (1993) summarized that the fine tails is dominantly kaolinite (55 - 65%) and illite (30 - 40%) with minute traces of mixed layer clay minerals.

The bitumen content of the fine tails based on the total mass of the fine tails averaged around 2 % (MacKinnon and Sethi, 1993). If the bitumen is calculated as a percent of the mass of the mineral solids, its content is 6.5%. The specific gravity of the fine tails varies between 2.1 and 2.5 due to the varying bitumen content in the fine tails since the specific gravity of the bitumen is 1.03. The average total unit weight of fine tails is about 12 kN/m^3.

Scott et al. (1985) reported that the liquid limit ranges from 40% to 80% and the plasticity index varies from less than 20% to slightly higher than 50%. The range reflects the effects of ionic concentration, clay mineralogy and bitumen content. The higher values in Atterberg limits in general indicate greater bitumen content and finer grained fine tails.

6.3 Laboratory Measurements of Compressibility and Permeability

Slurry consolidometer tests (Figure 6.1) have been used to obtain the compressibility and permeability characteristics of the fine tails samples (Suthaker and Scott 1994a). A step loading procedure, similar to a standard oedometer test, was employed because of its simplicity and the ability to perform permeability testing after the completion of consolidation under each load increment. Each load step required about 1.5 to 2 months to complete. A constant head technique was employed for the permeability measurements. Since a change in pore fluid chemistry may affect the permeability, tailings pond water was used as permeant to achieve consistent results with the field conditions. The total time required for a consolidation test was about 2 years. The void ratio - effective stress relationships at 20%, 25% and 30% initial solids contents (Figure 6.2) and the permeability - void ratio relationship (Case 1, Figure 6.3) were obtained from these tests.

6.4 Finite Strain Consolidation Program (FSCP)

The program used in this study uses the numerical model which was established by Somogyi (1980). This model of finite strain consolidation is based on the general non-linear equations developed by Gibson, England and Hussey (1967) with reformulations in terms of excess pore pressure. The theory is non-linear, includes self-weight of the soil, finite strains and places no restrictions on the initial void ratio. In Somogyi's model, an approximation that the void ratio - effective stress and permeability - void ratio relationships be restricted to the form of a power law, was introduced to handle the non-linearity. The model uses a fully implicit central finite difference method due to its stability. The FSCP, incorporating this model, was developed by Pollock (1988). This program also uses the consolidation parameters in the form of power law constants (A, B, C and D) as input data. Such relationships for compressibility and permeability are given by Equation 6.1 and Equation 6.2, respectively.

$$e = A \bullet \sigma'^{B} \tag{6.1}$$

where e is void ratio, σ' is effective stress in pascals, A and B are compressibility model constants.

$$k = C \bullet e^{D} \tag{6.2}$$

where k is hydraulic conductivity in m/s, C and D are permeability model constants.

The accuracy of the model then, will depend on the closeness of the power law relationships to the true fine tails behavior. The model does not incorporate the thixotropic behavior (Suthaker and Scott, 1995c) or the creep behavior (Suthaker and Scott, 1995a) of the fine tails. The influence of these properties on the fine tails consolidation behavior is evaluated by the analyses of the standpipe self weight consolidation tests.

6.5 Standpipe Tests

6.5.1 Two Metre High Standpipe Tests

Several self weight consolidation tests in 2m high standpipes have been conducted at the University of Alberta. The advantage of the 2m high standpipes is that they can be set up and performed with a reasonable cost, thereby allowing several to be tested. The consolidation in 2m high standpipes takes a relatively short time and the stress levels reached, although small, are significant. The 2m standpipe had pore pressure and sampling ports at 20 cm height intervals and the pore pressures were measured using manometers. Sampling was not done in the testing program because it might have disturbed the consolidation of the fine tails. Although seven different initial solids content fine tails were tested, only the results of the 32% initial solids test and 25% initial solids test have been reported here as they are typical. The properties of the fine tails materials tested in the standpipes and in the slurry consolidometers are shown in Table 6.1.

Figure 6.4 shows the measured and predicted settlement for different permeability cases in Standpipe A. The predicted settlement for Case 1 is larger than the measured settlement. This comparison indicates that the hydraulic conductivity of the material may be lower than the laboratory measured values. This was evaluated using Case 5 and Case 6 permeabilities. Case 6 shows excellent agreement with the measured settlements. At a void ratio of 5, the permeability for Case 1 is 8×10^{-8} m/s and that for Case 6 is 2×10^{-8} m/s, a difference of 400%. The measured and predicted excess pore pressure for Case 6 at 1000 days are shown in Figure 6.5 and agree reasonably well with a difference of about 20%. Figure 6.6 shows the measured and predicted solids contents after 300 days which are in good agreement.

Figure 6.7 shows the measured and predicted settlement for Standpipe 1. As found for Standpipe A, the Case 6 permeability shows the better prediction. The abnormal measured change in rate is probably due to gas being generated in the fine tails which held the interface up and then the interface settled as the gas escaped. Gas is generated from bacterial action in many fine tails samples in long term tests in the laboratory. Such gas generation is transitional and its effect varies. Figure 6.8 shows the measured and predicted excess pore pressures for Case 6 at 840 days and are in good agreement. Figure 6.9 shows the measured and predicted solids content after 840 days. The measured solids contents are fairly constant with depth, substantially different from the predicted values. Creep and thixotropy may be the cause for this difference.

The results of the two 2 m high standpipe tests indicate that the finite strain consolidation model using the consolidation test measurements does not model the fine tails consolidation very well. A permeability value (Case 6) four times smaller than the average laboratory measured permeability (Case 1) had to be used to obtain a fit. If the typical Case 1 material properties are used in the analyses, the rate of settlement, the rate of pore pressure dissipation and the increase in solids content are all over predicted.

6.5.2 Ten Metre High Standpipe Test

A self weight consolidation test on 32% initial solids content fine tails in a 10m high and 1m diameter standpipe has been conducted since 1982 (Suthaker and Scott, 1994b). The standpipe is equipped with pore pressure and sampling ports at 1m intervals (Figure 6.10). Additional ports at 0.5m intervals were used at the top and bottom of the standpipe where rapid changes in solids content and pore pressure took place. A sampling probe which has an O-ring seal was inserted to collect samples through each sample port. The samples were taken at different horizontal distances at each depth to minimize disturbance. For the high solids content fine tails which did not flow easily, a vacuum was applied to the sample bottle during sampling. Extraction and hydrometer tests were performed to determine the bitumen content and sand segregation in the fine tails samples in addition to solids content measurements.

The temperature of the fine tails was measured in the standpipe using a probe similar to the sampling probe. The variation in temperature in the standpipe has been constant throughout the test period with an increase of

$3^{\circ}C$ from the bottom to the top of the standpipe. This gradient is not large enough to have any significant effect on the consolidation behavior. Pore pressures were monitored by using a pressure transducer which was calibrated against a column of water before each set of readings. When the standpipe was first filled, the pore pressures were equal to the total stresses over the full height of the standpipe indicating zero effective stress. The water-fine tails interface, the settlement of which characterizes the amount of self weight consolidation, was continuously monitored.

The consolidation behavior of the fine tails in the 10 m high standpipe was modeled using the measured compressibility and permeability values from the 30% initial solids content fine tails consolidation test which is Case 5. For the fine tails in the 10m high standpipe, a material balance indicated the existence of gas; 0.69% by total volume. Therefore the specific gravity of the solids was lowered from 2.35 to 2.27 to reflect the gas content.

Figure 6.11 shows the measured and predicted settlement for the 10 m high standpipe. The measured settlement is slightly larger than the predicted settlement. This may be due to creep. The fine tails exhibit very high creep rates because of its high void ratio and high organic content (Suthaker and Scott, 1995a). Figure 6.12 shows the measured excess pore pressures at several elapsed times and the predicted excess pore pressures after 10 years. The predicted values show an excellent agreement with measured values. A large excess pore pressure remains in the standpipe even after 10 years of consolidation. The average pore pressure dissipation is only 17%.

Figure 6.13 shows measured and predicted solids contents after 10 years. Similar to the predicted excess pore pressure profile, the increase in predicted solids content began from the bottom, which is to be expected from normal consolidation behavior. However the data shows an increase in solids content down the entire depth of the fine tails with the greatest increase near the top of the fine tails. Therefore, the measured data indicate that consolidation is occurring at all depths with the highest solids content near the top. The high value near the top may be due to gas in voids, however, the fairly constant values with depth may be due to creep and thixotropy occurring in the standpipe which is not considered in the theory. These results are similar to those found in the 2 m standpipe 1 and indicate that the long term solids content distribution with depth can not be modeled without including the creep and thixotropy properties of the fine tails.

6.6 Wet Landscape

The fine tails can be reclaimed by maintaining the fine tails as a fluid and capping it with a layer of water at the top. This approach is analogous to most reclamation methods, where a poorer quality soil is capped with a better quality of soil capable of maintaining productive vegetation. In the wet landscape option, the biologically poorer quality fine tails would be capped with a layer of water, in which a viable ecosystem would evolve (Gulley and MacKinnon, 1993). The fine tails would be transferred from the tailings pond to a mined-out pit (Figure 6.14). In-pit containment will be below the original ground surface with a depth of more than 40m of fine tails and with surface areas from 2 km^2 to 10 km^2. The capping water layer would be about 5m thick and should be prevented from mixing significantly with the fine tails materials because such mixing could be detrimental to biological development within the water layer. The depth and the quality of the capping layer depend on the amount and rate of self weight consolidation of the fine tails.

The results from the slurry consolidation tests together with the FSCP were used to predict the consolidation behavior of the fine tails in the wet landscape. In general, settlement, excess pore pressure and solids content were considered in the analysis. The effects of the filling rate, bottom seepage and compressibility and permeability properties were investigated.

6.6.1 Effect of Filling Rate

This analysis was performed to determine the effect of the rate of filling of the disposal pit on the long term settlement of the fine tails. The filling rates chosen were 1 m per year for 50 years, 2 m per year for 25 years, 5 m per year for 10 years, 10 m per year for 5 years and 50 m instantaneously. The 30%

initial solids content compressibility and the average permeability characteristics (Case 1, Figure 6.3) were used in the analysis.

Figure 6.15 shows the height of fine tails with time. Although, during the filling period, the height of the fine tails are different, the long term settlements after the filling period are the same. Therefore analyses show that the filling rate does not have any influence on the long term consolidation, therefore further analyses have been performed for only 50 m of fine tails deposited instantaneously.

6.6.2 Effect of Seepage through Bottom of the Pond

The provision of drainage can dictate the consolidation behavior. If the wet landscape bottom is impermeable, it can be characterized as single upward drainage. However, if a permeable layer is provided at the bottom of the fine tails, a double drainage situation is created. Analyses were performed for both the single and double drainage conditions to determine their effects on the field consolidation behavior.

Figure 6.16 shows the settlement of the fine tails with time. The difference between single and double drainage is small in terms of the settlement. This small difference is further supported by the excess pore pressure dissipation with time (Figure 6.17). The bottom of the fine tails affected by the downward drainage is always less than one tenth of the original height. This small influence is due to a filter cake formation at the bottom of the fine tails, which significantly reduces the permeability and hinders the drainage. The small effects of double drainage suggest that the provision of a geosynthetic drainage layer or a sand layer at the bottom of the fine tails will not significantly improve the rate and amount of consolidation of the fine tails.

6.6.3 Effect of Compressibility and Permeability

Different compressibility characteristics were exhibited by fine tails with different initial solids contents (Figure 6.2). This behavior may be due to the gain in thixotropic strength with time. The wet landscape option will have an initial solids content of 30% which was considered as a base case for compressibility. However, the 30% solids in the wet landscape may have a different compressibility than that obtained in the laboratory, due to a different stress history. The effect of a variation in compressibility was studied by varying the compressibility according to the laboratory measured compressibility values for 30%, 25 % and 20% initial solids.

Hydraulic conductivity varies with bitumen content, grain size distribution, and hydraulic gradient. Hydraulic conductivity is not influenced by initial solids content. However, the field permeability may not be the same as that obtained from the laboratory data due to variations of bitumen content, grain size distribution, and hydraulic gradient in the field. Such deviations of permeability data input were simulated by a change in model constant D (equation 6.2). The variations in the permeability characteristics are shown in Figure 6.3 (Cases 1, 4, 5 and 6). Case 1 is the average laboratory measured hydraulic conductivity for hydraulic gradients less than 0.2 and for fine tails having about 50% grain sizes finer than 2 microns and 2% coarser than 45 microns (sand boundary), and a bitumen content of about 3% by total mass (Suthaker and Scott, 1995b).

Table 2 summarizes the input parameter variations of the compressibility and permeability relationships which were used to predict the field behavior. The analyses did not include the increase in sand content that accumulates in the fine tails due to wind action blowing sand in to the pond. Analysis in Case 1 (base case) characterizes the wet landscape behavior when the field compressibility and permeability are the average values obtained in the laboratory tests. Cases 2 and 3 represent the variations of the fine tails compressibilities. Case 4 and Case 5 are upper and lower boundaries of the laboratory measured permeabilities and Case 6 is a conservative estimate of the permeability.

6.6.4 Settlement Predictions

The variations of predicted settlement with time for different compressibilities are shown in Figure 6.18. In general, a relatively rapid rate of settlement (0.7 m/yr) was observed for about 30 years which was then followed by a slow rate of settlement diminishing towards an ultimate stage. The different compressibility relationships did not affect the predicted settlement during the period of rapid settlement and produced only a little deviation thereafter (Figure 6.18). Long term settlement amounts were only slightly less for lower compressibilities. Therefore, it can be concluded that if the compressibility of the fine tails in the wet landscape disposal pond is different from that was used in the model, the settlement may not substantially vary from the prediction.

The variation in the permeability relationship, however, had a strong impact on the predicted settlement, showing larger and faster predicted settlements with increased hydraulic conductivity (Figure 6.19). Therefore, an overestimation of settlement rate will arise from using a hydraulic conductivity larger than that of the fine tails in the wet landscape disposal pond. The ultimate settlement, 33m approximately, is not affected by the hydraulic conductivity.

6.6.5 Predicted Excess Pore Pressure

Pore pressure dissipation is an indication of strength development. The effect of compressibility on excess pore pressure is shown in Figure 6.20. In general, the deviation from the base case varied only slightly at the bottom of the pond after 50 years. Therefore the change in the compressibility characteristics did not affect the short-term predictions, while the long-term predictions changed slightly with depth. In contrast, variations in the permeability influenced the predicted pore pressures substantially (Figure 6.21). Long term predictions, especially, were considerably affected by changes in permeability. Excess pore pressure change with time at the bottom of the fine tails was less compared to that which occurred at the top. The greater reduction in the pore pressure at the top can be attributed to the settlement of the interface. There would be even higher pore pressures due to the increase in density of the fine tails if all the sand in the fine tails in the actual tailings ponds was included in the calculations.

6.6.6 Solids Content Variation

The effect of compressibility (Figure 6.22) and permeability (Figure 6.23) relationships on solids content with time were similar to that observed for the excess pore pressures. The variation in hydraulic conductivity again has a greater influence than compressibility. The solid contents after 200 years varied from 40% to 60% at the top to 60% to 70% at the bottom in all cases. These solids contents may be increased by about 5% to 10% due to the sand that is in the fine tails. However, as stated above, the increase in solids content or density from the sand only results in higher pore pressures not in an increase in strength or stability of the fine tails.

6.6.7 Water Release During Consolidation

The amount and rate of water released during consolidation is very important as it dictates the concentrations of harmful substances flowing in to the water cap. Figure 6.24 shows the water released per year for different surface areas with time. Water released per year started declining after approximately 20 years. A major portion of the water release occurs within 50 years after which the concerns of further contamination could be considerably reduced. The amount of water released during consolidation has to be used together with a water balance equation to establish the volume of the water capping layer with time.

6.6.8 Comparison with Other Estimates

Figure 6.25 shows the above analysis of the fine tails behavior in the wet landscape compared with analyses based on field measurements in the present tailings ponds. The tailings ponds are filled at a rate of 2 kg of solids /

m^2/day for the Syncrude tailings pond and 1.5 kg/m^2/day for the Suncor tailings pond. The tailings pond predictions were made using the measured fine tails behavior for the past 10 years in the Syncrude tailings pond and in the Suncor tailings pond by MacKinnon and Sethi (1993). They used the particle size distribution with depth and time in the tailings ponds to empirically predict the rate of increase in average solids content for the next 200 years. They used this empirical method as they considered that no theoretical model was available for prediction of fine tails behavior. The rates shown are for the < 44 μm fine tails in the tailings ponds. The average solids content shown is some what misleading in the sense that the solids content in the top of the deposit will be much lower.

The wet landscape prediction is made using the finite strain consolidation theory for instantaneous filling of 50 m with the laboratory measured compressibility and permeability values (Case 1), the lower bound of the laboratory measured values (Case 5) and a conservative permeability value (Case 6). Modeling of the standpipe tests showed that Case 6 material parameters were necessary to model the 2 m high standpipes and that Case 5 material parameters were necessary to model the 10 m high standpipe. The Case 1 predictions and the empirical predictions overestimate the rate of consolidation.

It is concluded that the permeability values for fine tails measured in the laboratory oedometer tests do not represent the permeability of the fine tails in the 10 m and 2 m standpipes. It has been shown that the structure of the fines in nonsegregating tailings mixes has a major influence on the permeability of the tailings (Suthaker and Scott, 1995b). Similarly, the gel structure of the fine tails in the different laboratory tests and in the field would have a controlling influence on the permeability. As the rate and amount of compression of the fine tails increases, the gel structure is broken down. The strain rate in the oedometer tests is considerably more than the strain in the 2 m and 10 m standpipes which results in less gel structure and higher permeabilities in the oedometer tests. For this reason the Case 1 permeability cannot model the standpipe tests.

As the tailings pond field strain rate in the fine tails is considerably less than the laboratory rates in the oedometer and standpipe tests, the greater gel structure in the field may result in considerably lower field permeabilities. All the predictions in Figure 6.25 therefore may be overestimating the rate of fine tails consolidation.

6.7 Conclusions

From this study, the following conclusions can be made.

1. The results of the two 2 m high standpipe tests indicate that the finite strain consolidation model using the consolidation test measurements does not model the fine tails consolidation very well. A permeability value (Case 6) four times smaller than the average laboratory measured permeability (Case 1) had to be used to obtain a fit. If the typical Case 1 material properties are used in the analyses, the rate of settlement, the rate of pore pressure dissipation and the increase in solids content are all over predicted.

2. For the 10m high standpipe, the measured settlements are slightly larger than the predicted settlement. The predicted and measured pore pressures agree extremely well except at the bottom which may be due to a segregated sand layer at the bottom. The solids content with depth, however, was poorly predicted possibly because of the thixotropic and creep behavior of fine tails. This indicates that the long term solids content distribution with depth can not be modeled without including creep and thixotropy of the fine tails in the model.

3. The rate of filling the disposal pit in the wet landscape option does not have any influence on the long term consolidation behavior after the filling period.

4. Providing drainage at the bottom of the fine tails did not considerably improve the consolidation. Therefore provision of a drainage layer in the wet landscape may not be viable as it does not accelerate the consolidation significantly.

5. Variations in the compressibility relationships had a small effect on the wet landscape predictions. Therefore, the model prediction will remain reasonably accurate even if the fine tails compressibility in the tailings pond is different from that measured in the laboratory.

6. Small variations in the permeability relationships can cause severe deviations from accurate predictions, especially during long term consolidation. Accurate characterization of field permeability is essential to predict the fine tails consolidation behavior in the wet landscape disposal option. Using samples which represent the field material and with proper measurement techniques, reasonable estimates can be made. It would appear, however, that field measurements of permeability are essential.

7. A relatively rapid rate of settlement is expected in the wet landscape disposal pond for the first 30 years which will then be followed by a slow rate of settlement diminishing towards an ultimate settlement of 33 metres in about 1000 years. The major portion of water will be released from the wet landscape within the first 50 years, after which the potential for contamination is greatly reduced.

8. Modeling of the standpipe tests showed that Case 6 material parameters were necessary to model the 2 m high standpipes and that Case 5 material parameters were necessary to model the 10 m high standpipe. The strain rate in the oedometer tests is considerably more than the strain in the 2 m and 10 m standpipes which results in less gel structure and higher permeabilities in the oedometer tests. For this reason the Case 1 permeability cannot model the standpipe tests.

9. As the tailings pond field strain rate in the fine tails is considerably less than the laboratory rates in the oedometer and standpipe tests, the greater gel structure in the field may result in considerably lower field permeabilities. All the predictions in Figure 6.25 therefore may be overestimating the rate of fine tails consolidation.

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Table 6.1 Summary of material properties

Properties	10m Standpipe	2m Standpipe A	2m Standpipe 1	2m Standpipe 30	Consolidation Tests (Initial Solids) 25	Consolidation Tests (Initial Solids) 20
Initial Solids (%)	32.4	31.9	25.6	29.1	25.7	21.47
Fines Content (%)	89	90	96	92	100	100
Bitumen Content (%)	8.6	4.6	3	7.1	2.67	3.1
Specific Gravity	2.35	2.27	2.3	2.39	2.57	2.58
Unit Weight (kN/m3)	12.2	12.13	11.26	12.04	11.57	11.33

Table 6.2 Summary of Input Data for FSCP Predictions

	Compressibility Model			Permeability Model		
Case	Compressibility	A	B	Permeability	C	D
1	30 % solids	29.04	-0.3118	Lab. measured	6.16e-11	4.468
2	25 % solids	40.29	-0.3169	Lab. measured	6.16e-11	4.468
3	20 % solids	54.12	-0.3224	Lab. measured	6.16e-11	4.468
4	30 % solids	29.04	-0.3118	Upper limit	6.16e-11	5.361
5	30 % solids	29.04	-0.3118	Lower limit	6.16e-11	4.021
6	30 % solids	29.04	-0.3118	Conservative	6.16e-11	3.574

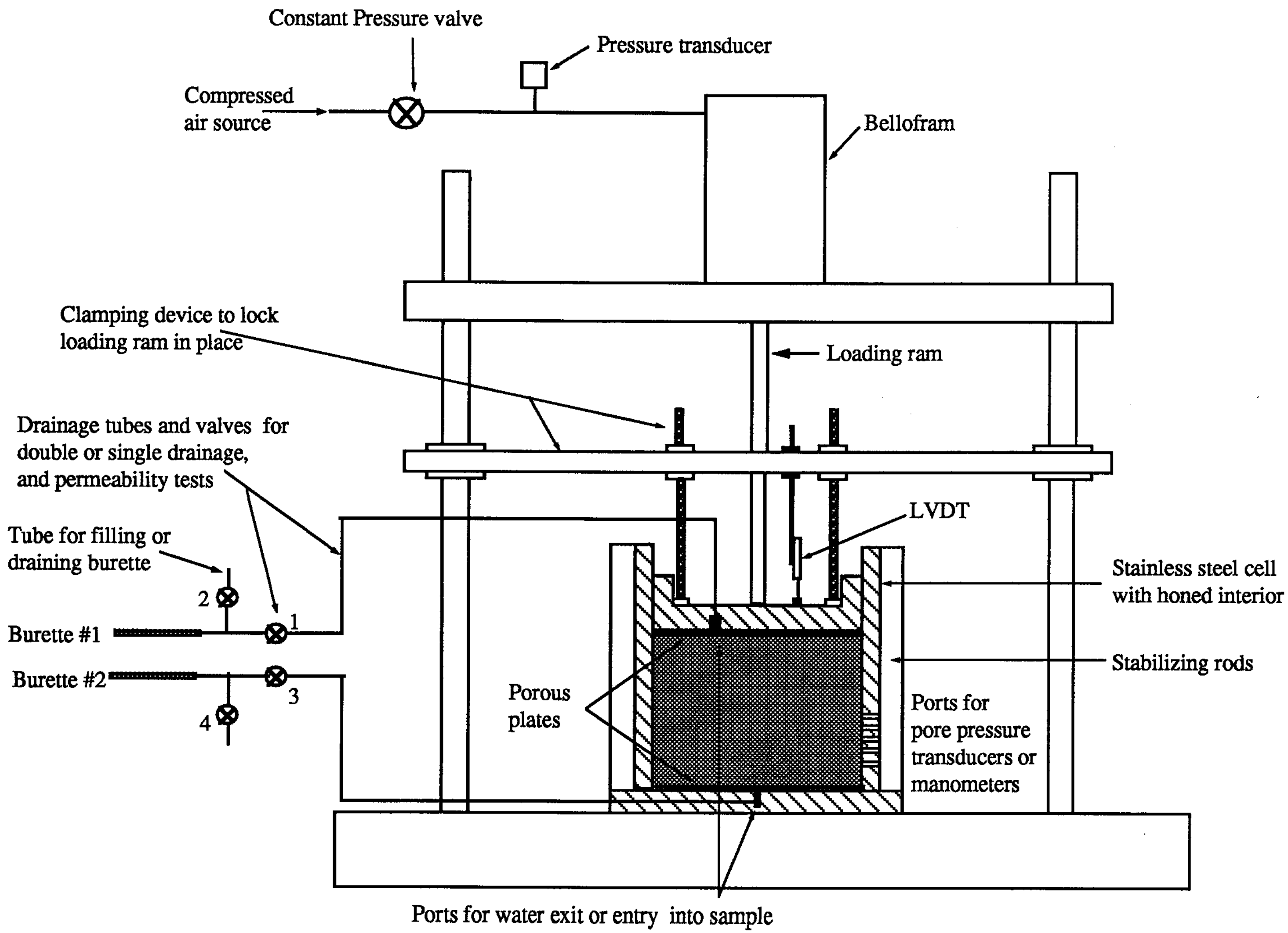

Figure 6.1 Slurry Consolidometer

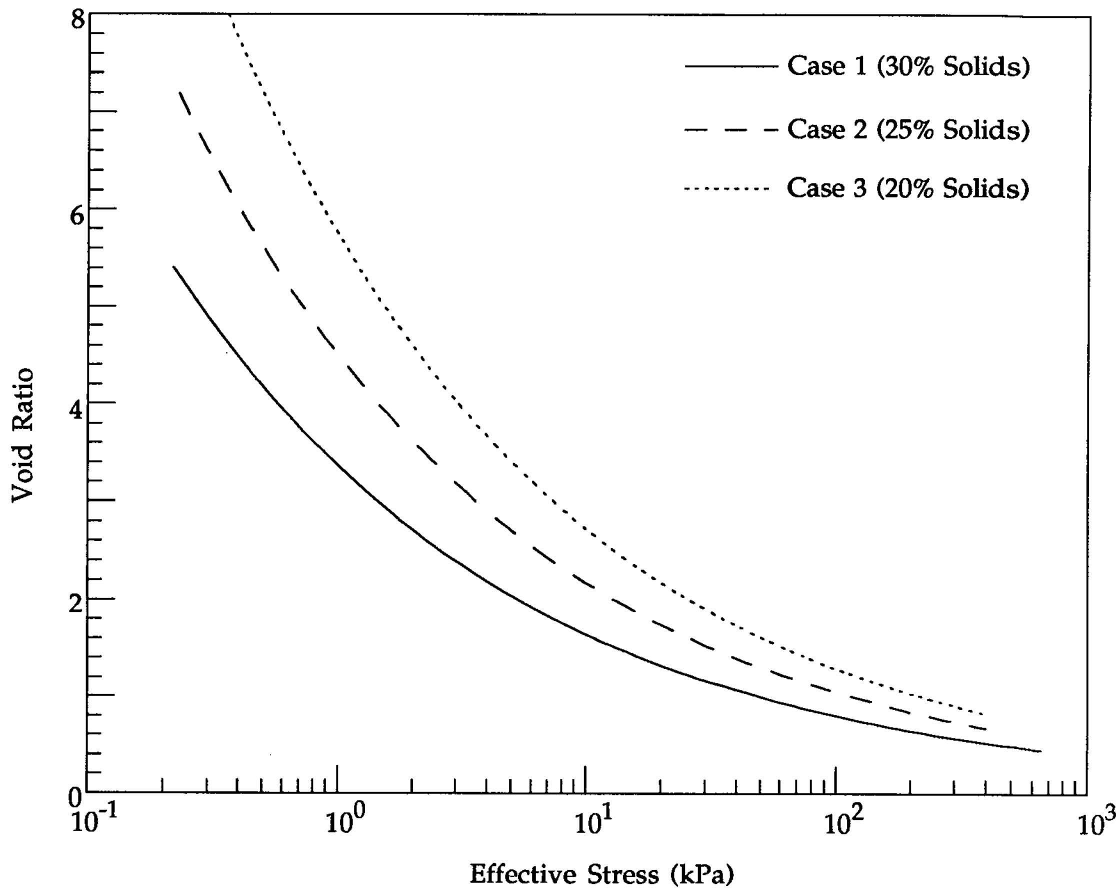

Figure 6.2 Compressibility Characteristics of Oil Sand Fine Tails

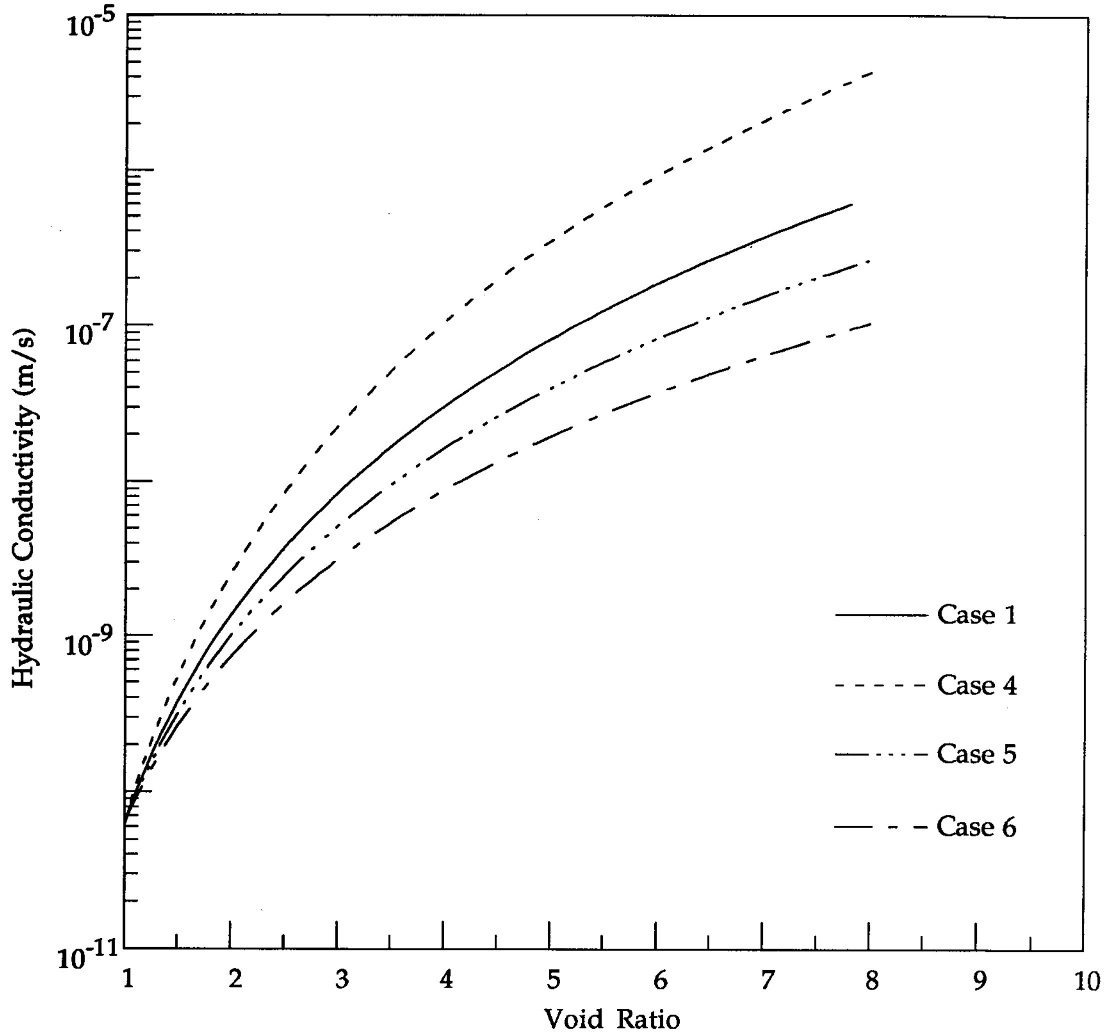

Figure 6.3 Permeability Characteristics of Oil Sand Fine Tails

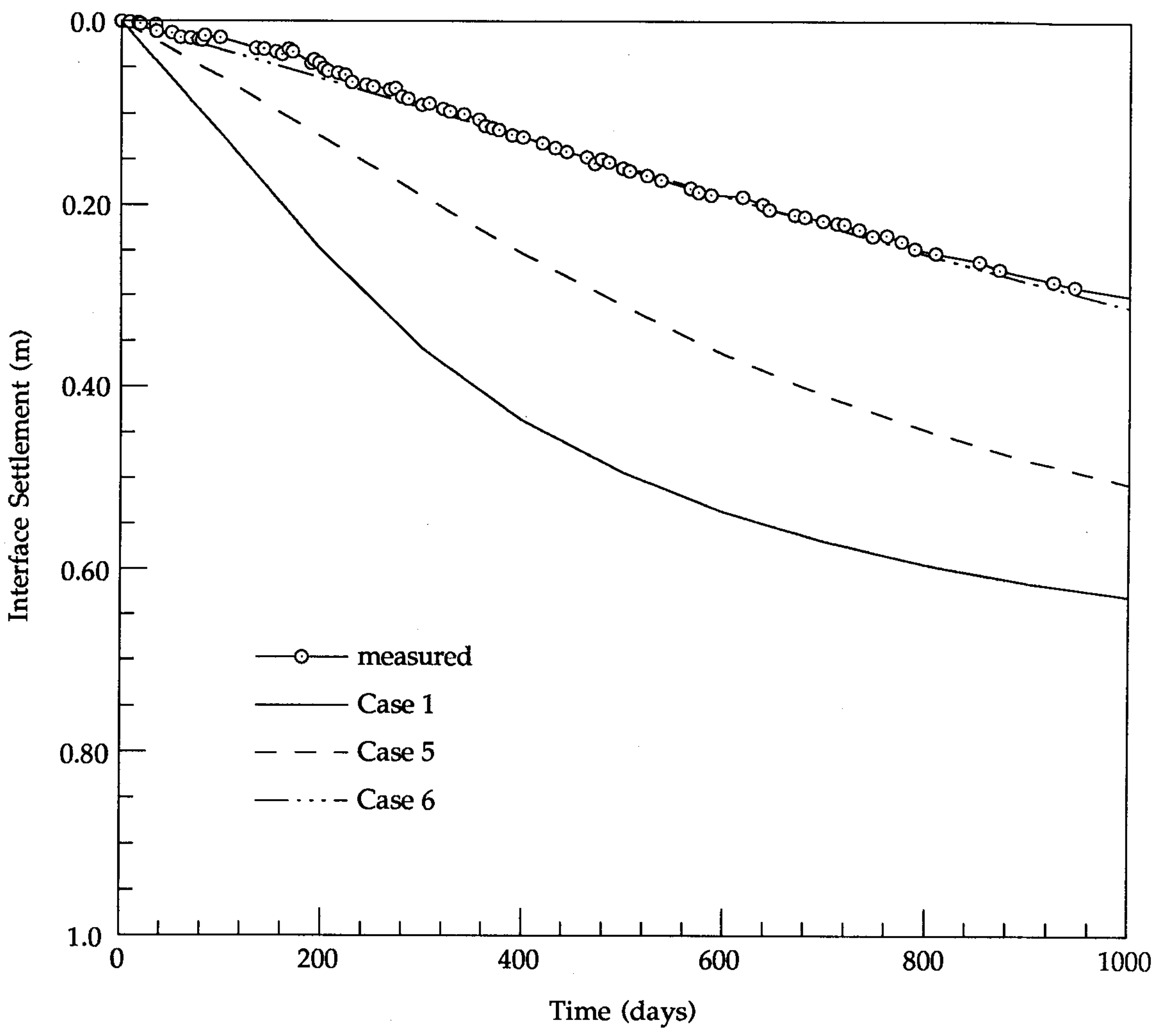

Figure 6.4 Measured and Predicted Settlement in Two Metre Standpipe A

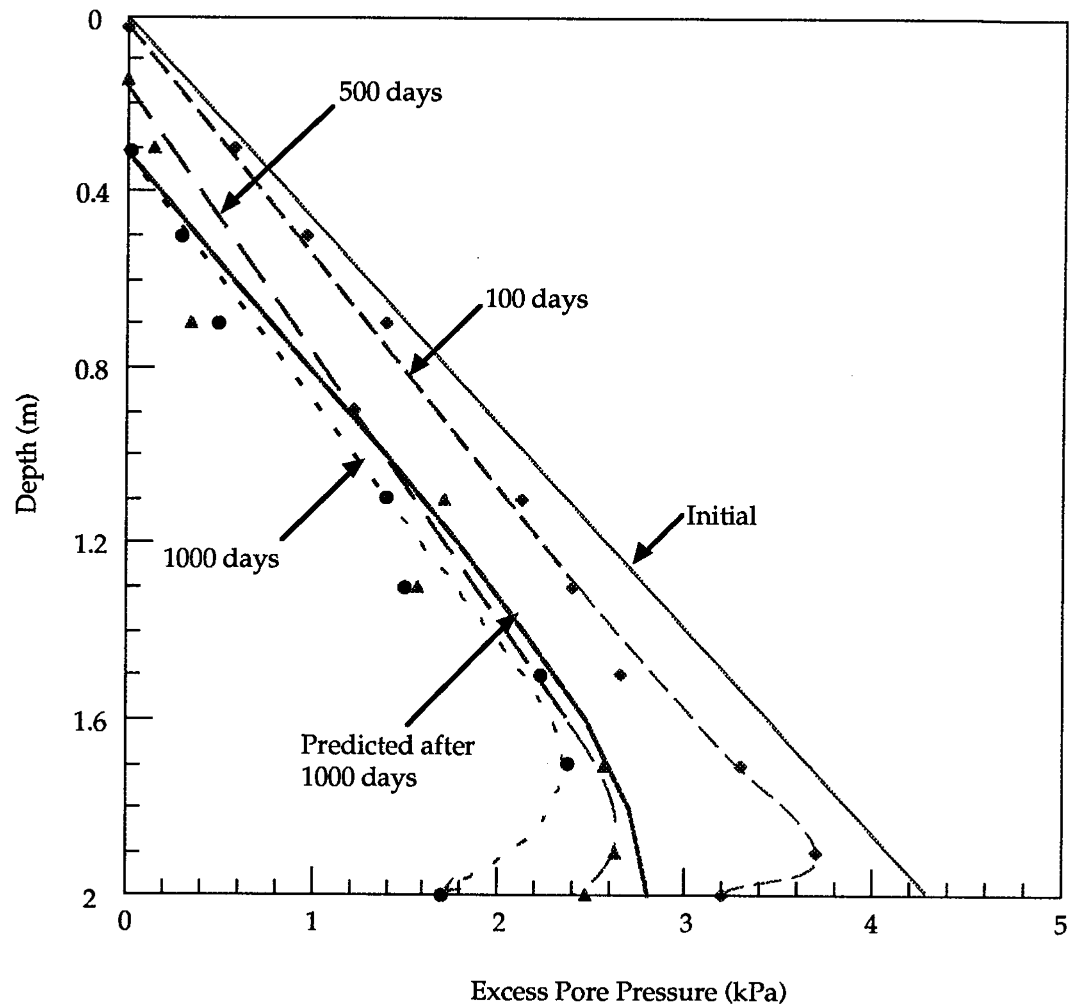

Figure 6.5 Measured and Predicted Excess Pore Pressures in Two Metre Standpipe A

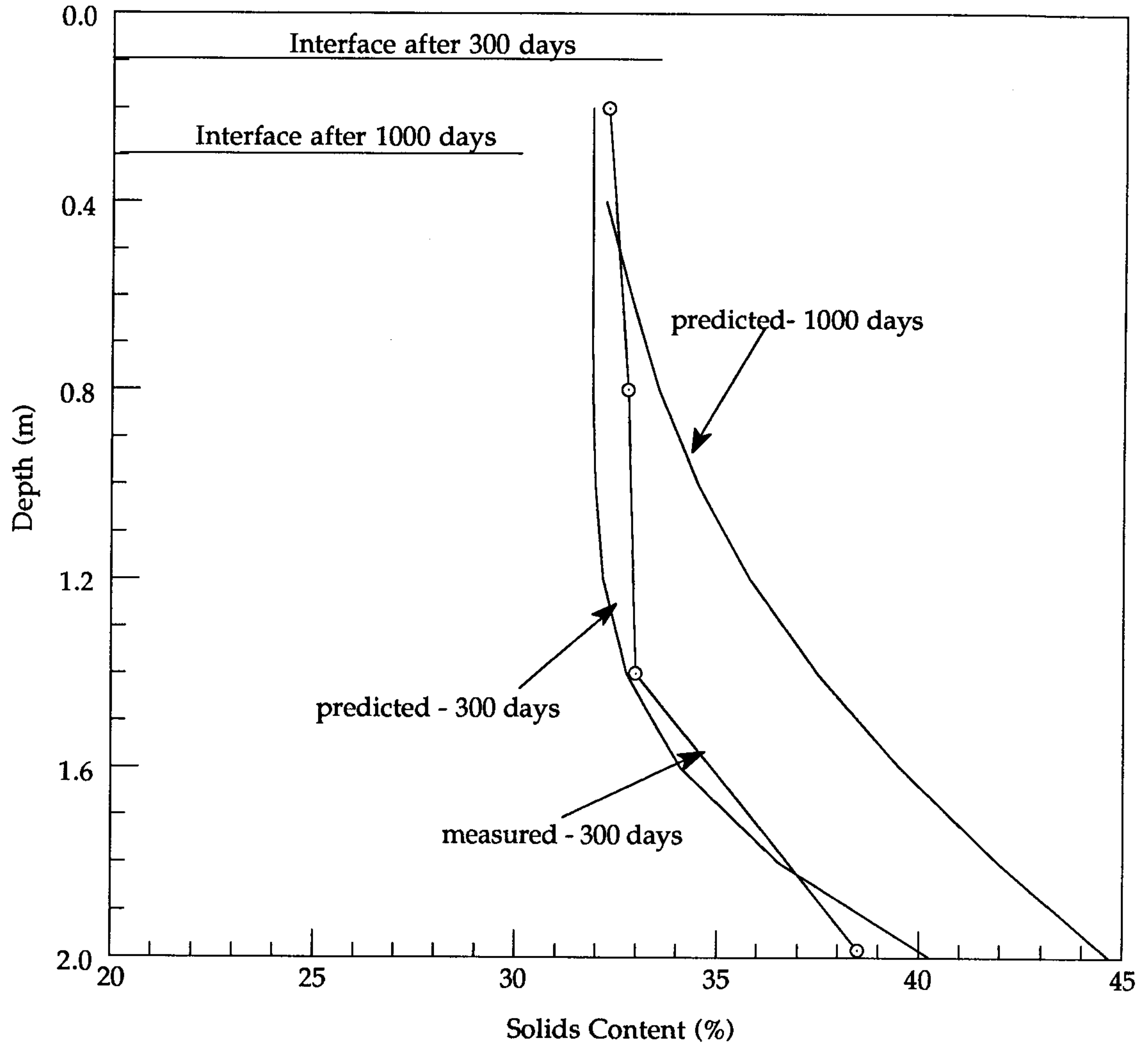

Figure 6.6 Measured and Predicted Solids Content in Two Metre Standpipe A

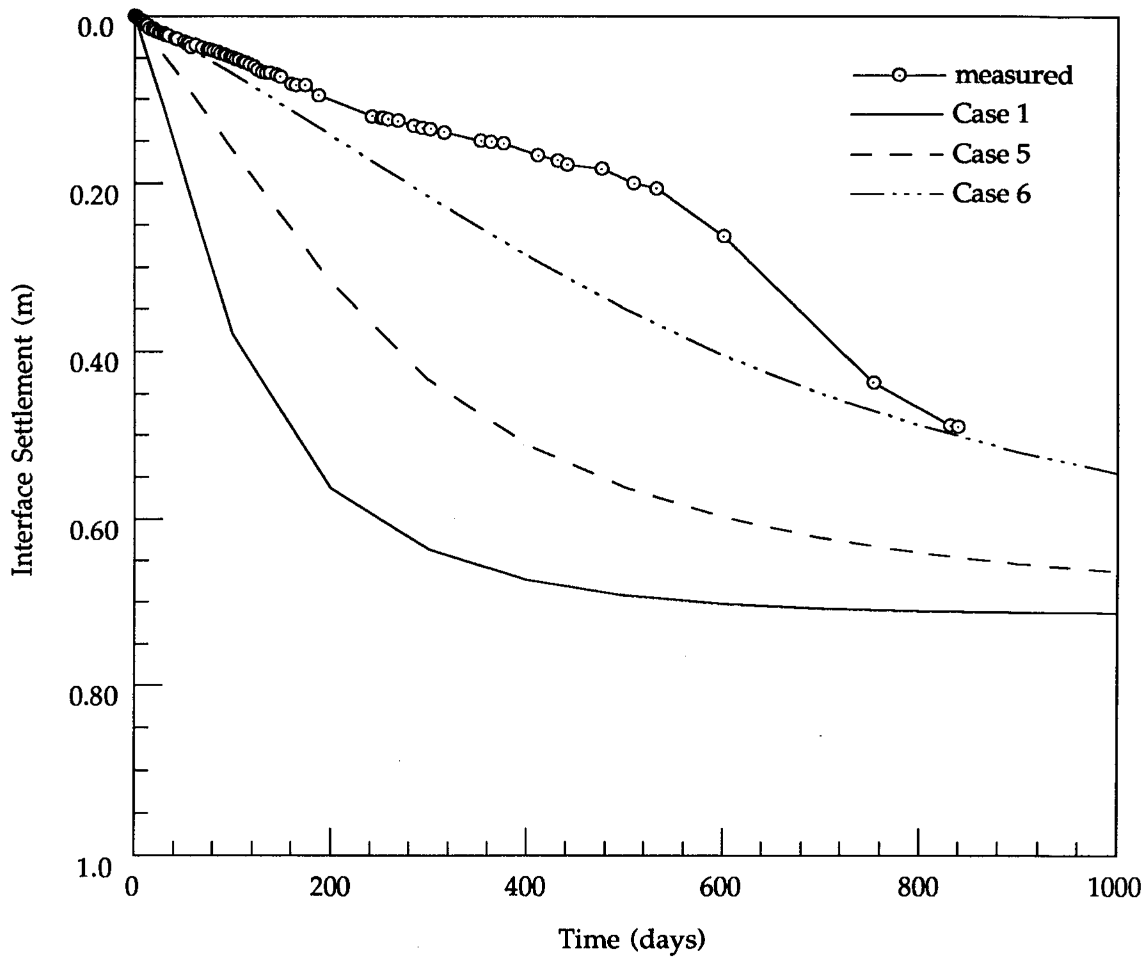

Figure 6.7 Measured and Predicted Settlement in Two Metre Standpipe 1

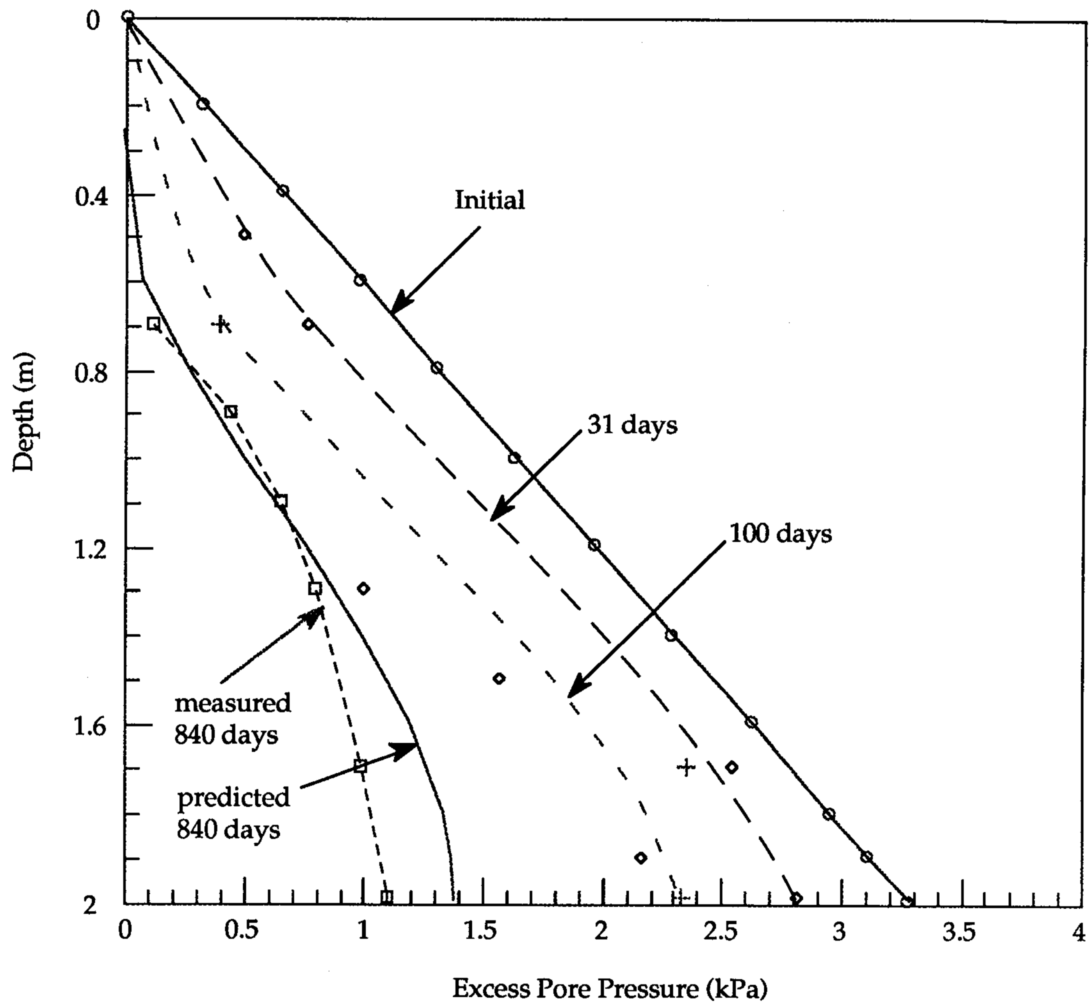

Figure 6.8 Measured and Predicted Excess Pore Pressures in Two Metre Standpipe 1

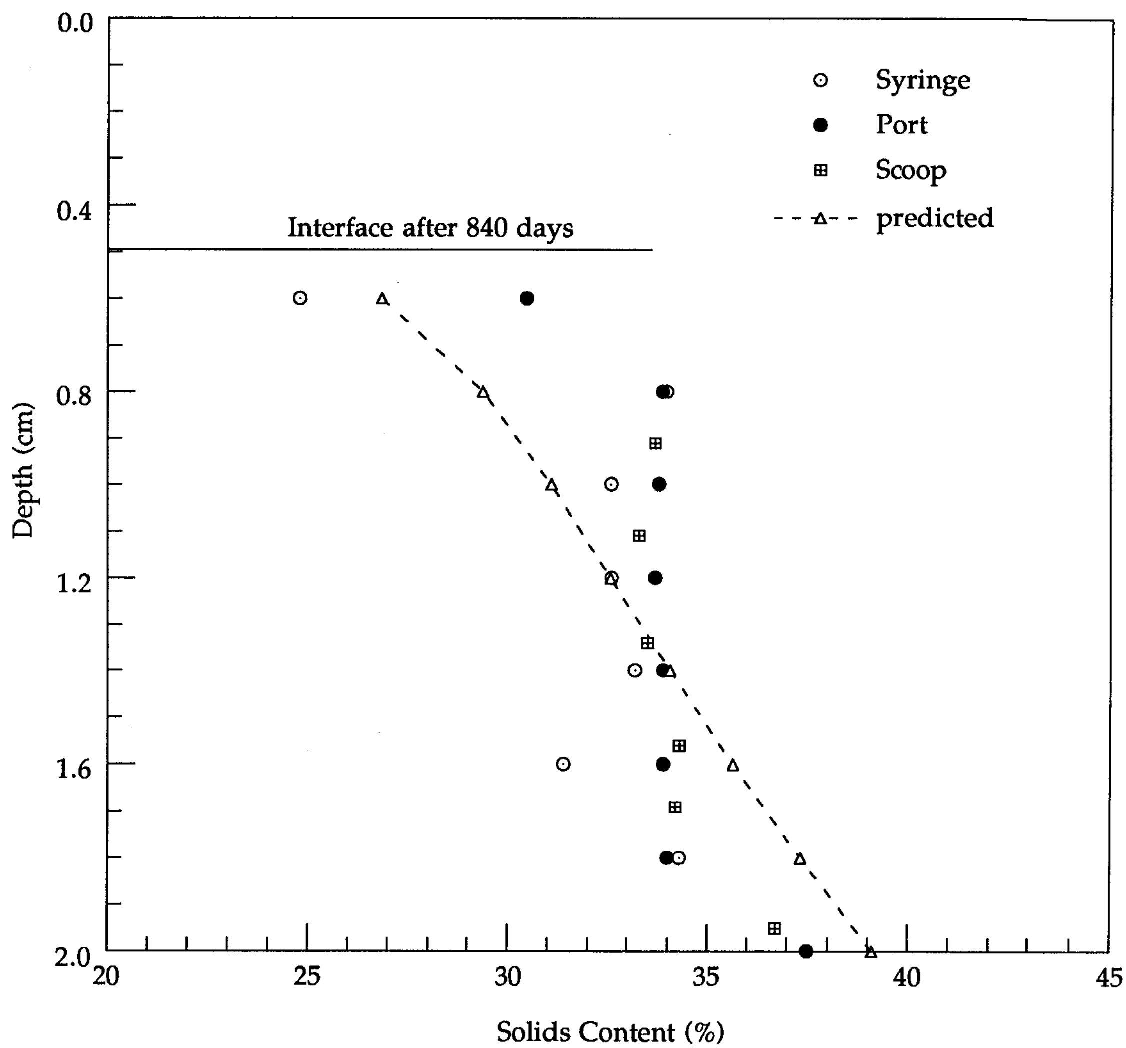

Figure 6.9 Measured and Predicted Solids Content in Two Metre Standpipe 1 after 840 Days

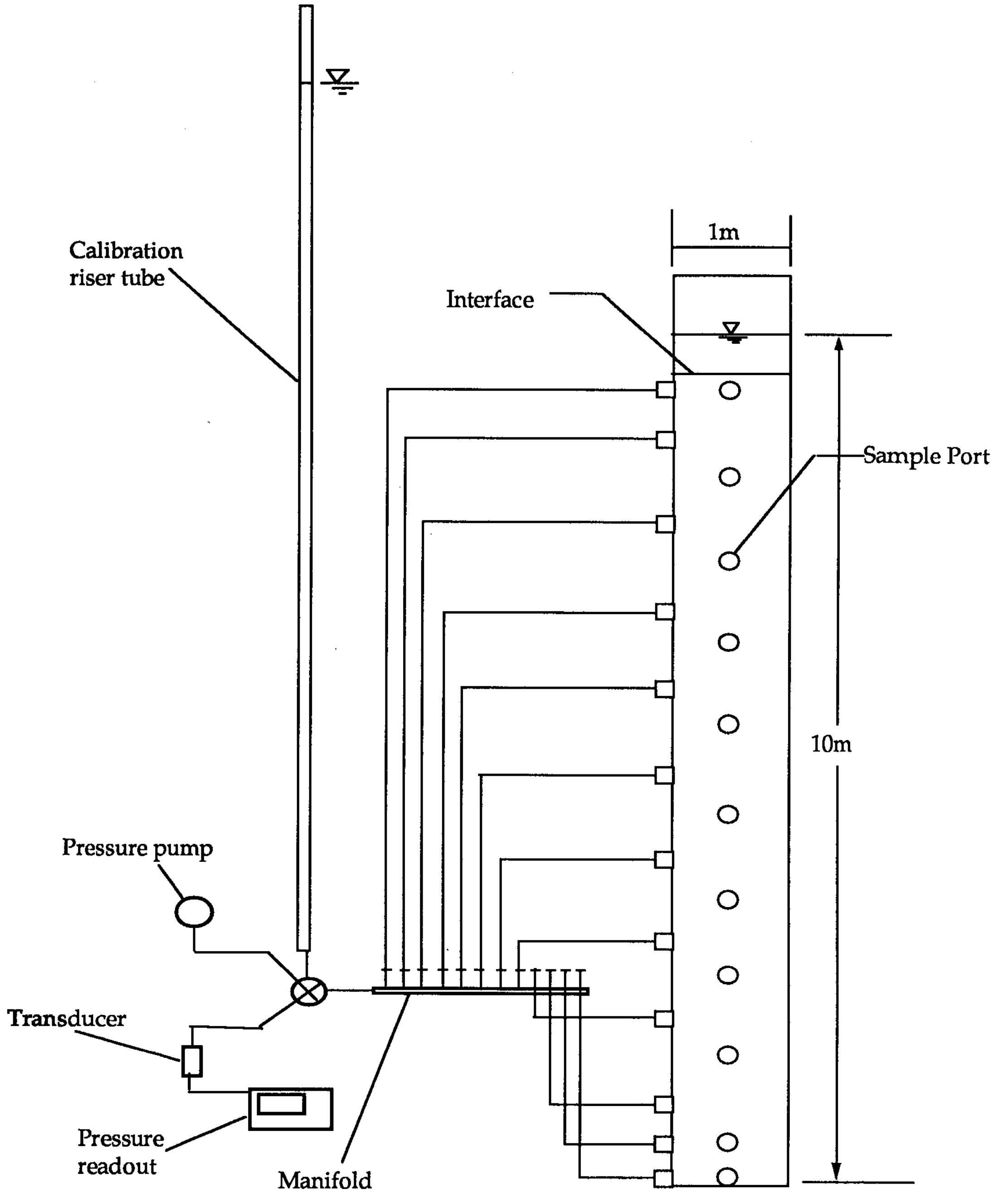

Figure 6.10 Ten Metre Standpipe

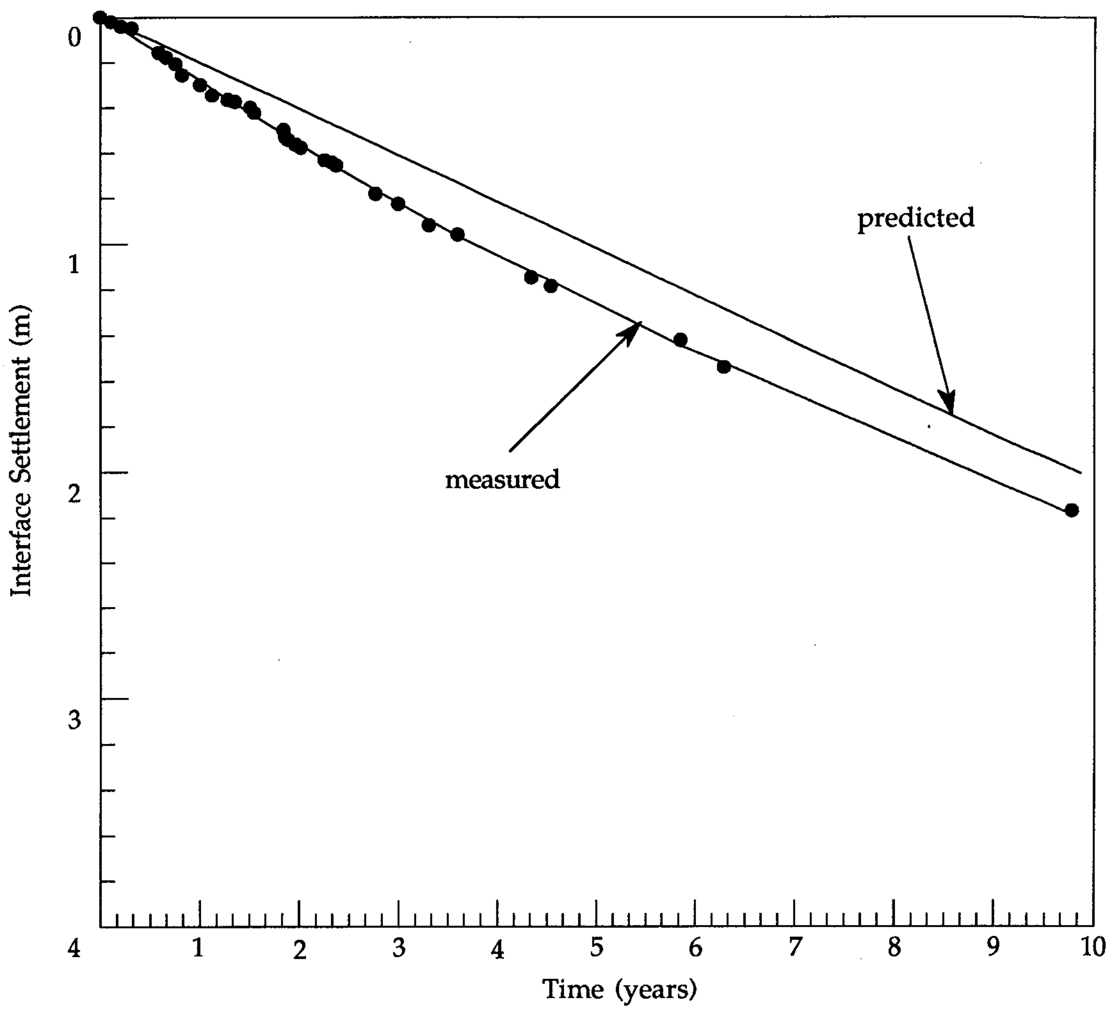

Figure 6.11 Measured and Predicted Settlement in a Ten Metre Standpipe

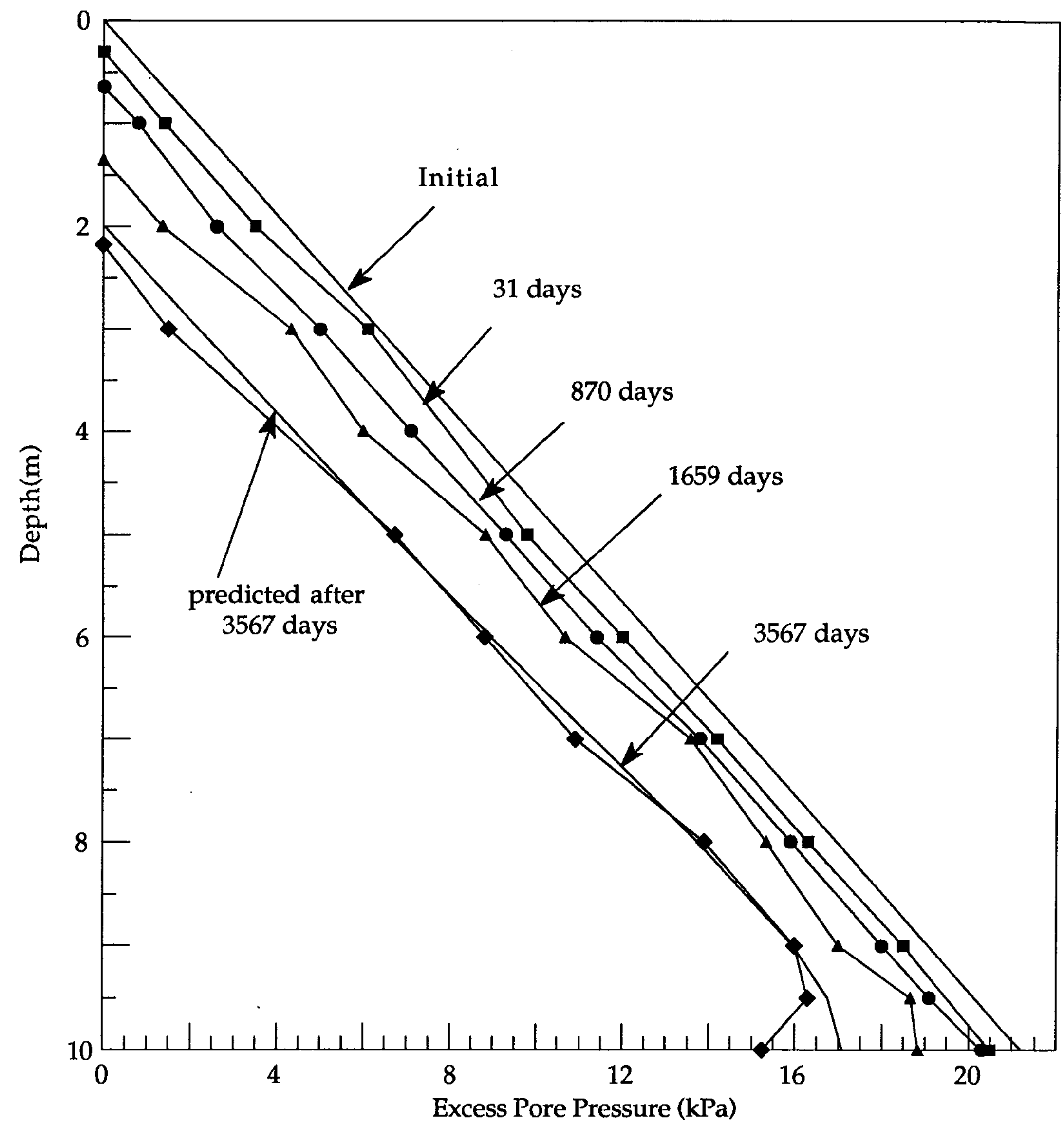

Figure 6.12 Measured and Predicted Excess Pore Pressures in a Ten Metre Standpipe

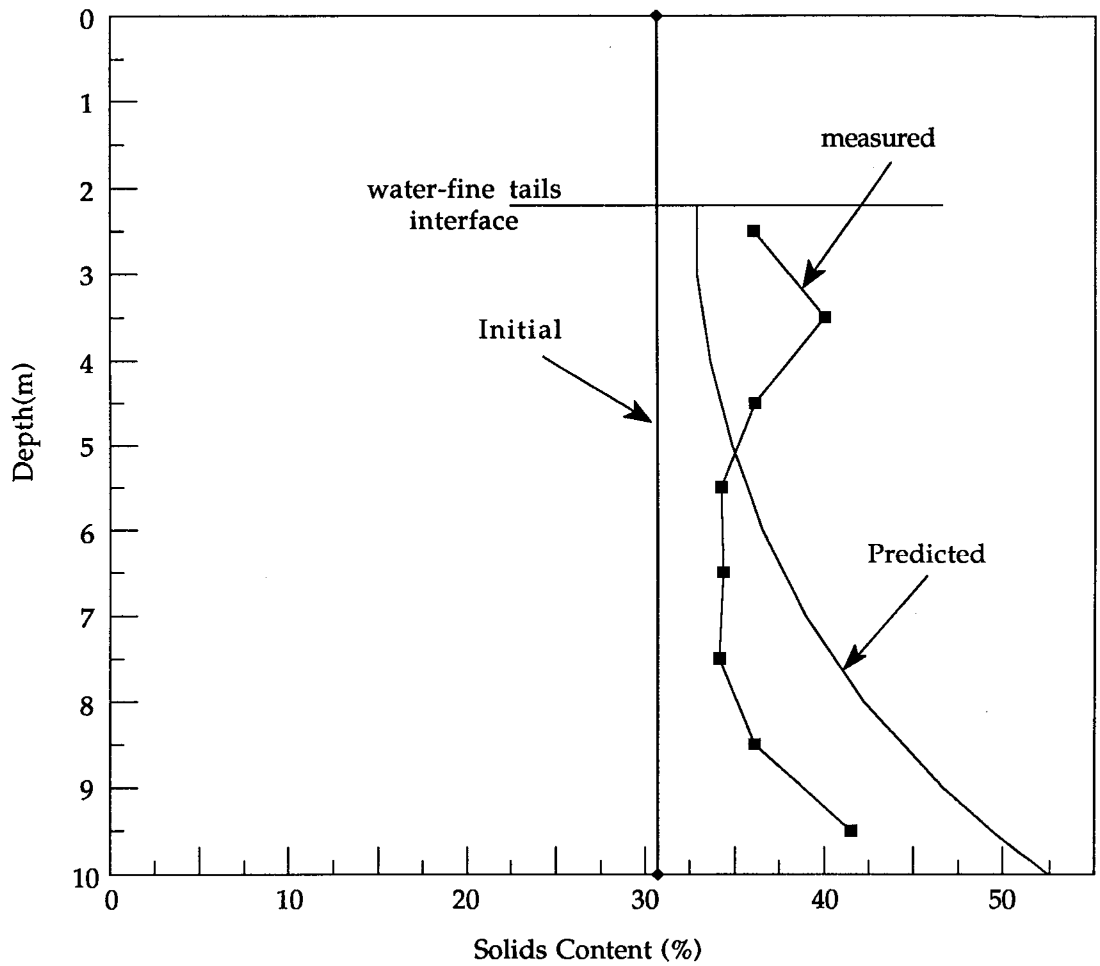

Figure 6.13 Measured and Predicted Solids Content in a Ten Metre Standpipe after 10 Years

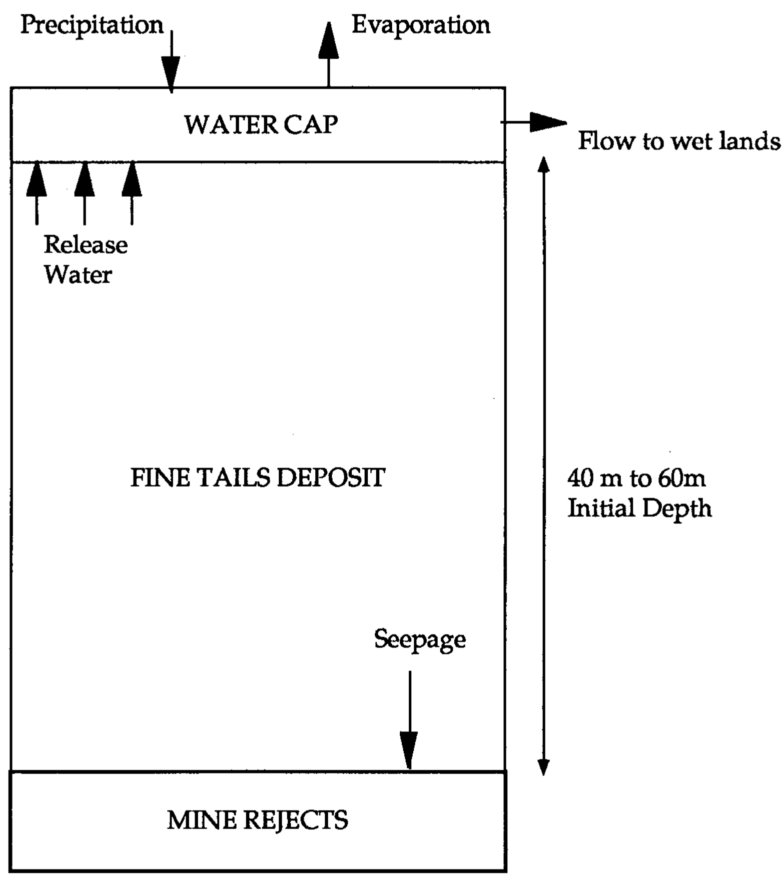

Figure 6.14 Proposed Wet Landscape Fine Tails Disposal
in Mine Pit

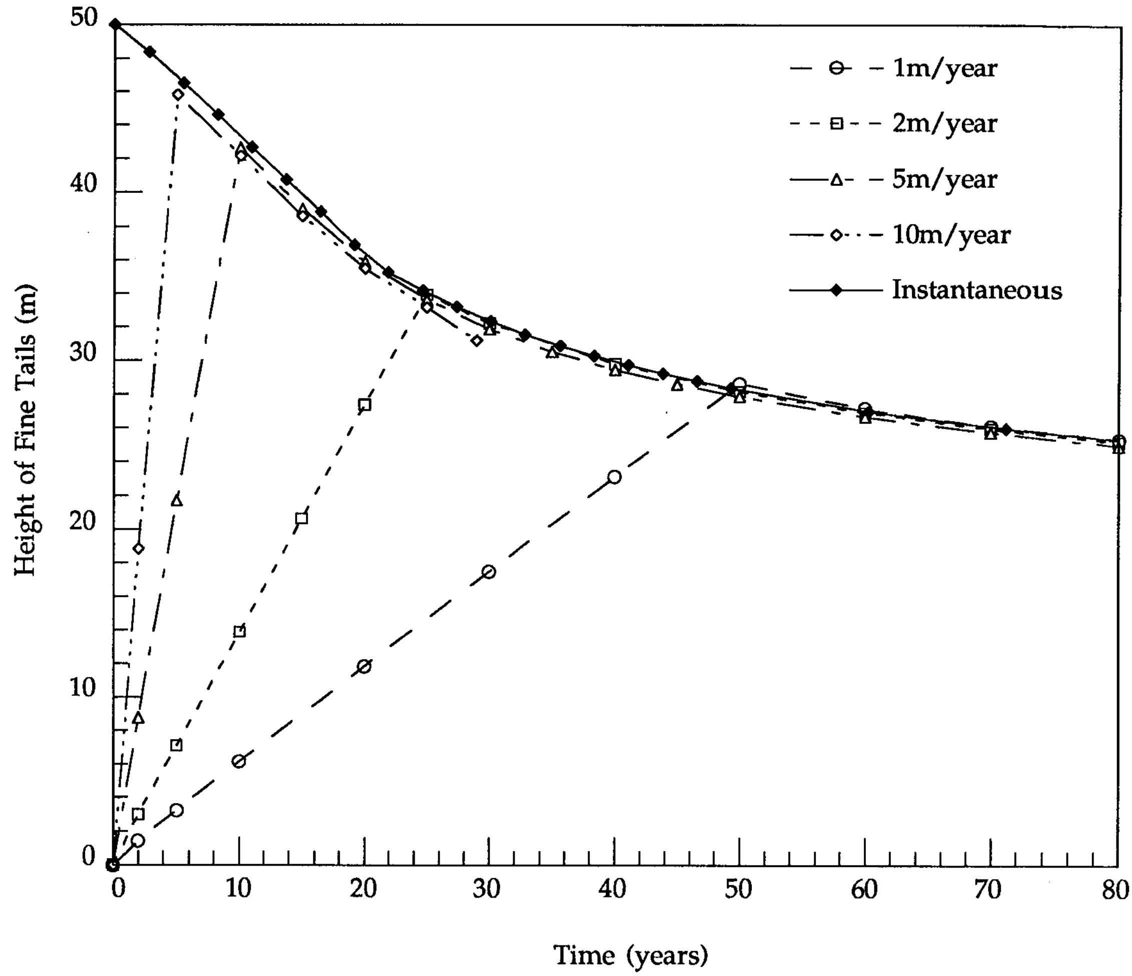

Figure 6.15 Interface Height with Time for Different Filling Rates

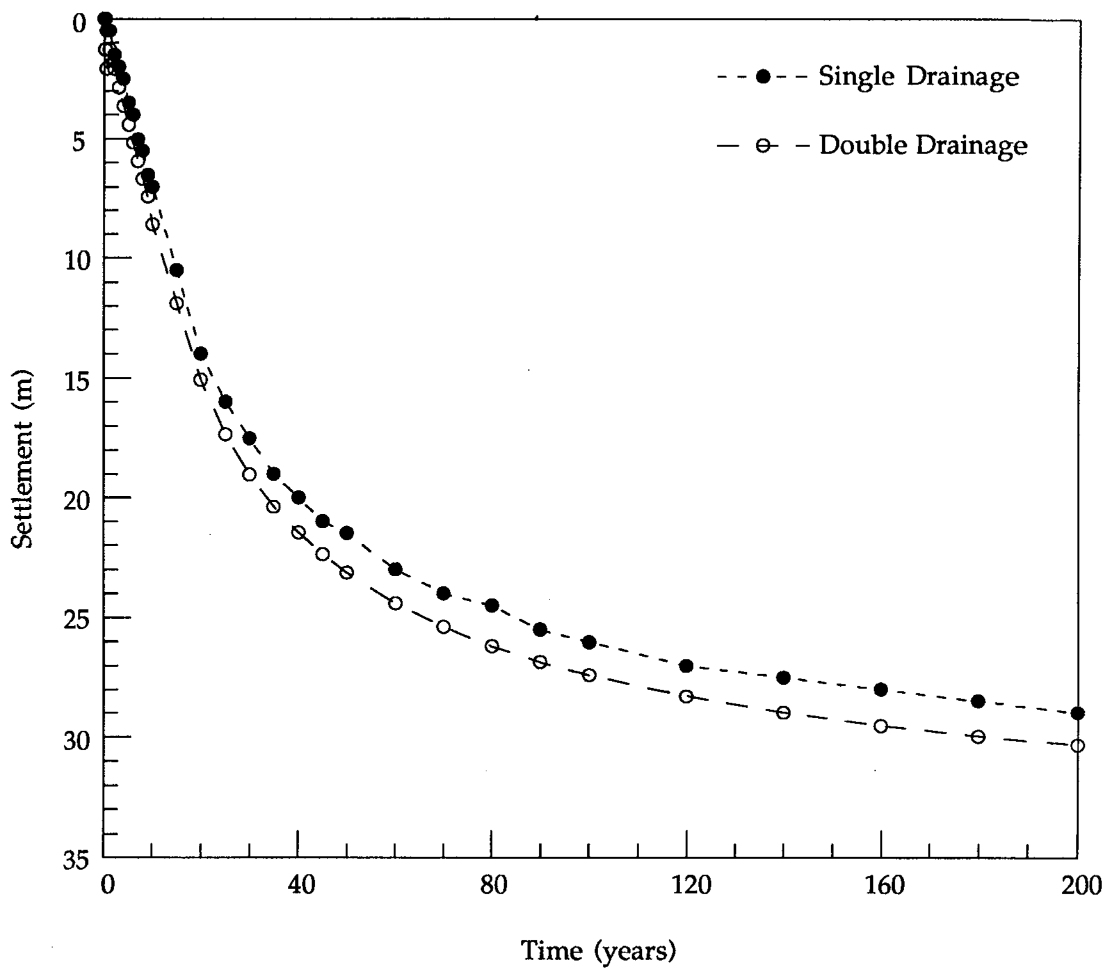

Figure 6.16 Settlement with Time in Single and Double Drainage

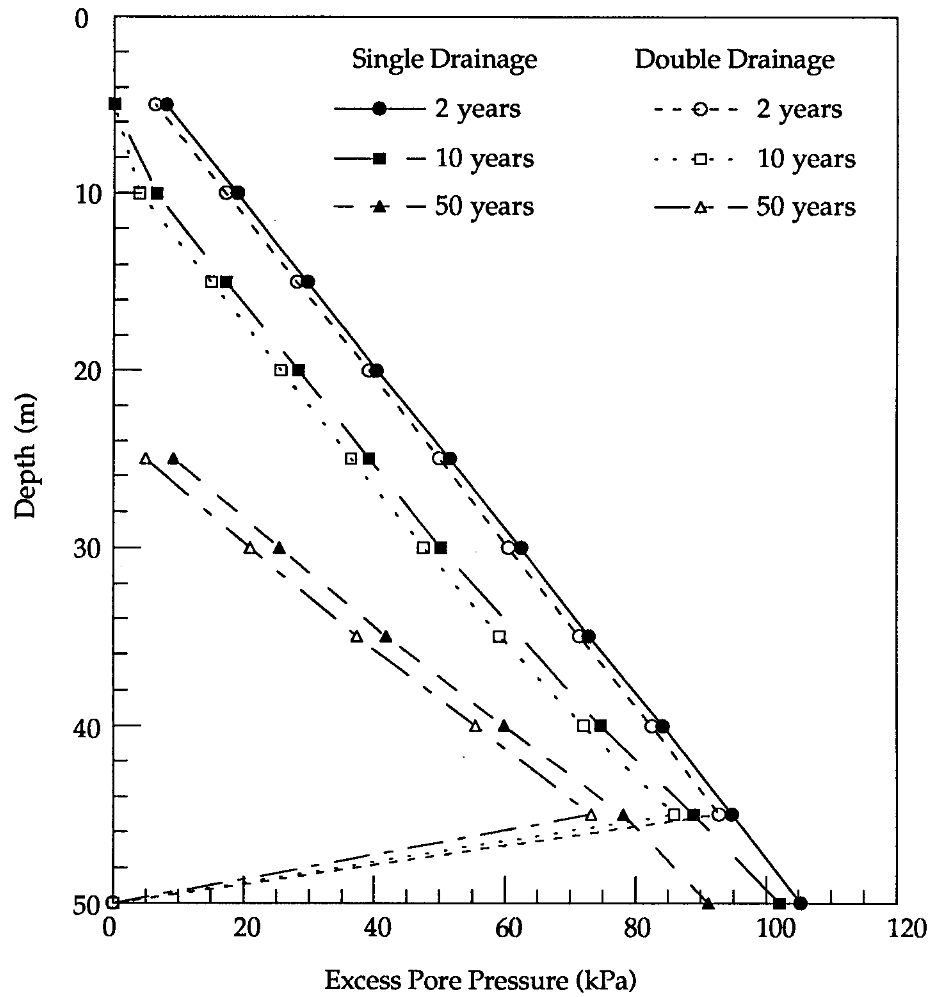

Figure 6.17 Excess Pore Pressure Distribution in Single and Double Drainage

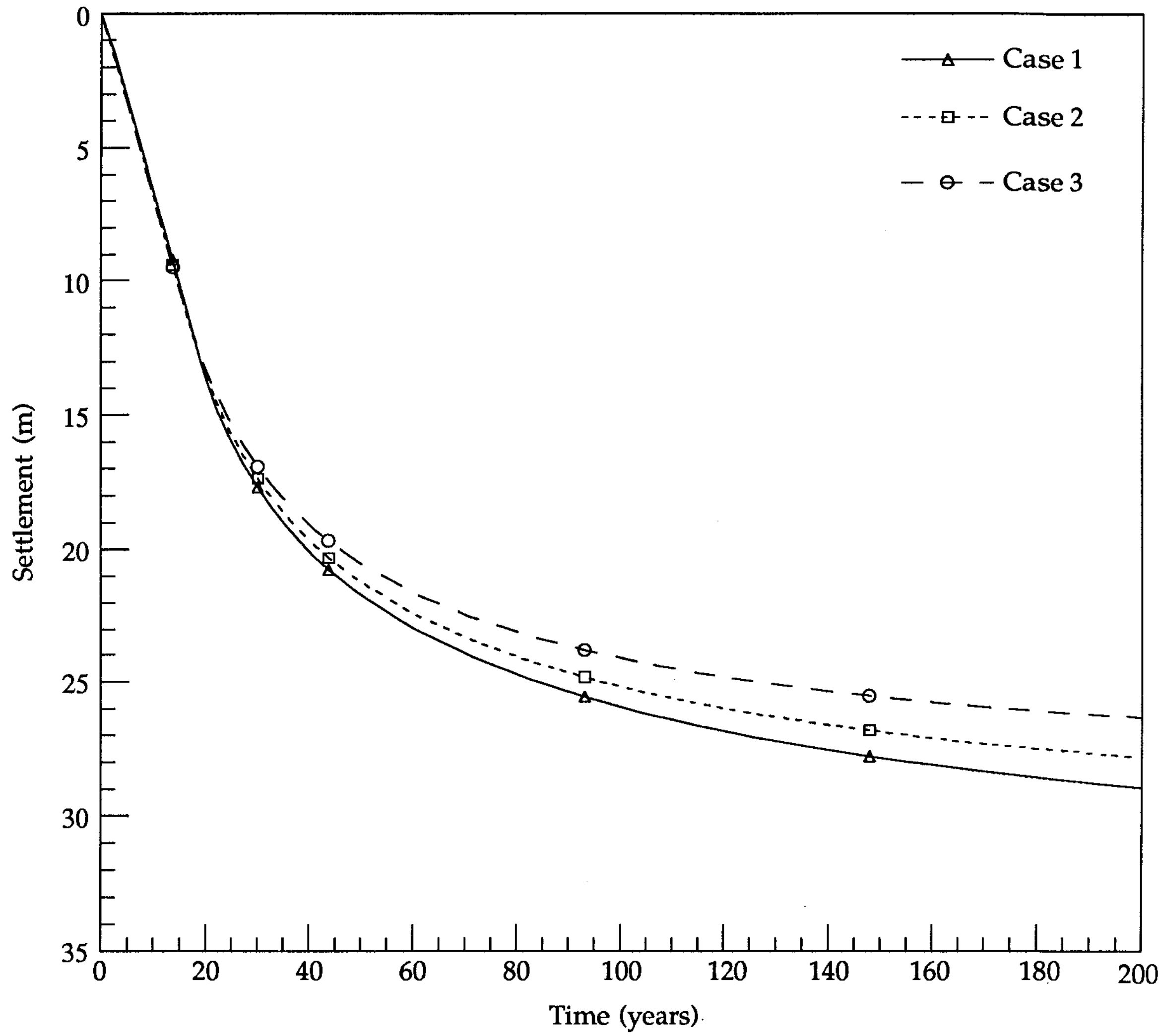

Figure 6.18 Effect of Compressibility on Predicted Settlement

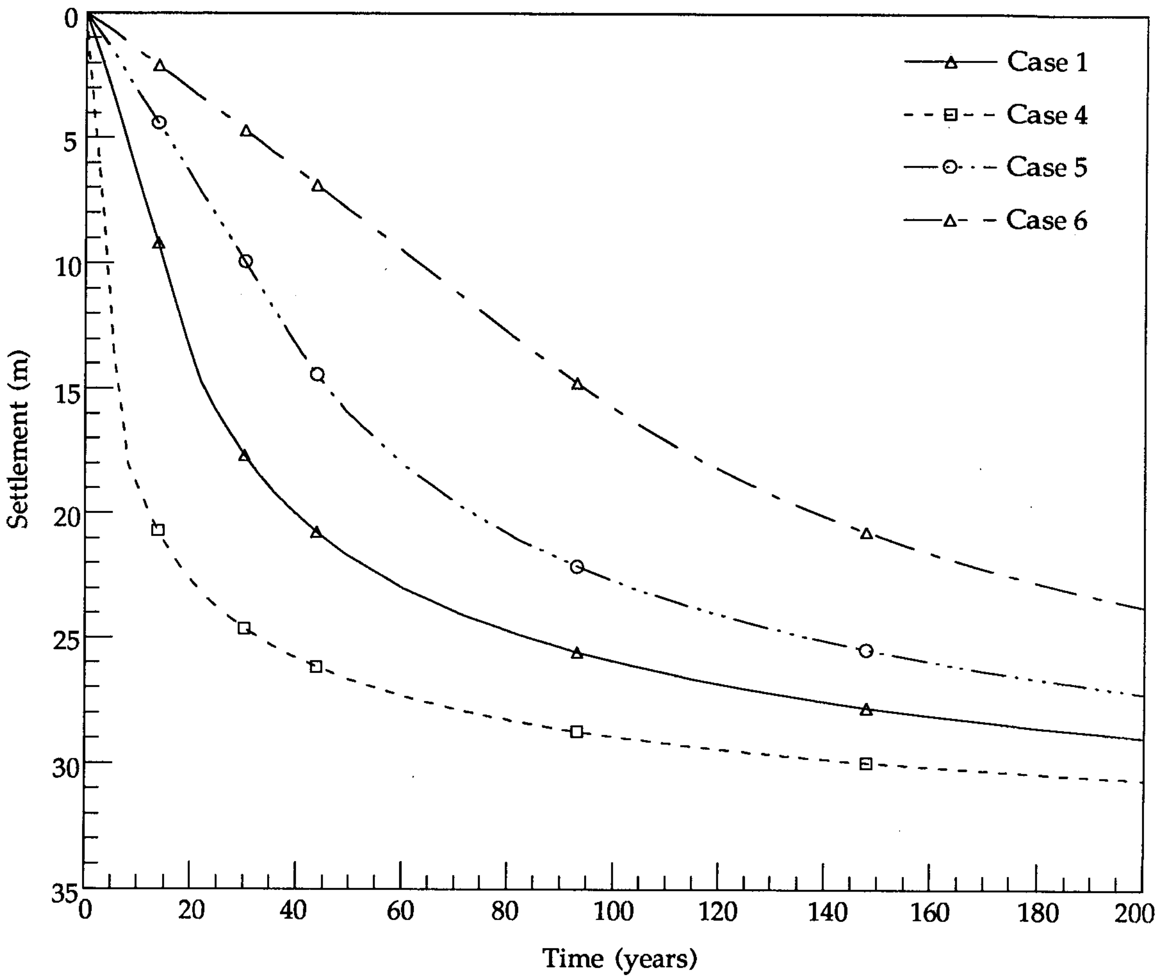

Figure 6.19 Effect of Permeability on Predicted Settlement

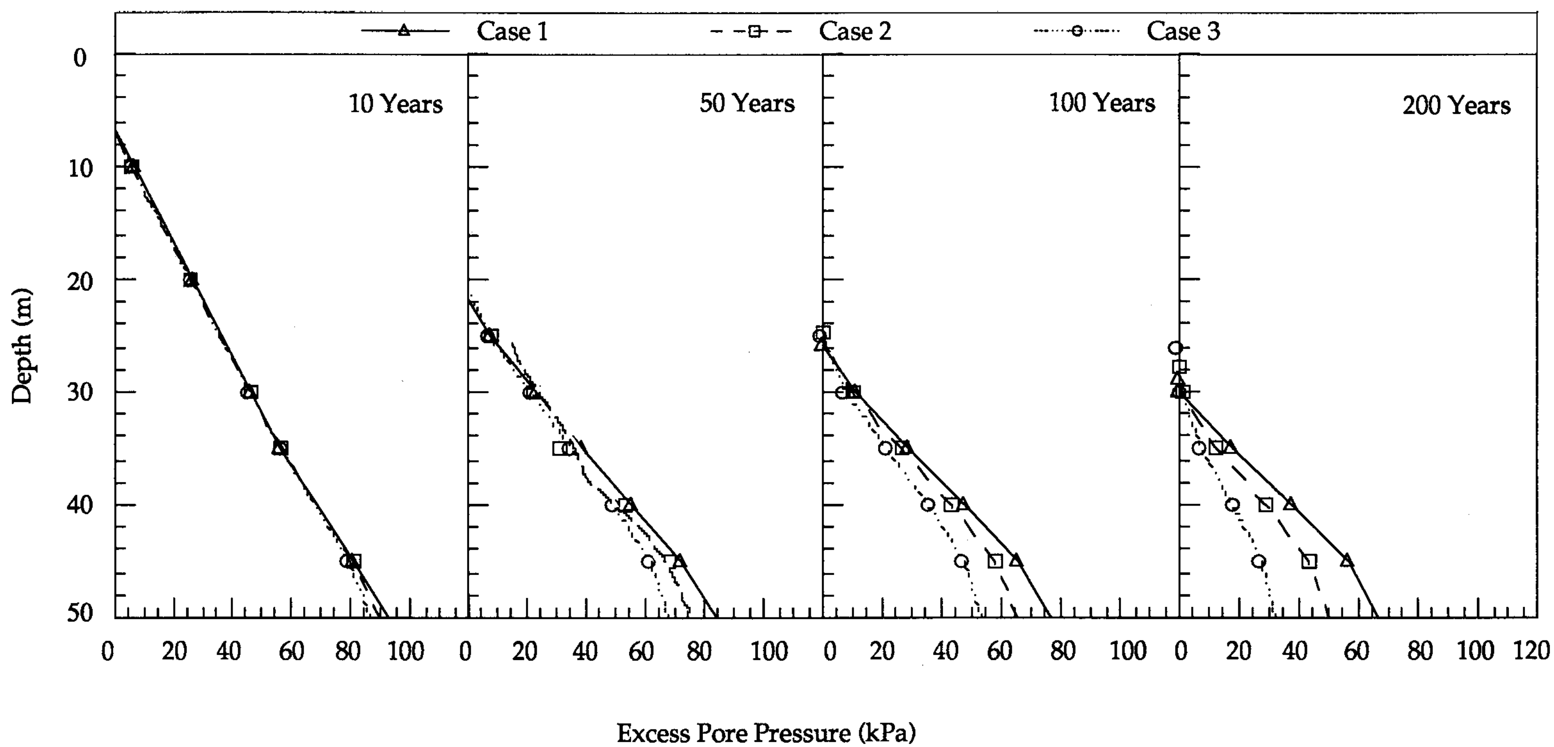

Figure 6.20 Effect of Compressibility on Predicted Excess Pore Pressure

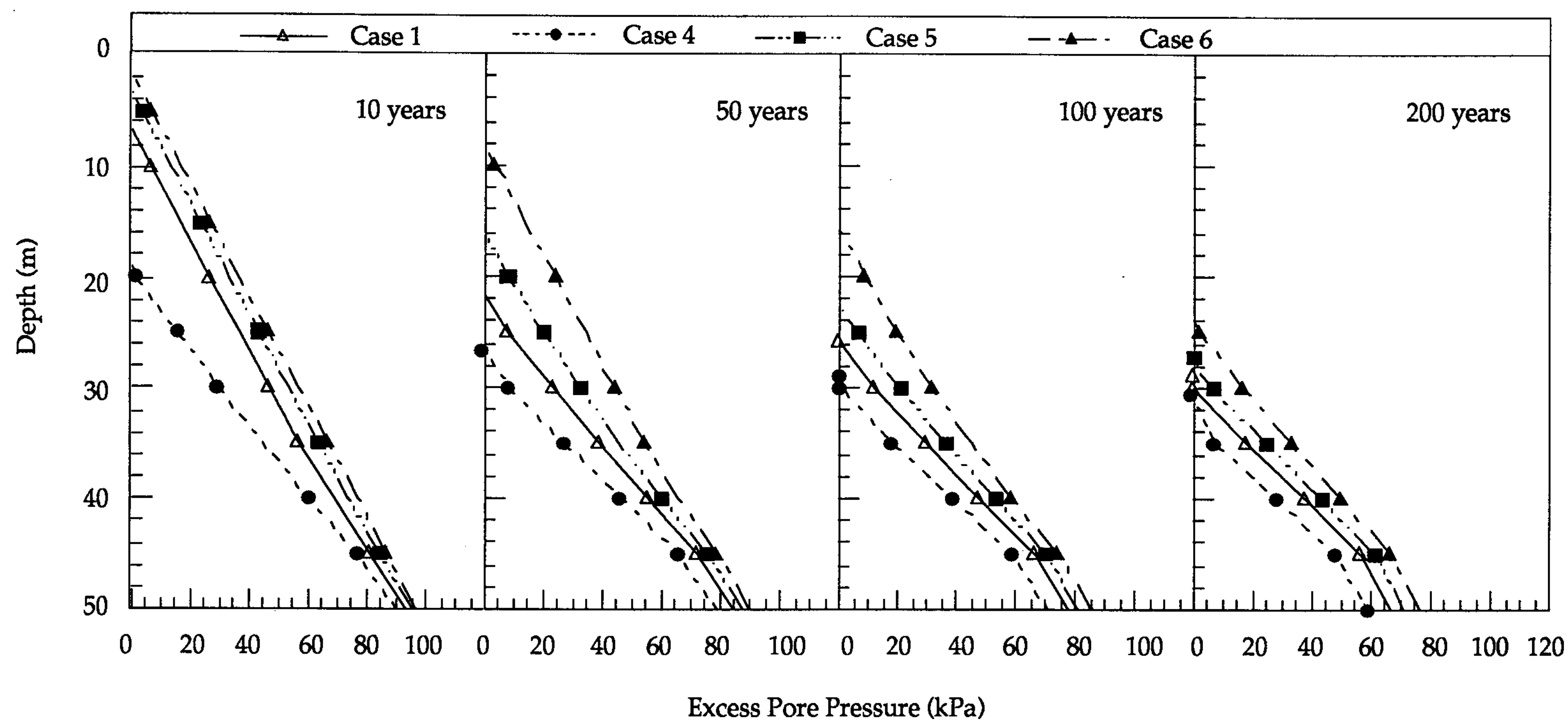

Figure 6.21 Effect of Permeability on Predicted Excess Pore Pressure

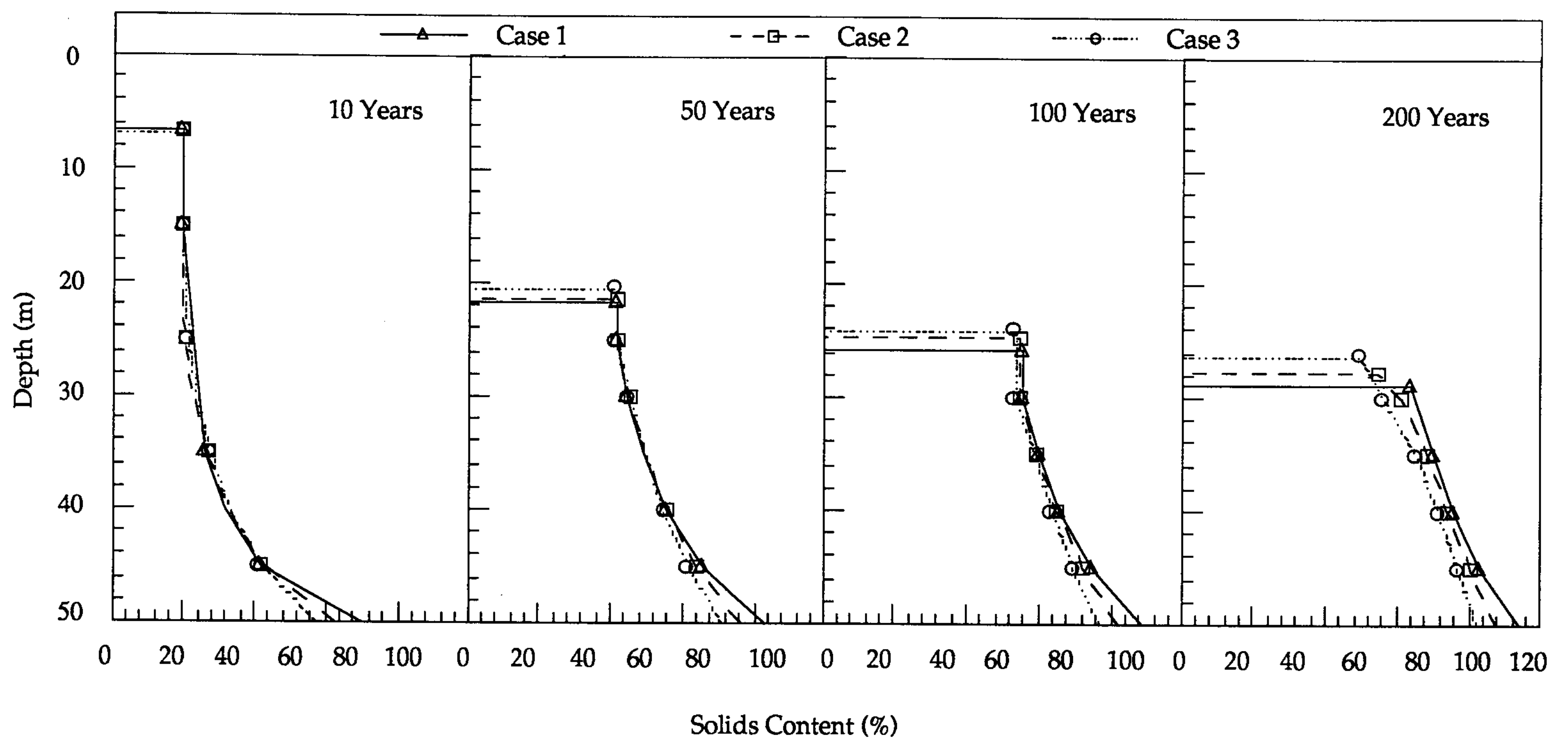

Figure 6.22 Effect of Compressibility on Predicted Solids Content

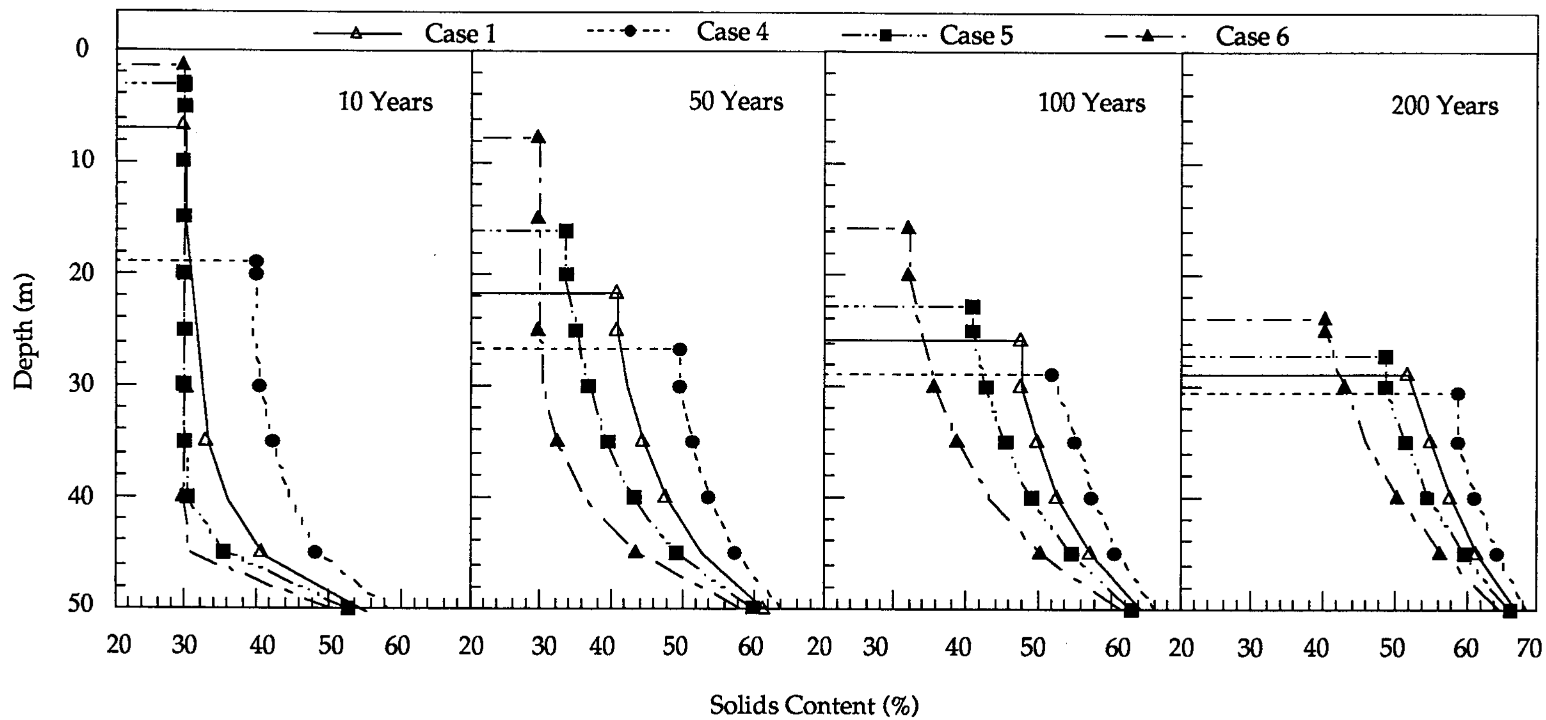

Figure 6.23 Effect of Permeability on Predicted Solids Content

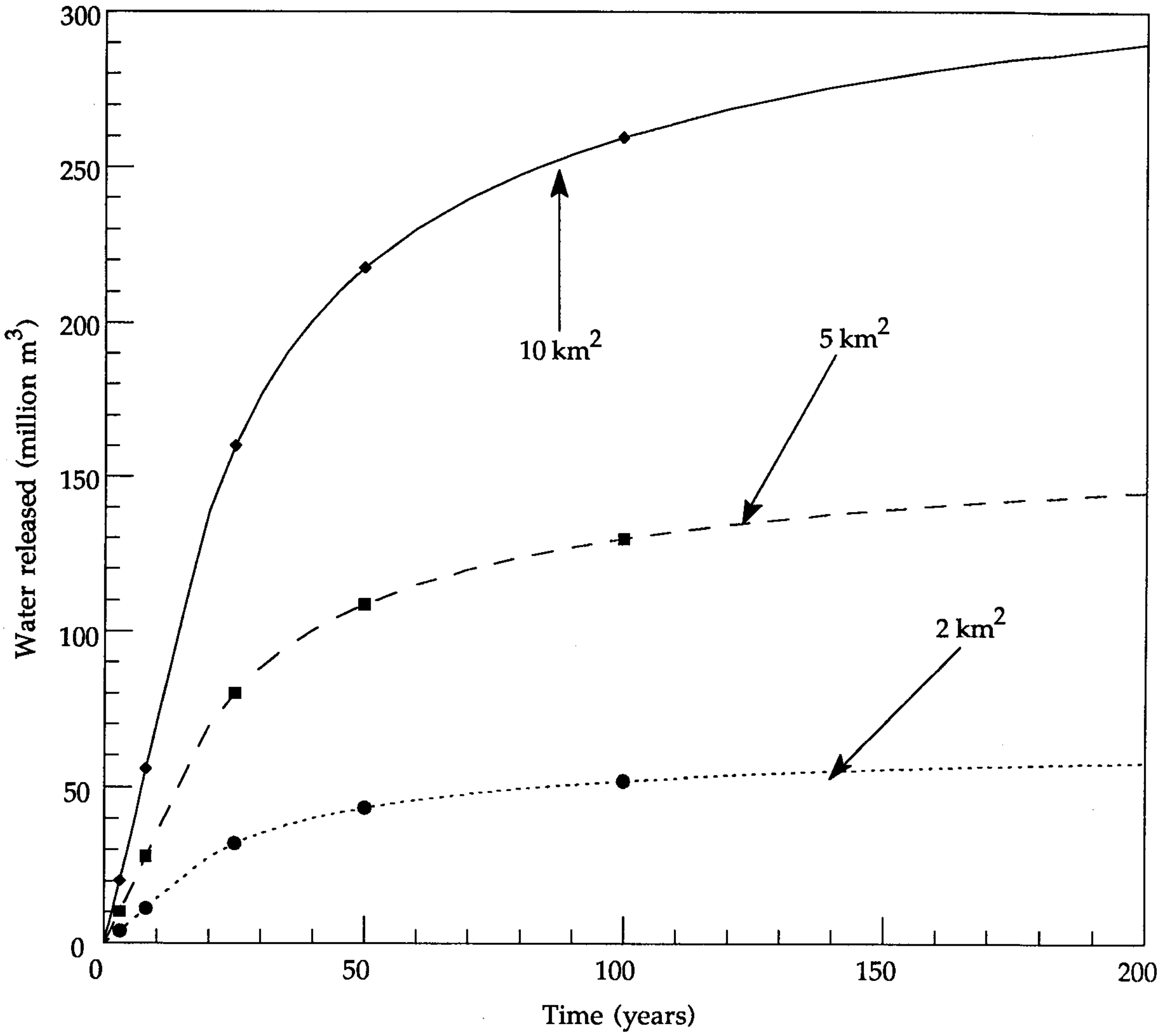

Figure 6.24 Water Released with Time

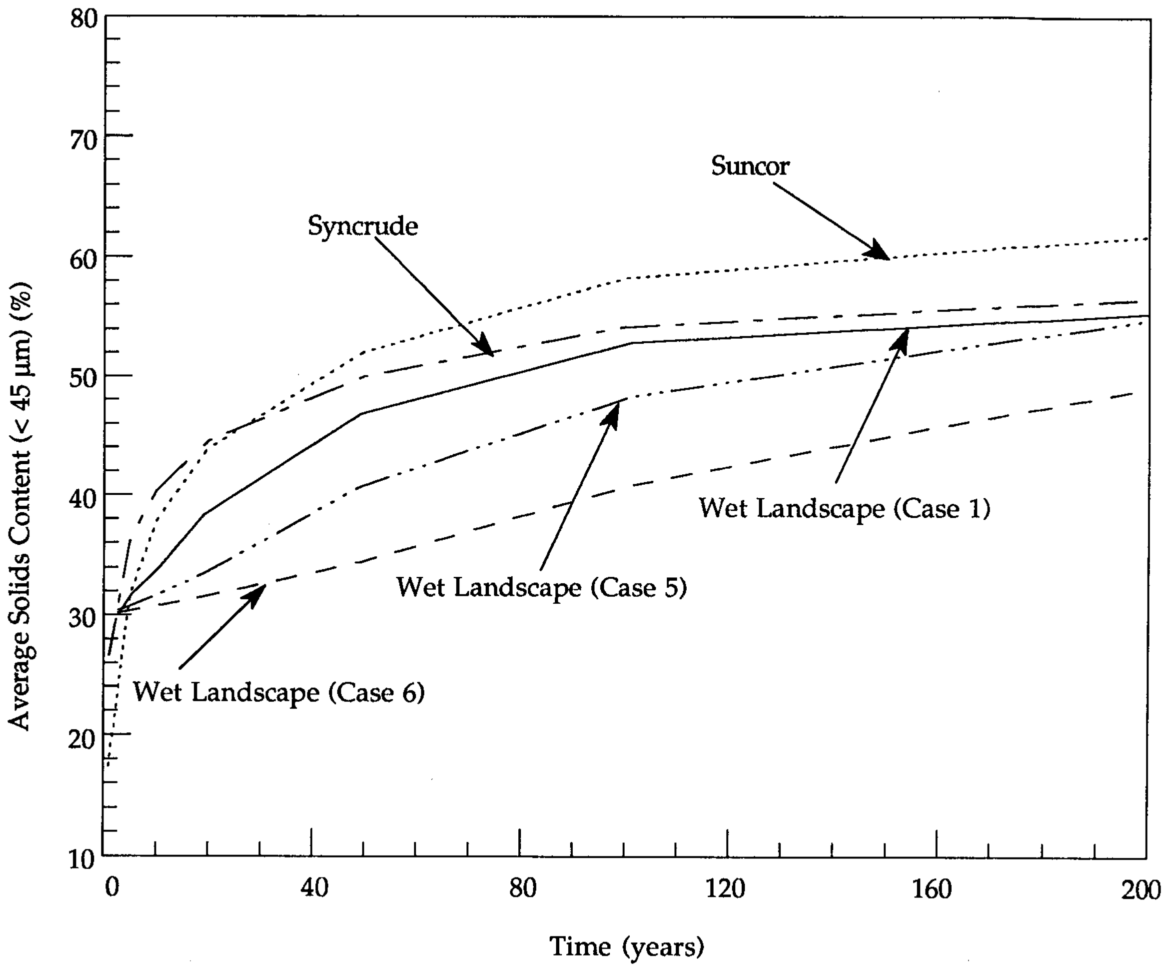

Figure 6.25 Comparison of Predicted Average Solids Content

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

7.1.1 Compressibility and Permeability

A slurry consolidometer has been developed to allow large deformations during consolidation and to allow permeability tests to be performed on the fine tails. Development of a top cap clamping system has permitted permeability testing to be conducted on the high-void-ratio slurries without introducing seepage-induced consolidation.

Unlike normal soils, the compressibility of the fine tails is found to be controlled by the initial void ratio of the sample, which suggests that aging changes the microstructure of the fine tails and hence the compressibility. It suggests that there is a time-dependent parameter involved in the compressibility of fine tails. The older the fine tails, that is, the longer they have consolidated in the tailings pond, the smaller the void ratio they will reach under an applied effective stress. Therefore a single void ratio-effective stress relationship is not sufficient to describe the consolidation behavior.

In general, the void ratio - permeability relationship of oil sands fine tails is influenced by hydraulic gradient and bitumen content. A transient state exists in the flow through the fine tails in the laboratory during permeability testing which requires some time to decrease to a steady state. This drop in flow velocity during the transient state decreases with decreasing void ratio. This phenomena can be attributed to the reorientation of fine particles due to the seepage force, which is found to be reversible. The presence of bitumen also can cause such transient behavior but to a lesser degree. For appropriate measurement of permeability in the laboratory, the flow velocity should be measured with time and the steady state velocity must be used.

Deformation of bitumen from seepage forces makes the permeability of fine tails dependent on the hydraulic gradient. The permeability of fine tails, therefore does not conform to Darcy's law. Measuring permeability at field hydraulic gradients of less than 0.2 will lead to reliable use in predictions. It was determined that the permeability of the fine tails can be expressed in a power law form in terms of void ratio.

7.1.2 Creep

The creep rate of fine tails increases linearly with void ratio as with other soils. Fine tails shows a large creep index when compared to other clayey materials and is similar to that of organic silt. C_α increases linearly with C_c and the ratio of C_α to C_c for fine tails is found to be 0.085 which is high and similar to other organic soils.

For fine tails-sand mixes, there exists an optimum e_f where the C_α was a maximum. The optimum e_f increased with an increase in the sand content. The optimum e_f might represent the void ratio at an apparent preconsolidation stress caused by the sand content. There exists a limiting fines content beyond which the creep behavior of fine tails-sand mixes is similar to that of fine tails. This occurs because at a high fines content the sand grains are not in contact and the fines control the geotechnical behavior.

In the 10 m deep deposit, the measured effective stresses indicated that creep is occurring in the column and the calculated settlement using the laboratory measured creep index showed that 30% of the total settlement at the end of 10 years can be attributed to creep.

In the tailings ponds, the primary consolidation of the fine tails will take a long time to complete because of its low permeability and large depth. The very small effective stresses which have developed over 15 to 20 years indicate that the volume decrease of the fine tails has a creep component which is taking place during primary consolidation. At its present void ratio of about 6, the fine tails in the pond has a very high creep index of about 22%. Finite strain consolidation analysis and creep analysis of a 50 m deep pond indicates that over a 20 year period about 27% of the total settlement would be due to creep.

7.1.3 Thixotropic Strength Measurements

Cavity expansion tests can be successfully used to estimate the thixotropic strength of fine tails and other similar materials. The strain rate in cavity expansion tests did not significantly affect the strength.

Long term strength developed in slurries can be either thixotropic or a combination of thixotropic and consolidation strengths. At low water contents (< 100%), the effect of consolidation strength development did not exist as the water content remained unchanged. At higher water contents, water content changes were prominent indicating the existence of consolidation effects. Interpolation techniques, however, were used to isolate the thixotropic strength development.

Oil sands fine tails is a highly thixotropic soil. It exhibits higher thixotropic gain in strength than typical clays and clay minerals. The thixotropic strength in the fine tails developed rapidly and increased quadratically with age. Strength increases were still continuing at the end of the test period of 450 days. The thixotropy of fine tails is highly dependent on water content. Substantial thixotropic strength increases were seen for fine tails with water contents less than 150%. The lower the water content, the higher the thixotropic strength. The thixotropic strength ratio for fine tails was at a minimum at 150% water content. Generally, the thixotropic strength ratio increased with an increase in water content.

The thixotropic strength may be affected by self weight consolidation. Particle shearing resulting from consolidation reduces physico-chemical bonding produced by thixotropy. The higher the rate of consolidation, the smaller the thixotropic strength gain. The study of the physico-chemical properties of fine tails should follow the same time-consolidation strength history as that of the fine tails in the tailings ponds to model the combined effect of thixotropy and consolidation.

7.1.4 Standpipe Tests

The results of the two 2 m high standpipe tests indicate that the finite strain consolidation model using the consolidation test measurements from the slurry consolidometer does not model the fine tails consolidation in the 2m standpipe very well. A permeability value (Case 6) four times smaller than the average laboratory measured permeability (Case 1) had to be used to obtain a fit. If the typical Case 1 material properties are used in the analyses, the rate of settlement, the rate of pore pressure dissipation and the increase in solids content are all over predicted.

For the 10m high standpipe, the measured settlements are slightly larger than the predicted settlement. The predicted and measured pore pressures agree extremely well except at the bottom which may be due to a segregated sand layer at the bottom. The solids content with depth, however, was poorly predicted possibly because of the thixotropic and creep behavior of fine tails. This indicates that the long term solids content distribution with depth can not be modeled without including creep and thixotropy of the fine tails.

7.1.5 Wet Landscape

The rate of filling the disposal pit in the wet landscape option does not have any influence on the long term consolidation behavior after the filling period. Providing drainage at the bottom of the fine tails did not considerably improve the consolidation. Therefore provision of a drainage layer in the wet landscape may not be viable as it does not accelerate the consolidation significantly.

Variations in the compressibility relationships had a small effect on the wet landscape predictions. Therefore, the model prediction will remain reasonably accurate even if the fine tails compressibility in the tailings pond is different from that measured in the laboratory. Small variations in the permeability relationships can cause severe deviations from accurate predictions, especially during long term consolidation. Accurate characterization of field permeability is essential to predict the fine tails consolidation behavior in the wet landscape disposal option. Using samples which represent the field material and with proper measurement techniques, reasonable estimates can be made. It would appear, however, that field measurements of permeability are essential.

A relatively rapid rate of settlement is expected in the wet landscape disposal pond for the first 30 years which will then be followed by a slow rate of settlement diminishing towards an ultimate settlement of 33 metres in about 1000 years. The major portion of water will be released from the wet landscape within the first 50 years, after which the potential for contamination is greatly reduced.

Modeling of the standpipe tests showed that Case 6 material parameters were necessary to model the 2 m high standpipes and that Case 5 material parameters were necessary to model the 10 m high standpipe. It can be concluded that the field performance will probably be closer to the Case 5 and Case 6 predictions. The Case 1 predictions and the empirical predictions overestimate the rate of consolidation.

7.2 Recommendations

Further advances in the understanding of fine tails consolidation require field measurements in the existing tailings ponds. It is essential that in situ measurements of total vertical stress and pore pressure be made at the same time and location so the present small effective stresses can be determined. Field permeability measurements are also necessary to allow reliable predictions of consolidation rates to be made.

The S-shape compressibility curves should be taken into account in using the finite strain consolidation analysis. This procedure would require a reformulation of the finite strain consolidation model.

Long term creep tests should be conducted to verify the measured creep properties for fine tails. The incorporation of creep properties in the finite strain consolidation theory is needed for long term standpipe test and field predictions.

Equipment should be developed to measure the shear strength of the fine tails in field conditions, as well as in the laboratory, other than vane shear tests. An in-situ testing program should be performed to confirm the laboratory measured values.

It is necessary to develop a non sampling procedure, such as, nuclear density measurements, to determine the density of the fine tails in the 10 m and 2 m standpipes. The monitoring of the consolidation of fine tails in the 10 m high standpipe should continue as the behavior can be best be studied from such large scale, long term, well controlled tests.

APPENDIX A: LARGE SCALE TESTING OF OIL SAND FINE TAILS[1]

1. Introduction

In Northern Alberta, oil sand deposits are mined and processed to recover heavy oil from two oil sand plants, Syncrude Canada Ltd. and Suncor Inc. Syncrude and Suncor, respectively, produce about 480,000 tonnes per day and 170,000 tonnes per day of tailings which varies in solids content from 40% to 60%. The tailings streams are composed of 75% to 90% sand and 10% to 25% fines by weight with a small amount of heavy oil which bypasses the extraction plant. The disposal of the tailings stream is accomplished by allowing the sand to settle out to construct dykes and beaches which accumulate at the rate of 225,000 m^3 per day. Much of the fines are carried into the pond as a fine tails stream around 8% to 10% solids. When the stream flow slows, sedimentation of the fine tails particles begins.

Sedimentation is the process in which the solids fall through an ambient fluid where stress is not transferred from one particle to another. At a solids content of around 15%, the oil sand fine tails solids begin to form a matrix such that stress can be transferred from one particle to another. After reaching this solids content, any increase in fine tails density arises from the process of self weight consolidation. The fine tails forms a deposit at about 30% solids after 1 to 2 years of consolidation. This fine tails deposit is accumulating at a rate of 45,000 m^3 per day. Due to the slow process of self weight consolidation the solids content increases slowly above 30% solids and the fine tails must be stored in large tailings ponds which are a major environmental problem.

The rate and amount of consolidation of fine tails can be measured in the laboratory by large strain slurry consolidation tests. The results from these tests can be extrapolated to predict the long term behavior of the fine

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[1] -A version of this appendix has been published. Suthaker N.N. and Scott J.D. Proceedings of the First International Congress on Environmental Geotechnics, Edmonton, Alberta, Canada, July 10-15, 1994, 557-562.

tails in the tailings ponds using a finite strain consolidation theory. The extrapolation of short term, laboratory tests to long term field behavior, however, requires verification so that the extrapolated results can be used with confidence for long term planning and design of tailings ponds. A large scale self weight consolidation test in a ten metre high standpipe has been conducted for more than ten years to provide this verification. The objective of this study is to review the results of the testing programs and compare the consolidation test performance and the 10 m standpipe performance.

2. Two Metre Standpipe Tests

Self-weight standpipe tests from 180 cm to 200 cm in height have been conducted to determine the completion of sedimentation and subsequent progress of consolidation of the fine tails. The two metre height was selected in order to obtain a measurable effective stress. The standpipes (Figure A.1) were continually monitored for settlement of the fine tails-water interface. Pore pressure changes were measured at 20 cm height intervals by small diameter manometers. Samples (5 ml) were obtained from another set of ports and the density and solids content were determined.

Figure A.2 (Scott et al., 1986) shows different stages of a 180 cm standpipe sedimentation and consolidation test on a fine tails sample (initial solids content of 10% and density of 1.05 g/cm^3). The initial pore pressure at the base of the column was equal to the total mass of the column of fine tails showing that no effective stress existed in the slurry at the beginning of the test. During sedimentation of the fine tails, a density increase has started from the bottom of the tube, where the fine tails particles had developed effective stress between them. As the water-fine tails interface settled, the effective stress front traveled upwards. After 2.5 days, the full column had developed an initial interparticle stress. No significant consolidation had taken place at this time. Consolidation proceeded downward from the fine tail surface not upwards from the bottom as consolidation theory predicts. This phenomenon was also observed in the ten metre standpipe and an explanation is provided later in this article. Consolidation was practically complete after 300 days. The interface continued to settle slowly after this time due to secondary consolidation.

Such laboratory tests coupled with field measurements in the tailings pond showed that sedimentation of the fine tails is fairly rapid and could be considered to be complete when the solids content had increased to approximately 15%. Further increases in density of the fine tails has to take place by consolidation. Imai (1981) pointed out that the solid content at which sedimentation is complete is not unique for a material but depends on the initial solids content of the slurry. Similar results were found with the fine tails, but the range of solids content at the end of sedimentation was between 10% and 15% for fines-water mixtures with initial solids contents of 5% to 8%.

3. Slurry Consolidometer Tests

The slurry consolidometers were designed for samples 20 cm in diameter and 30 cm in height. With this apparatus, strains up to 80% of the initial sample height may be measured. A typical experimental arrangement for a step loading consolidation test is shown in Figure A.3.

Because the finite strain consolidation theory used requires a knowledge of the variation of the coefficient of consolidation of the material during a test, the slurry consolidometers were designed so that void ratio - effective stress and void ratio - permeability relationships could be measured for each load increment. After consolidation under a load increment was complete, a permeability test was conducted on the sample. During the permeability tests, the hydraulic gradient was kept small so that seepage induced consolidation was minimized. Figure A.3 shows a method of overcoming the tendency for seepage induced consolidation. When consolidation under a load increment was complete, the loading ram was locked in place and an upward flow constant head permeability test was performed. This procedure eliminates the risk of having seepage induced consolidation.

A step loading consolidation test was performed with a 29% initial solids content (water content of 245%) fine tails. After each increment as described earlier constant head permeability tests were also performed. Figure A.4 shows the relationship between void ratio and effective stress. The variation of permeability with void ratio is depicted in Figure A.5 (Pollock, 1988). When modeling the consolidation of fine tails, it is necessary to use the consolidation parameters in the form of power law constants for input to the finite strain computer program used.

The relationship for compressibility and permeability are

$$e = 28.71 \cdot \sigma'^{(-0.3097)} \tag{A.1}$$

where σ' is in pascals.

$$k = 7.425 \times 10^{-11} \cdot e^{(3.847)} \tag{A.2}$$

where k is in m/s.

These relationships described the laboratory data well (Figure A.4 and Figure A.5), for both permeability and compressibility.

4. Ten Metre Standpipe Test

A self weight consolidation test in a 10 m high by 1 m diameter standpipe on 31% solids content fine tails was commenced in 1982 to determine whether the material properties measured in large strain slurry consolidation tests could predict the consolidation behavior of fine tails when used in the finite strain consolidation theory.

The ten metre standpipe (Figure A.6) is similar to a two metre standpipe except that pore pressure and sample ports are located at 1 m intervals. Additional ports at 0.5 m intervals are available to allow closer measurements, but were not used except at the top and bottom of the standpipe where rapid changes occurred.

To obtain density and solids content measurements a sample probe was inserted to a desired location through a sample port which has an O-ring seal. Specimens were selected at different distances from the port to reduce disturbance at any single location. The ball valve remained closed until the probe tube passed the seal. The valve was then opened and the tube was inserted to the location to collect the sample. For high solids content fine tails which did not flow easily, a vacuum was applied to the sample bottle. The main measurements on the samples were density and solids content measurements. Extraction tests and hydrometer tests were carried out to measure bitumen content and settling of coarse sand grains in the mix.

Temperatures of the fine tails were measured in the standpipe using a probe similar in diameter to the sampling probe. The variation in temperature in the standpipe has been constant throughout the test, in summer or winter, with an increase of about 3°C from the bottom to the top of the standpipe. This gradient is not considered sufficient to have any significant effect on the consolidation behavior.

The consolidation of fine tails in the self weight consolidation test is characterized by the settlement of the water-fine tails interface. Figure A.7 shows the test results and the predicted values based on the test results from slurry consolidation tests and a finite strain consolidation numerical model. The numerical model consists of a finite difference computer program (Pollock, 1988) based on the general non-linear equations developed by Gibson et al. (1967) The compressibility and permeability were expressed as power functions (Somogyi, 1980) in order to define the consolidation parameters over a large range in void ratio. The predicted settlement was found to be more sensitive to permeability than to the compressibility.

Pore pressures were measured with a pressure transducer which was calibrated against a column of water as shown in Figure A.6. Figure A.8 shows the variations of excess pore pressure with time along the height of the standpipe. Figure A.9 shows the measured and predicted excess pore pressures after 10 years and a very good agreement is noticeable. Larger sand grains appeared to be settling through the fine tails and accumulating at the bottom of the standpipe. A small layer of segregated sand could have created a reduction in the pore pressure at the bottom of the standpipe.

The measured solids content variations with time and depth are shown in Figure A.10. Figure A.11 shows measured and predicted solids contents after 10 years. Similar to the predicted excess pore pressure profile, the increase in predicted solids content began from the bottom, which is to be expected from normal consolidation behavior. However the data shows an increase in solids content along the entire depth of the fine tails with the greatest increase near the top of the fine tails. Therefore, the measured data indicate that consolidation is occurring at all depths with the highest solids content near the top. This might be due to free gas in the voids or to creep and thixotropy occurring in the standpipe which was not considered in the theory. Due to the high compressibility and high void ratio of the fine tails, they exhibit very high creep rates. The fine tails also display a large increase in thixotropic strength which is probably due to the organic bitumen and chemical additives. The pore pressure reduction found at the bottom of the standpipe after 10 years from sand accumulation explains the solids content increase found at the bottom of the standpipe.

5. Conclusions

A slurry consolidometer test has been performed to determine the void ratio-effective stress and the void ratio-permeability relationships for the fine tails. The laboratory data for permeability and compressibility were well described by power laws. Sedimentation in the two metre standpipes was fairly rapid and was complete in a short period of time, indicating that is not a governing factor of the long term behavior of the fine tails. The ten metre standpipe consolidation results are generally in good agreement with the predicted values from the slurry consolidometer test results employing the finite strain consolidation theory. Such agreement will allow the laboratory consolidation test results to be extrapolated with confidence to predict the consolidation behavior of oil sand fine tails.

6. References

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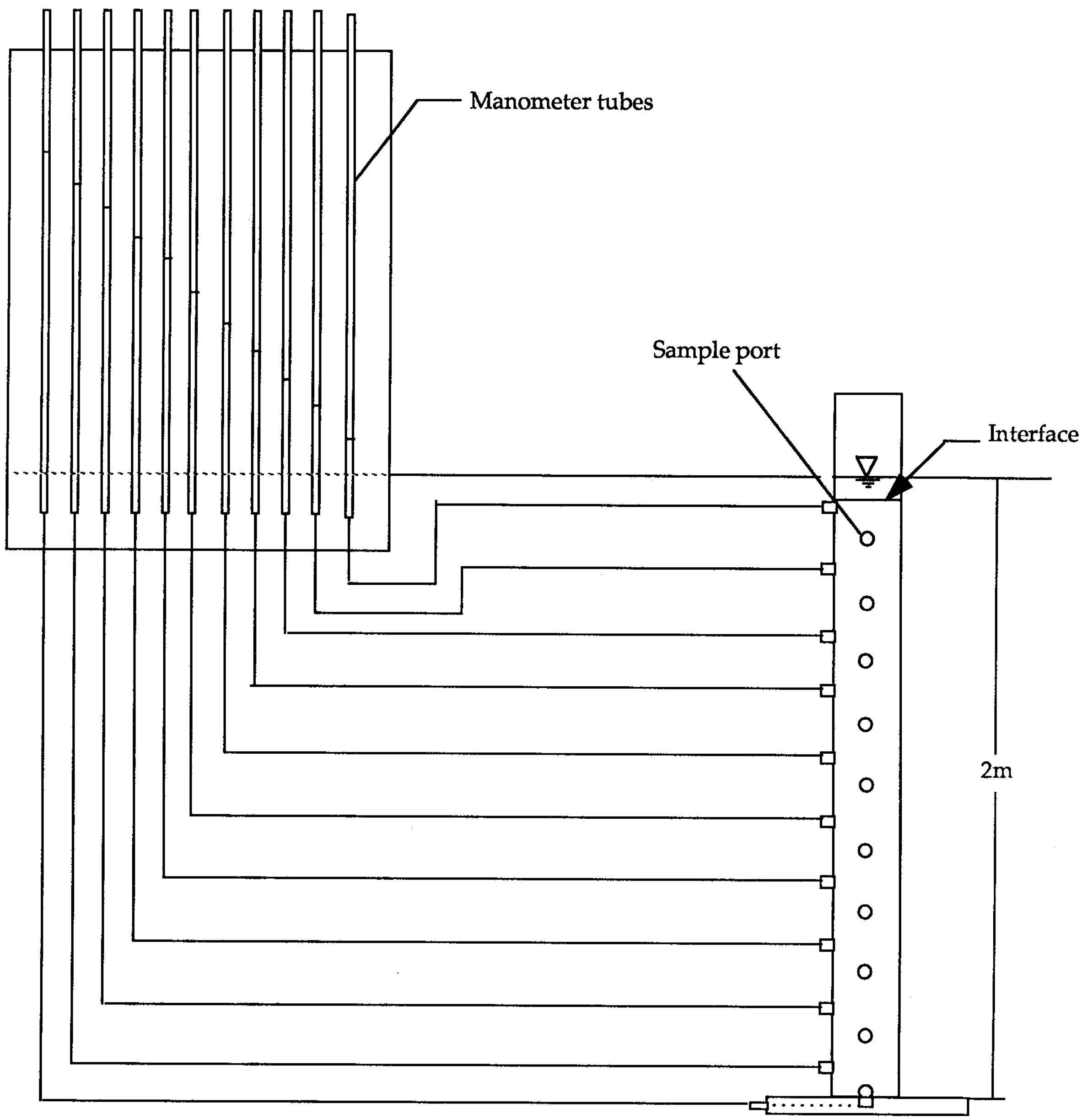


Figure A.1 Two Metre Standpipe

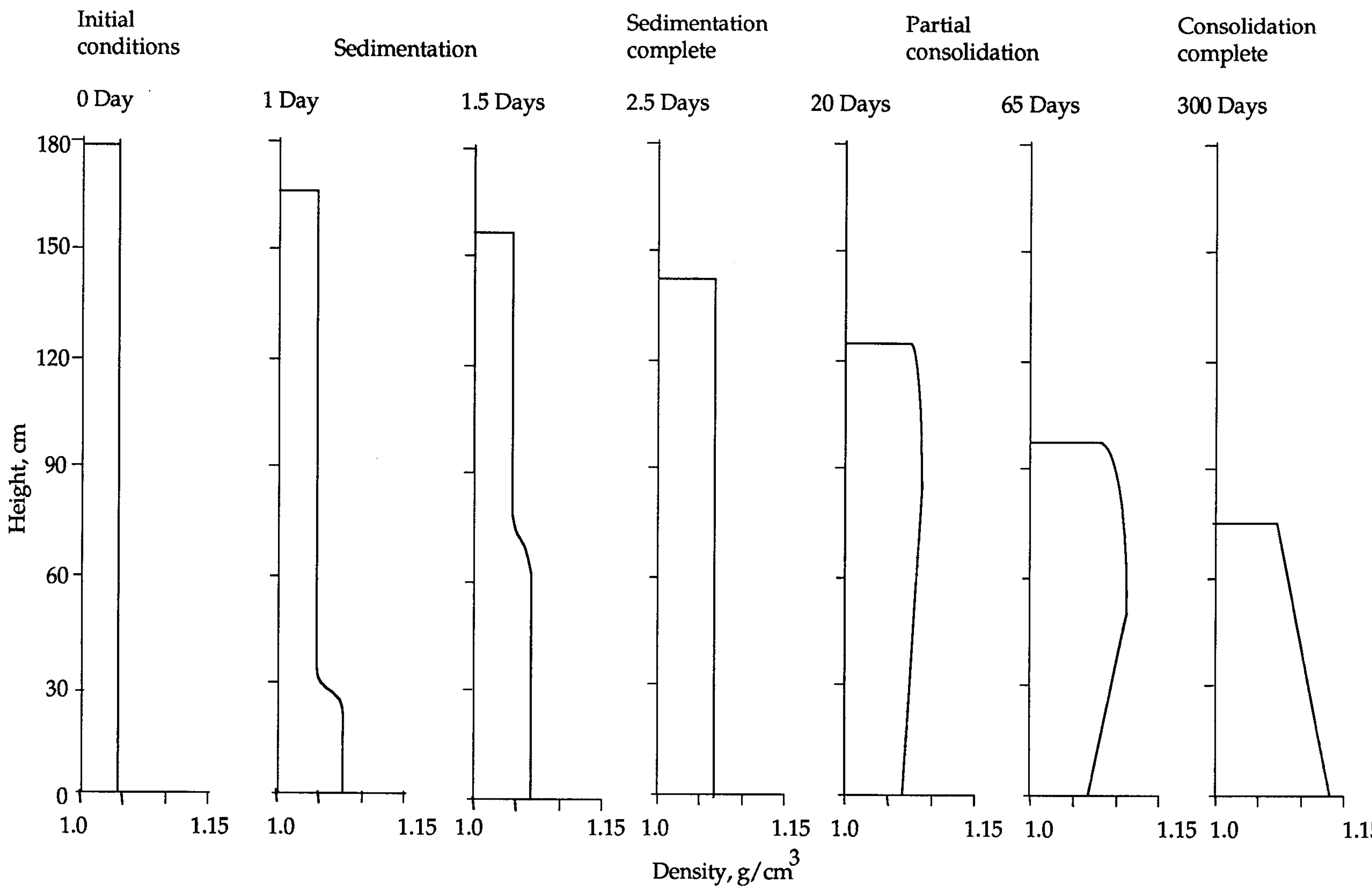


Figure A.2 Stages of Sedimentation

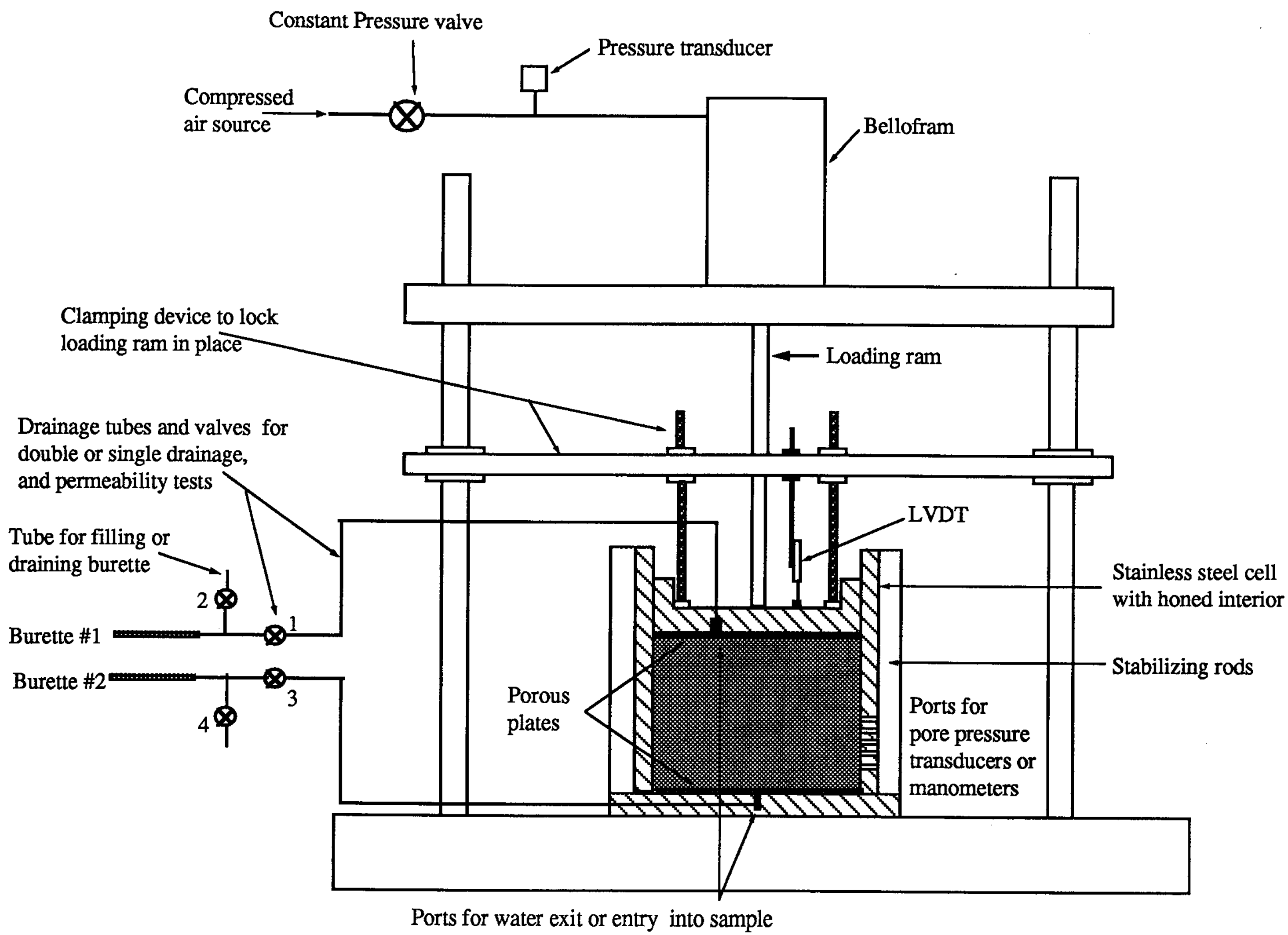


Figure A.3 Slurry Consolidometer

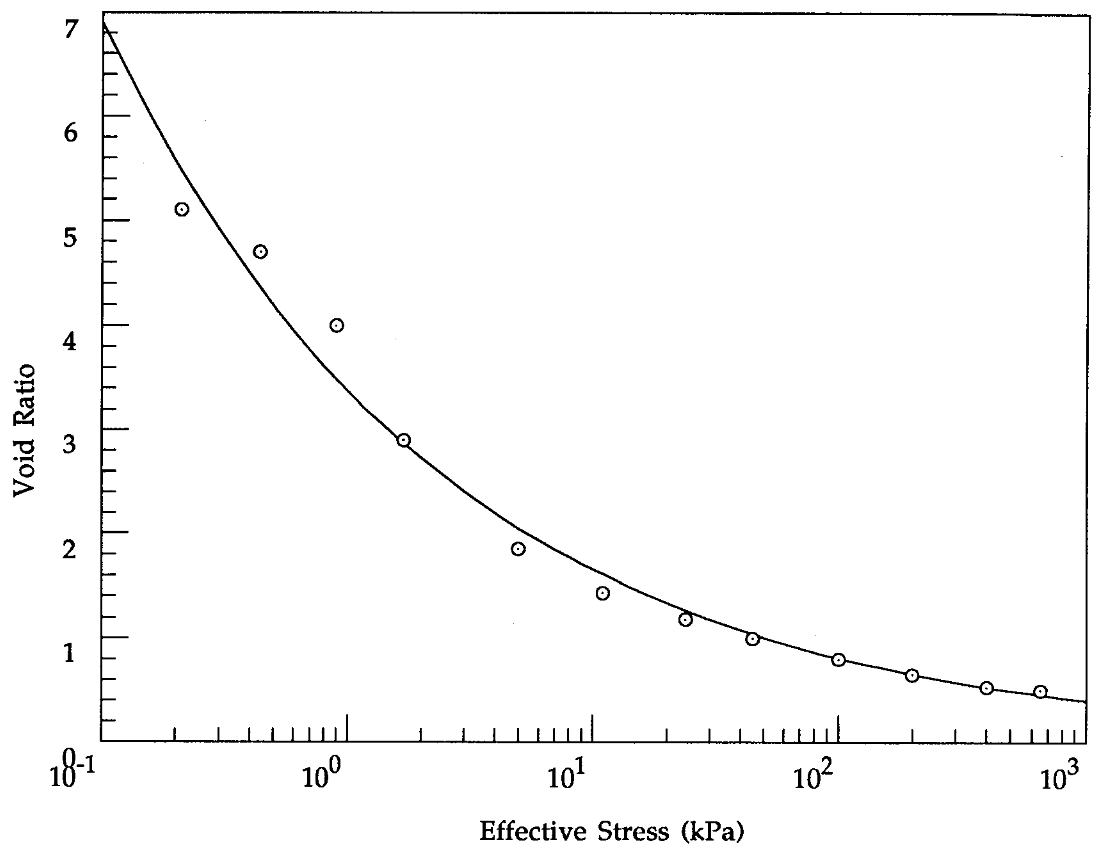


Figure A.4 Compressibility of Oil Sands Fine Tails
(Pollock, 1988)

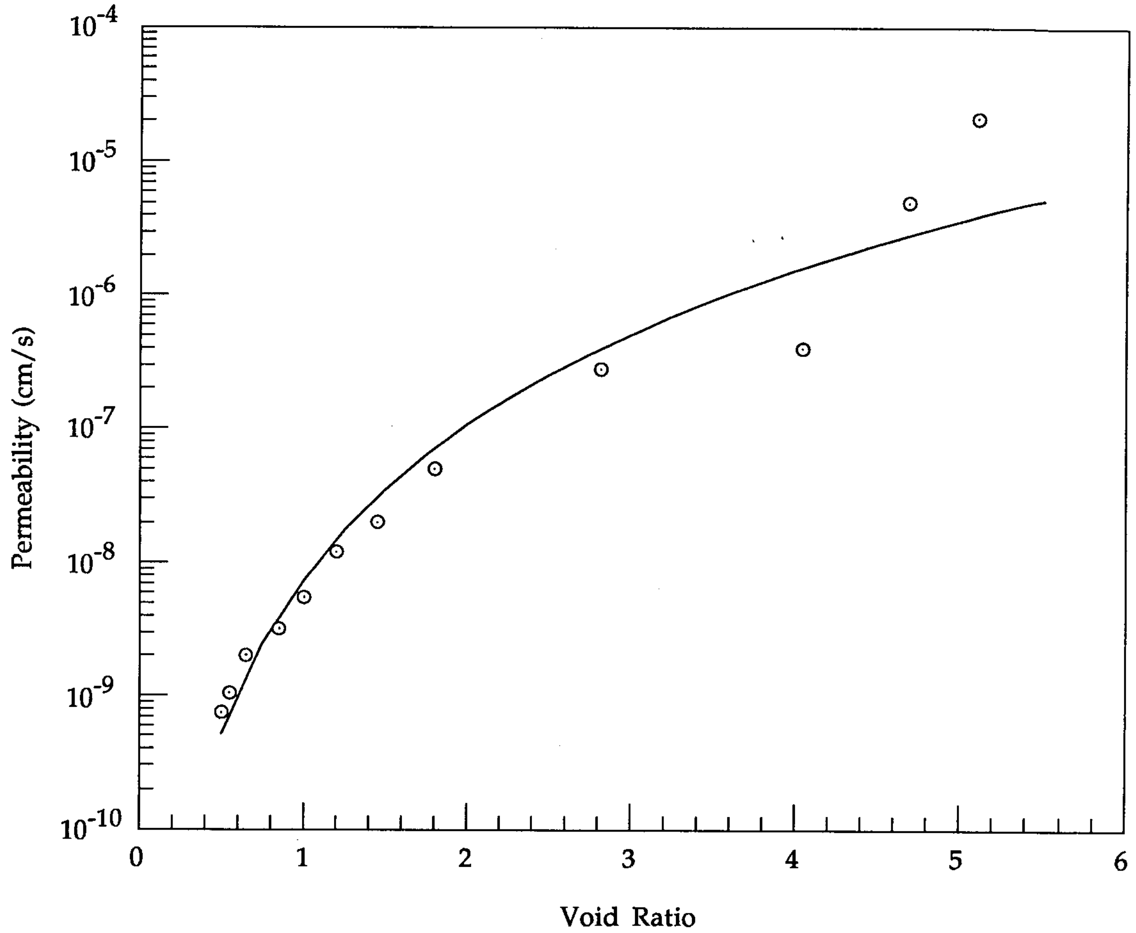


Figure A.5 Permeability of Oil Sands Fine Tails
(Pollock, 1988)

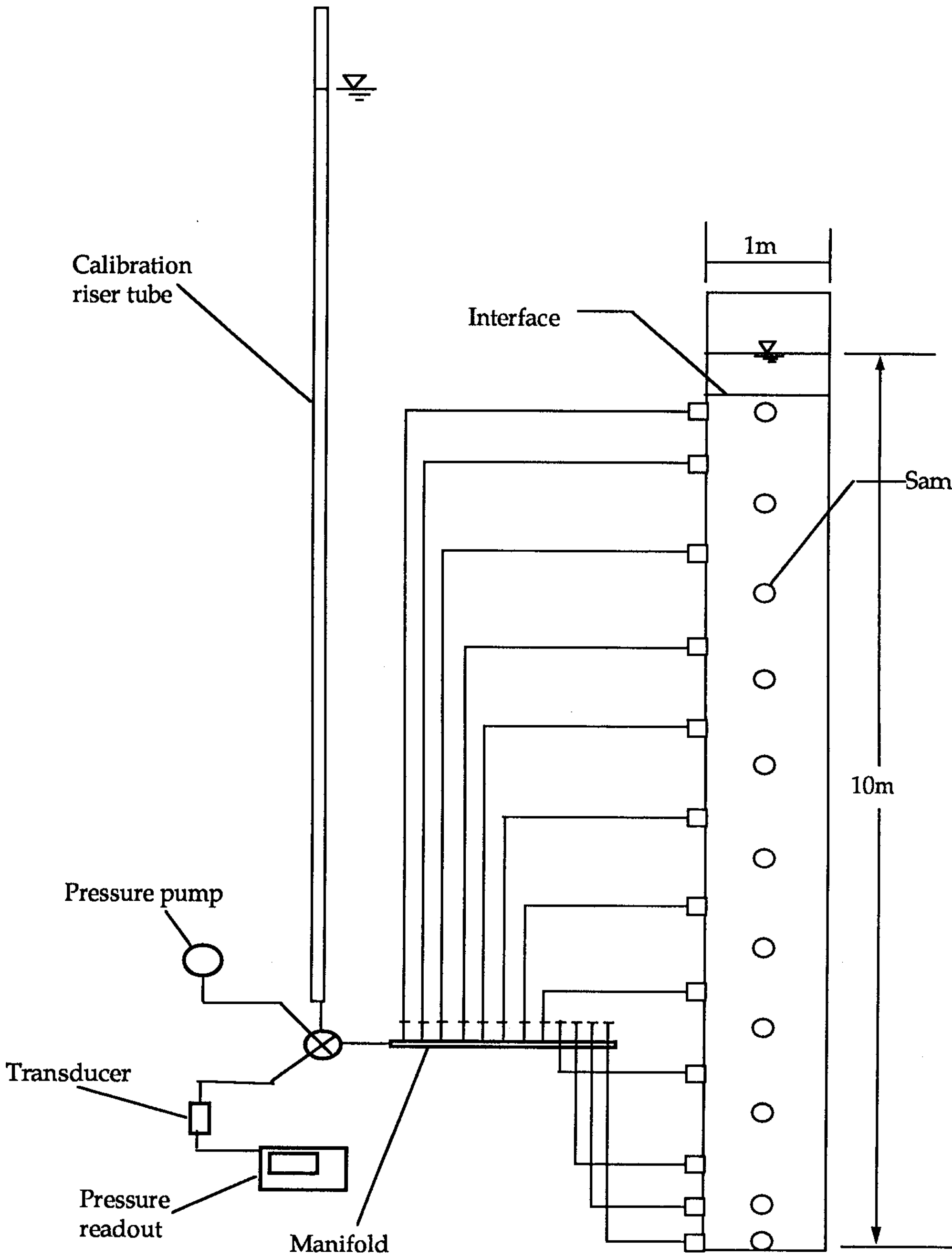


Figure A.6 Ten Metre Standpipe

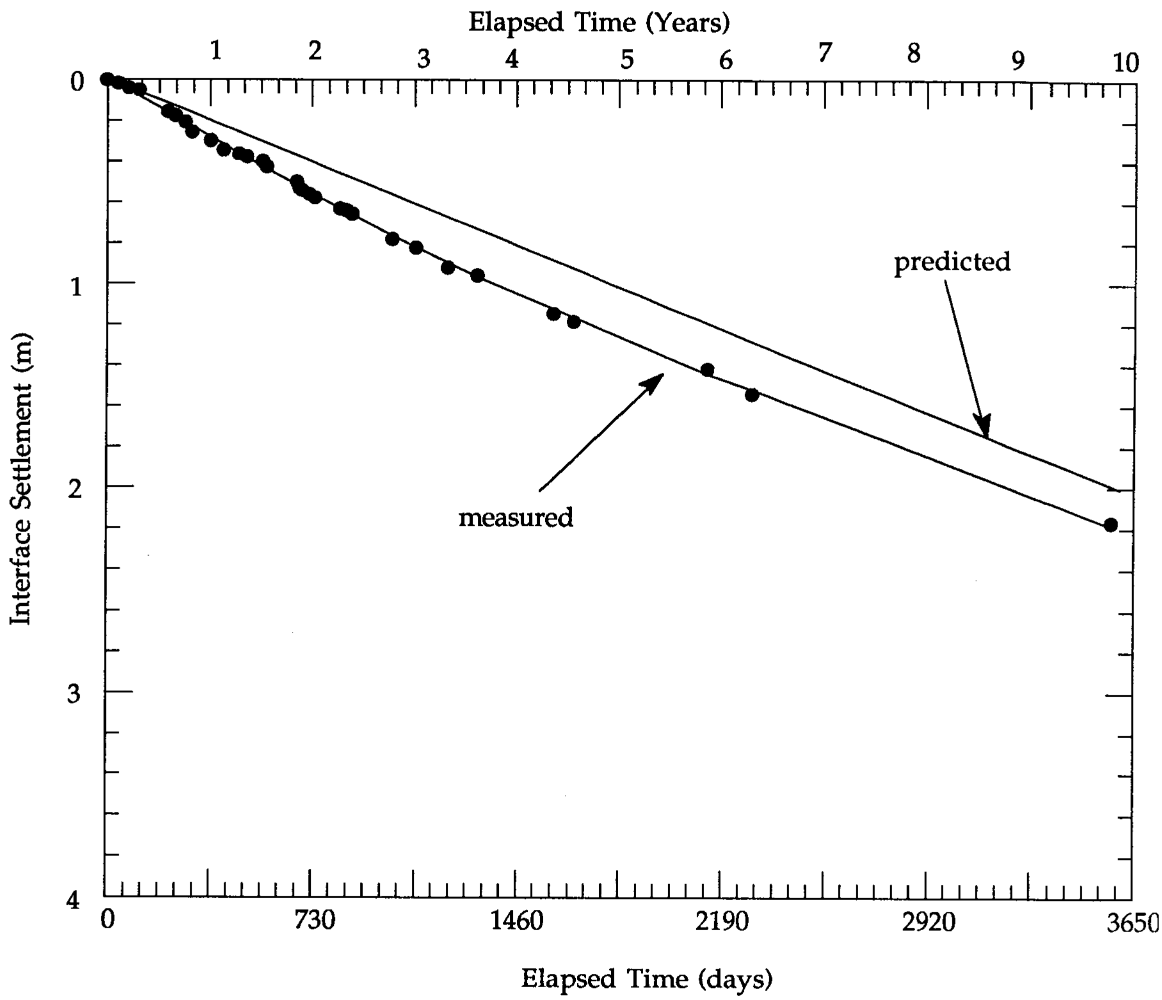


Figure A.7 Comparison of Measured and Predicted Consolidation

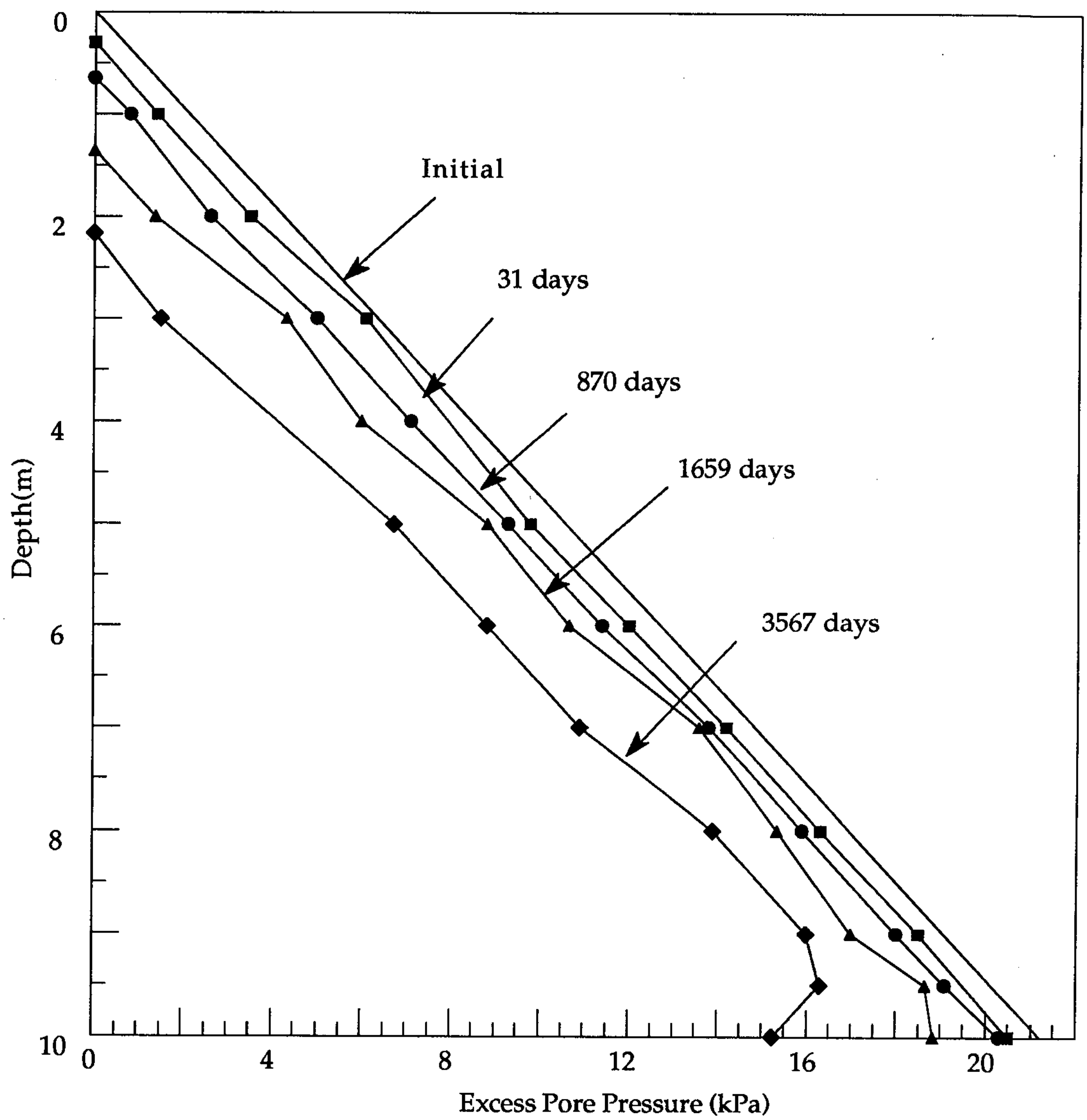


Figure A.8 Variation of Pore Pressure with Time

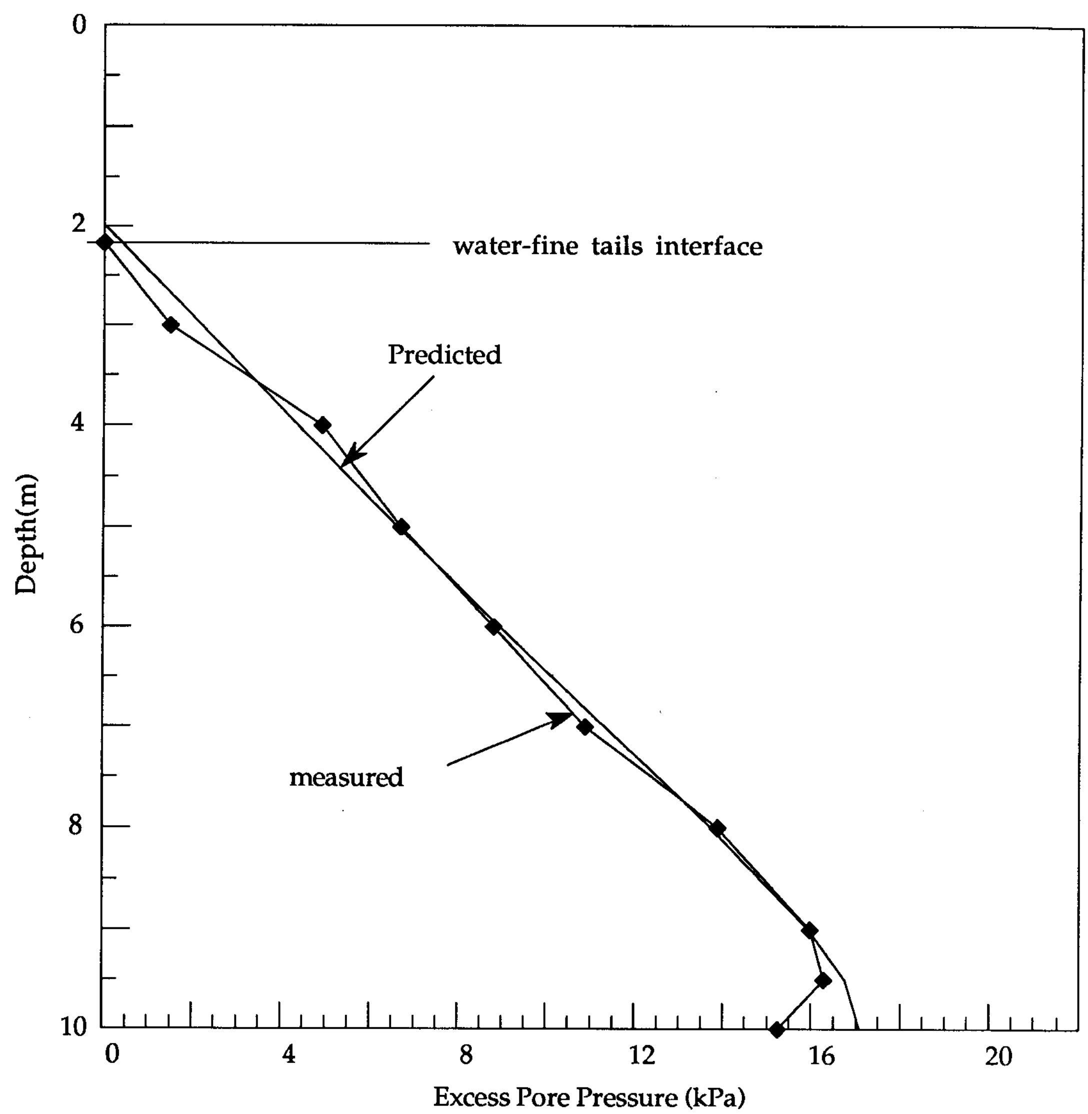


Figure A.9 Predicted and Measured Pore Pressure Profiles after 10 years

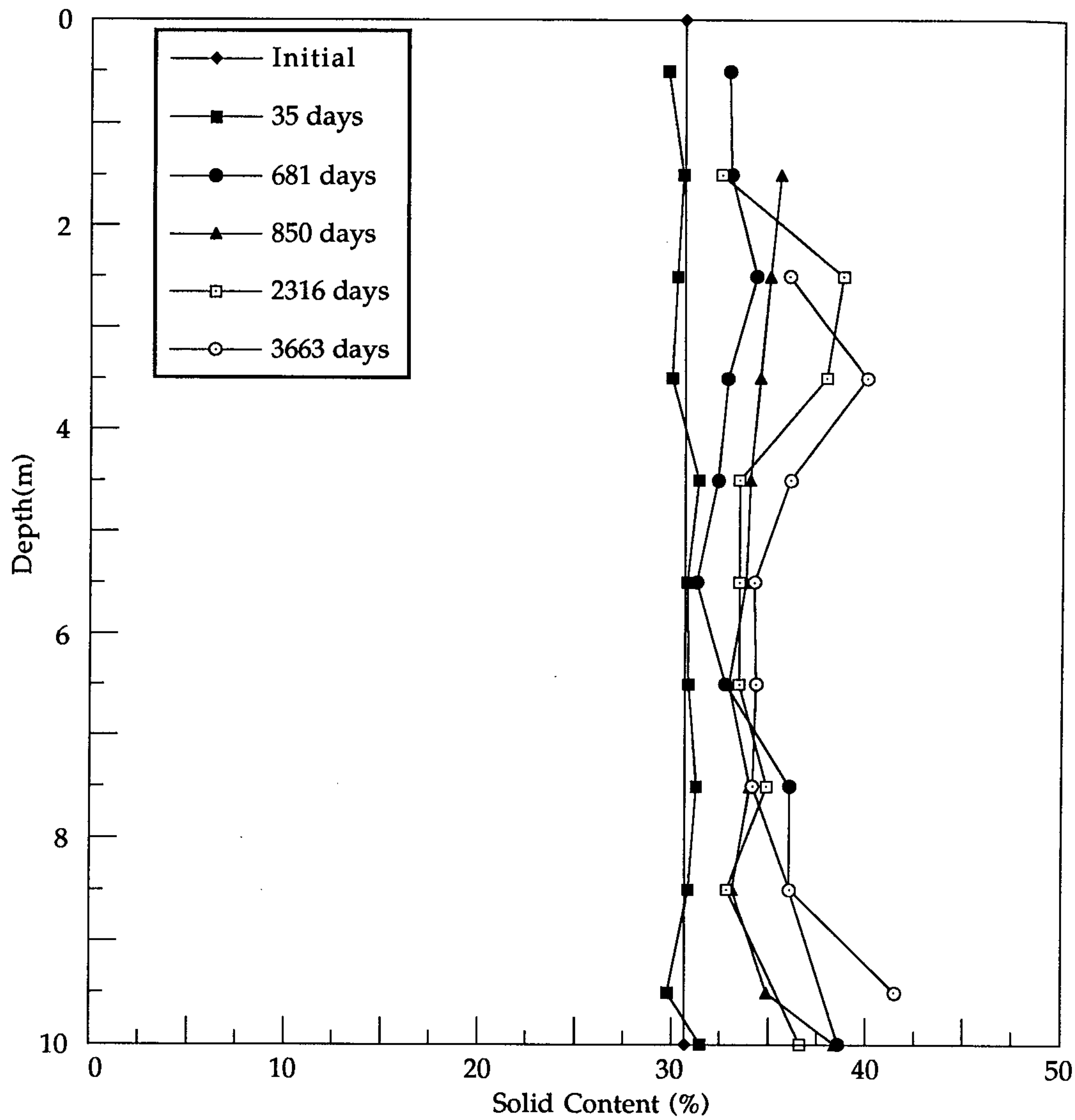


Figure A.10 Variation of Solids Content with Time

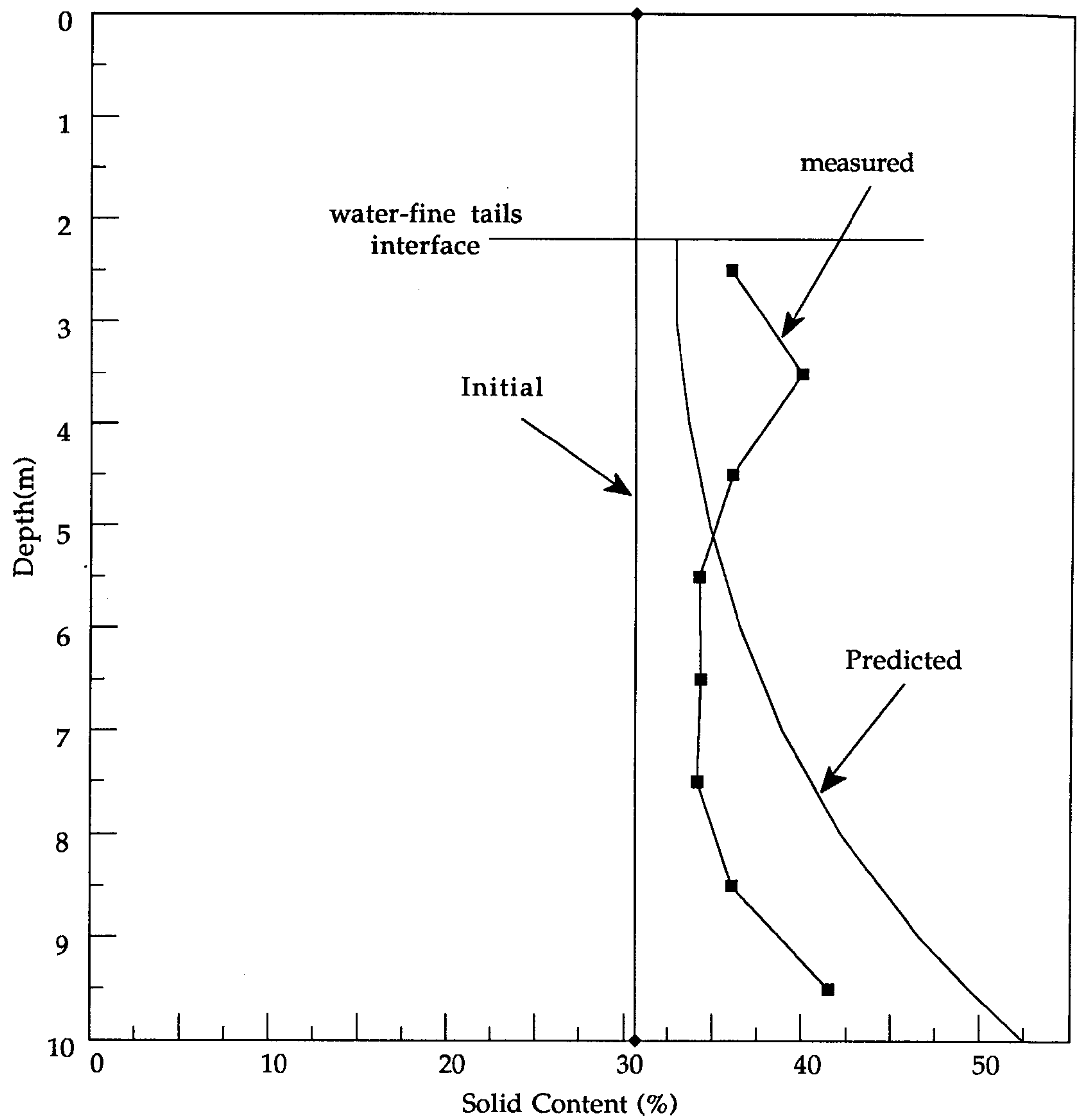


Figure A.11 Predicted and Measured Solids Content Profiles after 10 years

APPENDIX B: CONSOLIDATION & STRENGTH TESTING : APPARATUS, CALIBRATION AND TEST RESULTS

Consolidation Tests

Equipment Details

Diaphragm Air Cylinder

To transfer the air pressure from the air source to the loading ram, a diaphragm air cylinder from Bellofram Corporation was used. The particular models used had the following specifications.

Type:	SS
Size:	36
Series:	F
Rod:	BP
Bore:	6.8 in
Stroke:	6.0 in

Detailed specifications as well as an explanation of how the diaphragm works are available from the manufacturer.

Geotextile Filter

A melded geosynthetic fabric by ICI fibres was used. The fabric had the following manufacturer given specifications.

Fabric Name:	Terram 1500
Thickness:	1 mm
Porometry:	0.076 mm
Permeability:	0.035 m/s

Additional Specifications are available from the manufacturers.

Surface Displacement Measuring Equipment

The travel of the top caps was measured by a Hewlett Packard Linear varying Displacement Transducer (LVDT). The model for both consolidometers was a 24 DC DT-1000 which had a 50 mm displacement range. The LVDTs were calibrated using a high precision micrometer, the results of which are shown in Figure C.1 and C.2. specifications of the product is available from the manufacturers.

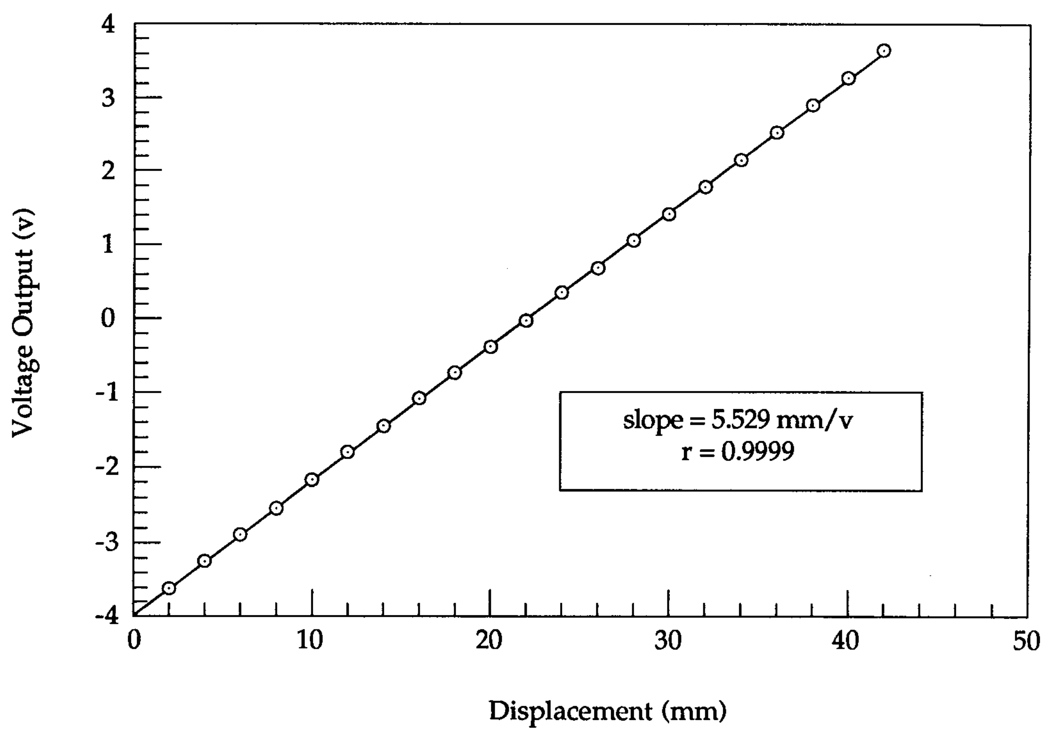


Figure B.1 Calibration of LVDT # 1

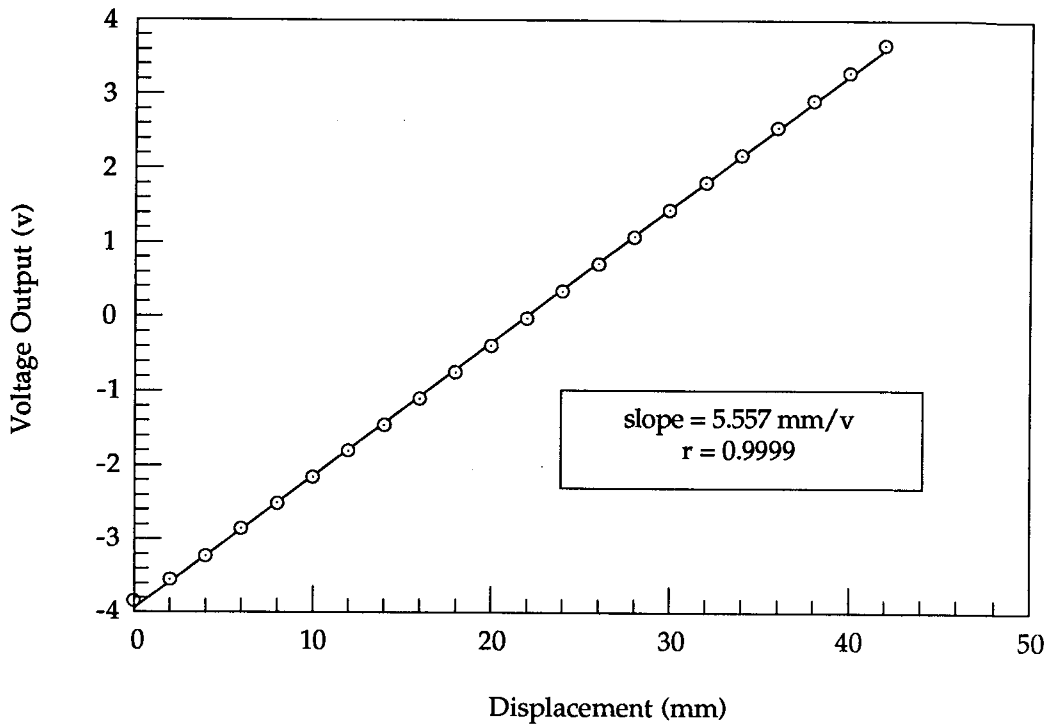


Figure B.2 Calibration of LVDT # 2

Load Cell

The load cell was used to regulate the air pressure applied to the cell. The load cell used was made at the University of Alberta. The calibrations of the load cells were done with the deadweight tester.

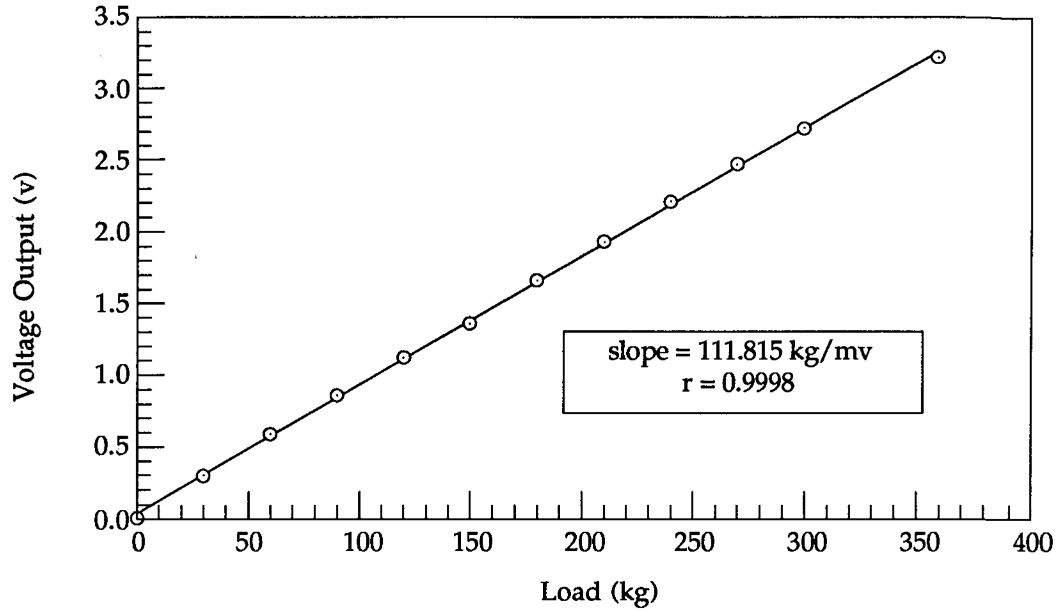


Figure B.3 Calibration of Load Cell #1

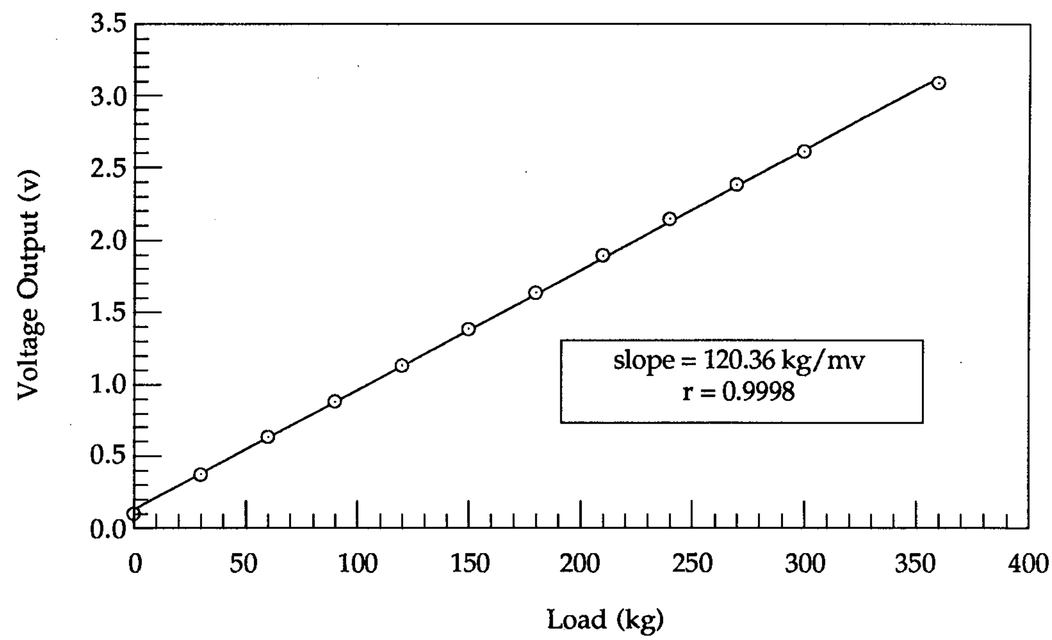


Figure B.4 Calibration of Load Cell #2

Pore Pressure Measuring Equipment

Consolidometers were equiped with 5 pore pressure ports each. For one consolidometer, lines from the ports led to Whitey ball valves and then to a Validyne Δp pressure transducer as shown in Figure C.5. For the second consolidometer, lines form the ports led to a series of valves and then to a pressure transducer as shown in Figure C.6. Before a series of pore pressure measurements were taken, a quick calibration was done to ensure that the related equipment was functioning properly. Figure C.7 shows the simple set up for this procedure.

The pressure transducer used in test #1 was Validyne Engineering Corporation multi range Δp pressure transducer, model DP 103-10. This type of transducers allows for different pressure ranges to be used with only one transducer. This is accomplished by substituting different range disphragms into the transducer when required. The diagram 8-34 was used in this research.The other pressure transducer used in test #2 was by Durham Instruments; model No. P707-0025-IOMO, 0-10 psi range. The calibrations of transducers were done with the Deadweight Tester Pressure Balance from Pressurements Ltd.(U.K.) type M1900-3. The results of the calibration for the two diaphragms used are shown in Figures C.8 and C.9.

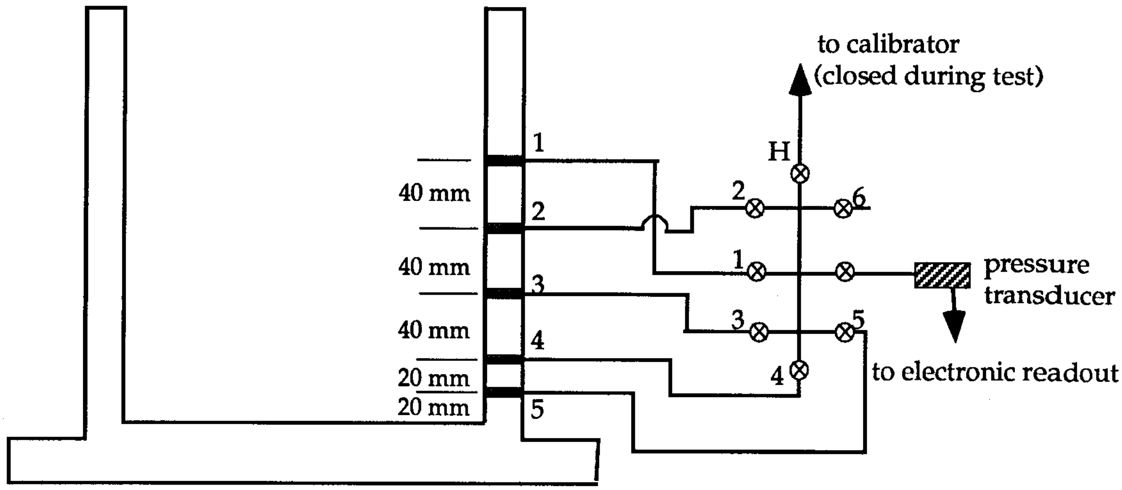


Figure B.5 Pore Pressure Measurement Setup #1

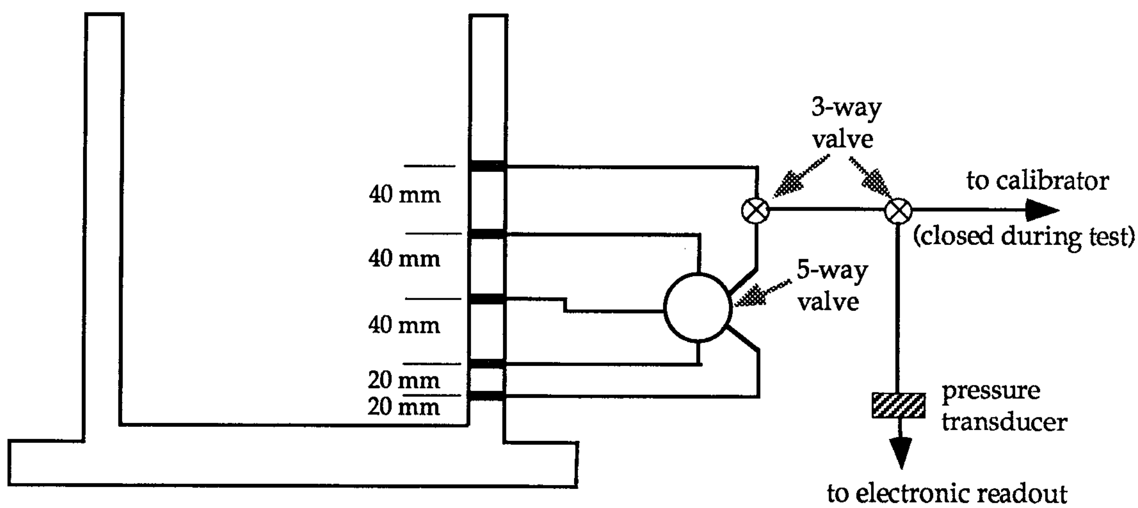


Figure B.6 Pore Pressure Measurement Setup #2

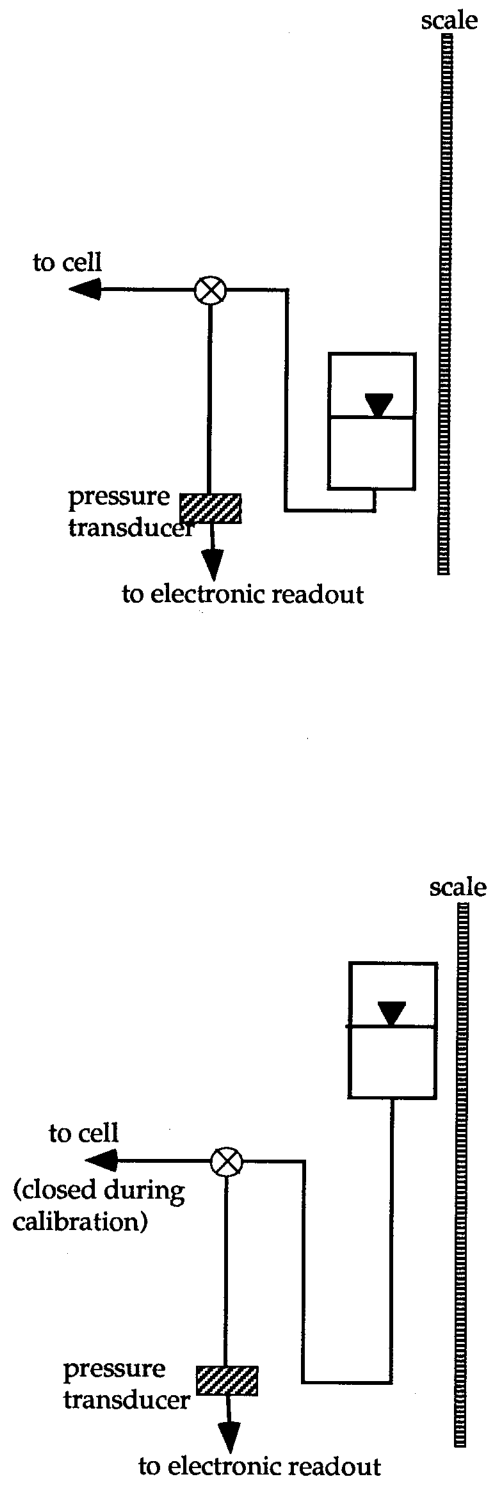


Figure B.7 Calibration Setup

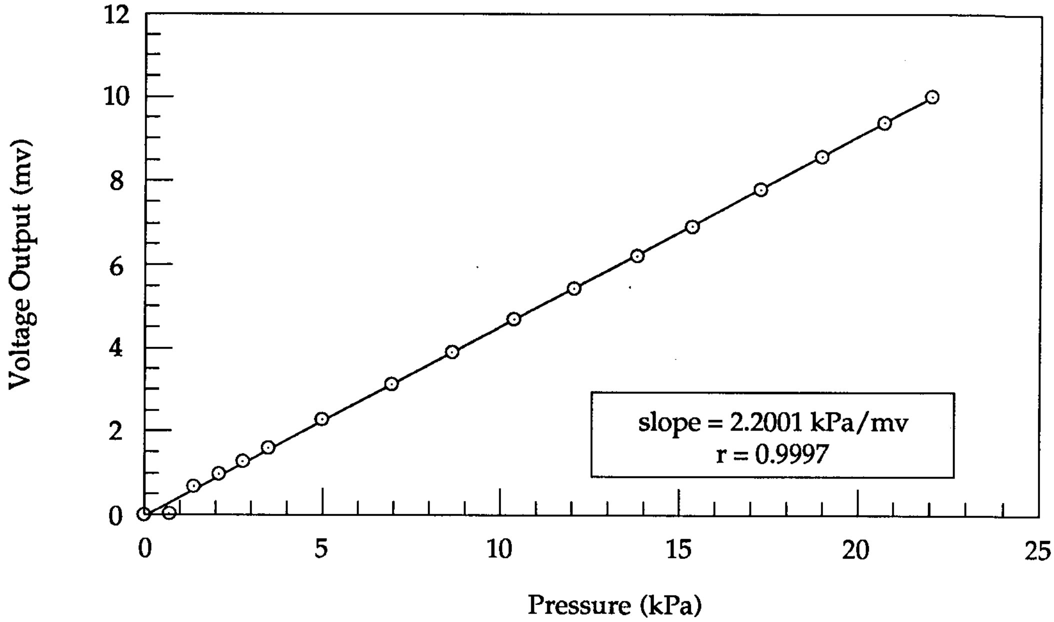


Figure B.8 Calibration of the Validyne Pressure Transducer #1

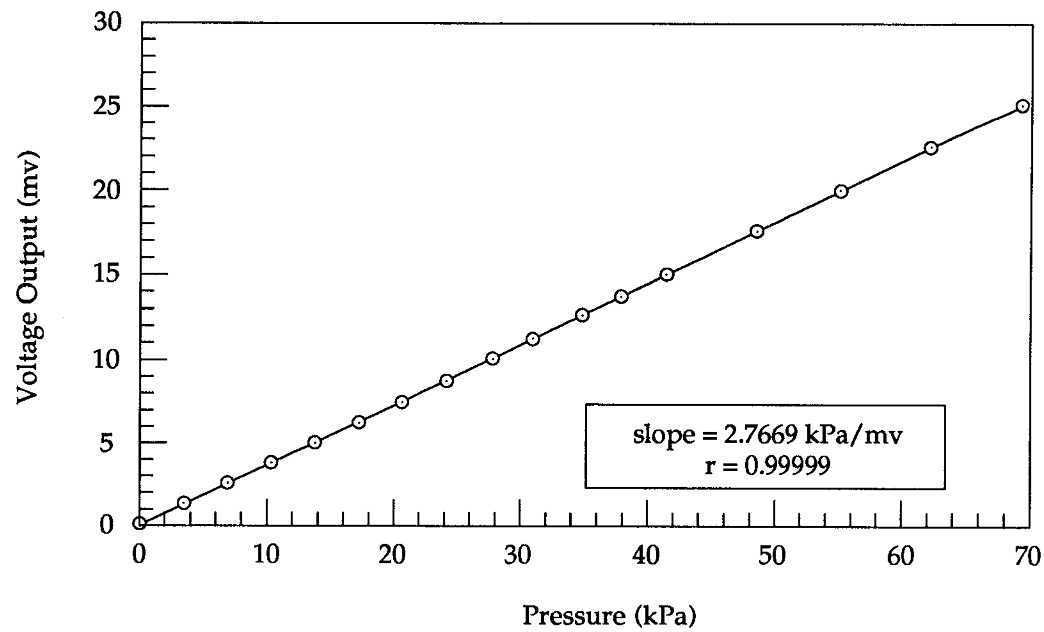


Figure B.9 Calibration of Pressure Transducer #2

Table B.1 Permeability Test Results - Summary - 25% Initial Solids

Trials	Void ratio	Hydraulic gradient	K (cm/s)
1	6.47	0.09	7.00E-05
		0.17	7.20E-05
2	6.1	0.20	3.11E-05
		0.17	3.66E-05
3	5.92	0.16	1.22E-05
		0.20	1.50E-05
		0.17	1.00E-05
4	5.53	0.20	1.13E-05
		0.19	1.08E-05
		0.22	9.90E-06
5	6.25	0.19	6.67E-06
		0.20	7.70E-06
		0.23	5.84E-06
6	3.48	0.19	1.60E-06
		0.22	1.48E-06
		0.28	1.24E-06
7	2.04	0.37	4.50E-08
		0.43	3.78E-08
		0.43	2.80E-08
8	1.49	0.27	2.99E-08
		0.34	2.32E-08
		0.46	1.62E-08
		0.56	1.54E-08
9	1.22	0.27	1.00E-08
		0.35	1.00E-08
		0.54	9.95E-09
10	1.02	0.35	4.20E-09
		0.43	3.90E-09
		0.53	4.38E-09
		0.58	3.20E-09

Table B.2 Permeability Test Results - Summary - 20% Initial Solids

Trials	Void ratio	Hydraulic gradient	K (cm/s)
1	7.69	0.10	2.41E-04
		0.15	1.45E-04
		0.09	1.00E-04
2	7.23	0.09	2.48E-05
		0.18	3.12E-05
		0.16	2.88E-05
3	6.98	0.12	5.62E-05
		0.18	3.67E-05
		0.15	1.09E-04
4	6.57	0.11	1.94E-05
		0.17	2.89E-05
		0.18	6.36E-05
		0.18	7.03E-05
5	5.37	0.20	2.98E-05
		0.20	3.34E-05
		0.18	3.20E-05
6	4.01	0.18	3.06E-06
		0.24	3.24E-06
		0.27	2.00E-06
		0.29	2.63E-06
7	2.54	0.27	7.75E-08
		0.54	5.27E-08
		0.54	5.39E-08
8	1.47	0.30	2.42E-08
		0.45	9.92E-09
		0.54	8.50E-09
9	1.36	0.30	1.80E-08
		0.39	1.80E-08
		0.58	1.70E-08
10	1.17	0.38	6.10E-09
		0.58	3.83E-09
		0.60	5.79E-09

Cavity Expansion Tests

Equipment Details

<u>LC-5000 Syringe Pump</u>

To inject the fluid at a preset rate, a syringe pump was used. The syringe barrel and piston are made of stainless steel and pump seals are graphite filled Teflon. The model used in the study had the following specifications.

Flow rate range: 0.1 ml/hr to 400 ml/hr
Capacity: 500 ml

Dimensions
Pump:
width: 26 cm
depth: 42 cm
height: 99 cm

Refill rate: 0 to 100 ml/min
Dead volume: 5.27±0.2 ml

Detailed specifications as well as an explanation of how the syringe pump works are available from the manufacturer.

<u>Cavity Pressure Measuring Equipment</u>

The pressure transducer used were Validyne Engineering Corporation multi range Δp pressure transducer, model DP 103-10. The diaphragms used in this tests were 8-26 and 8-10 for 3.5 kPa and 0.862 kPa range transducers respectively. The results of the calibration for the two diaphragms are shown in Figure B.1 and B.2

Additional details are available from the manufacturers.

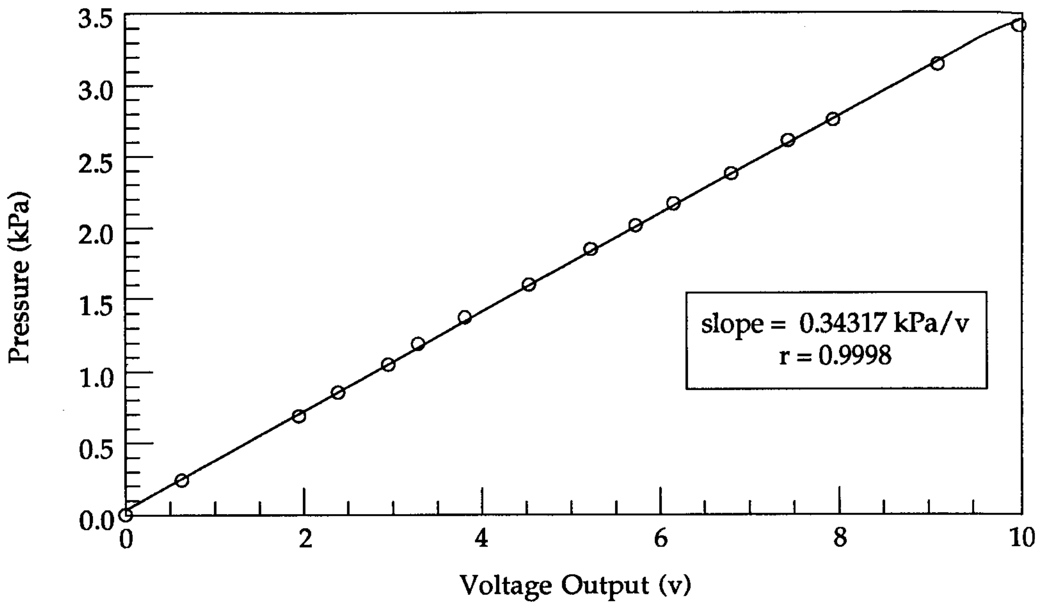


Figure B.10 Calibration of Pressure Transducer
(Plate no. 8-26, Range 0-3.5 kPa)

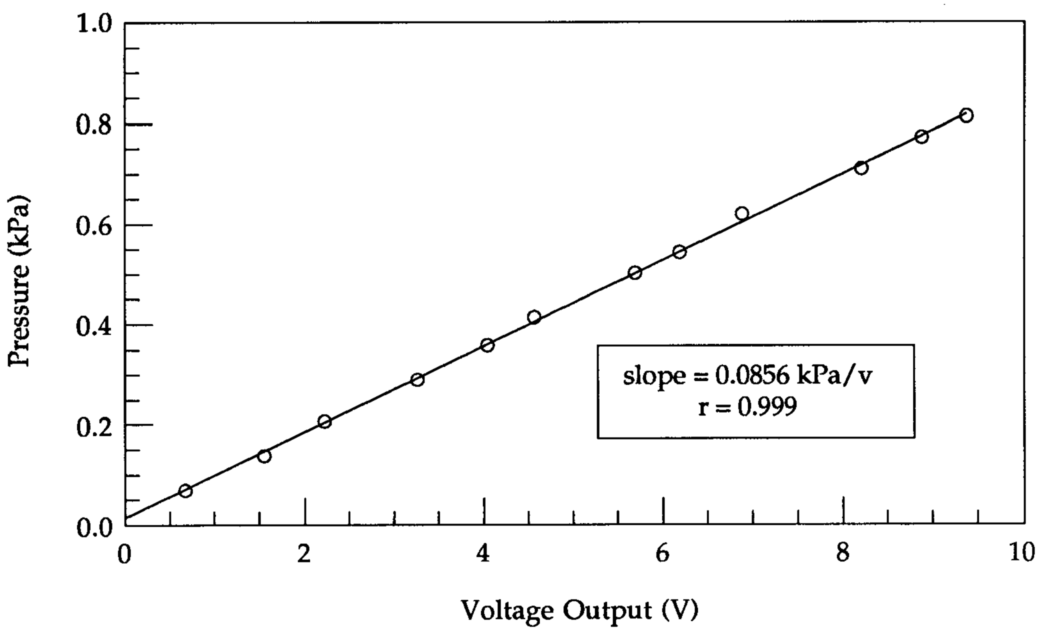
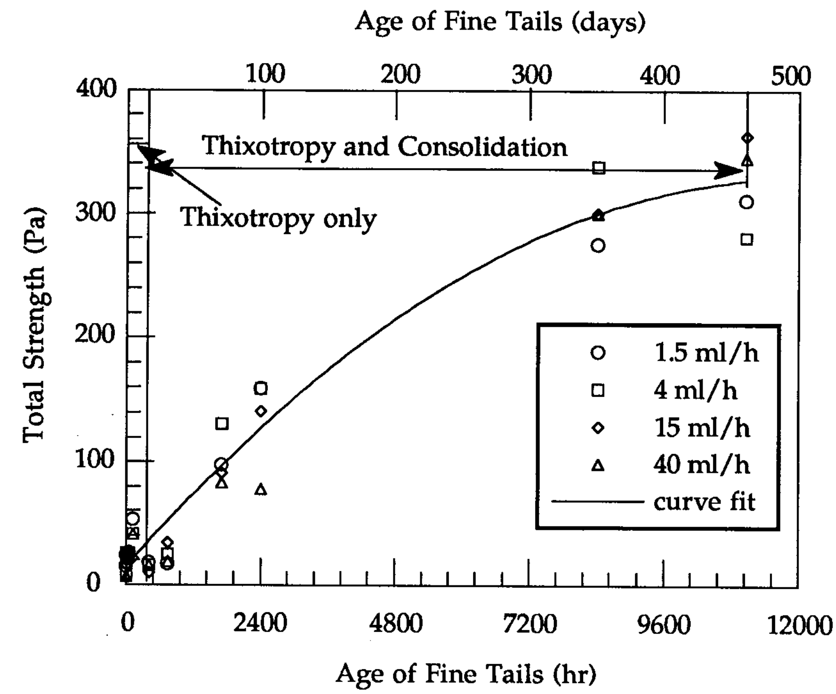


Figure B.11 Calibration of Pressure Transducer
(Plate no. 8-10, Range 0-0.862 kPa)



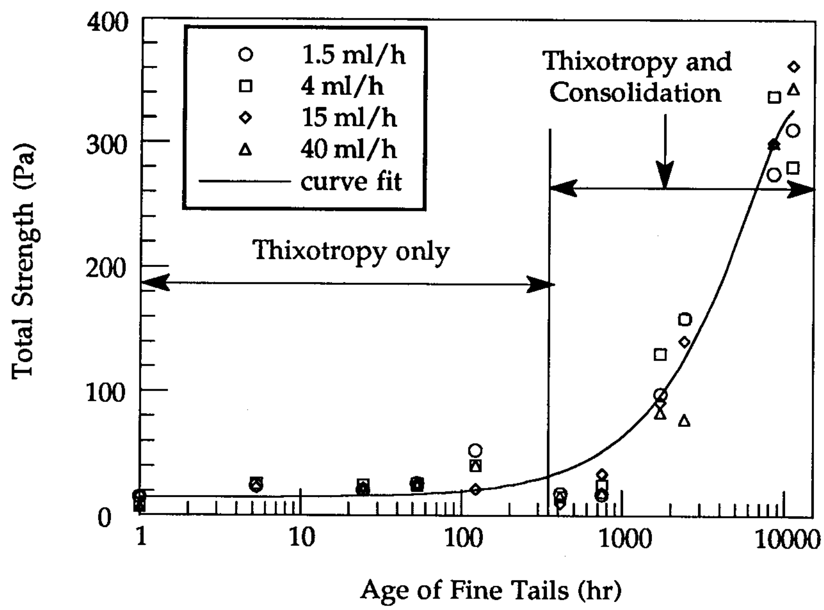
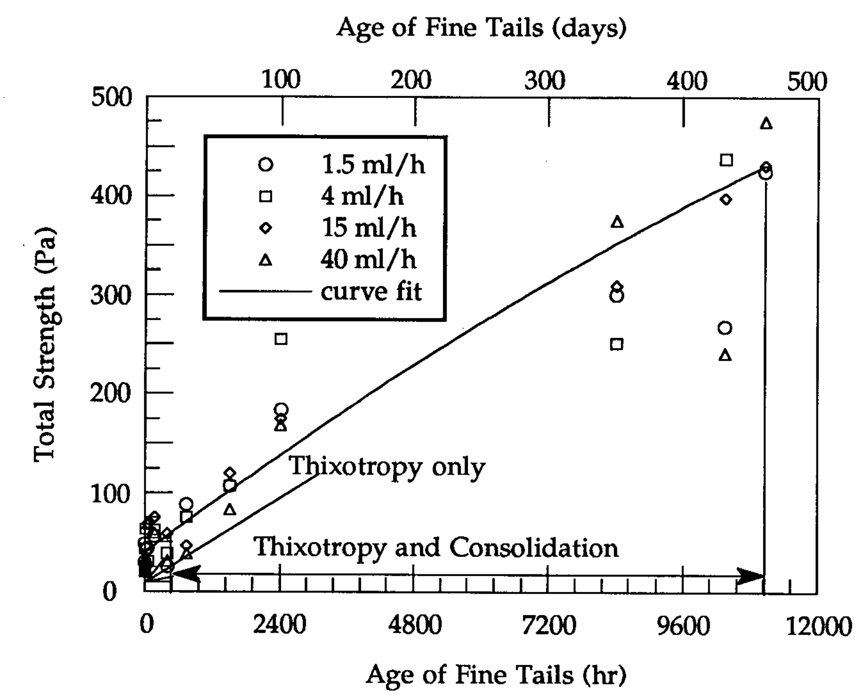


Figure B.12 Cavity Expansion Test Results
(Initial Water Content of 400%)



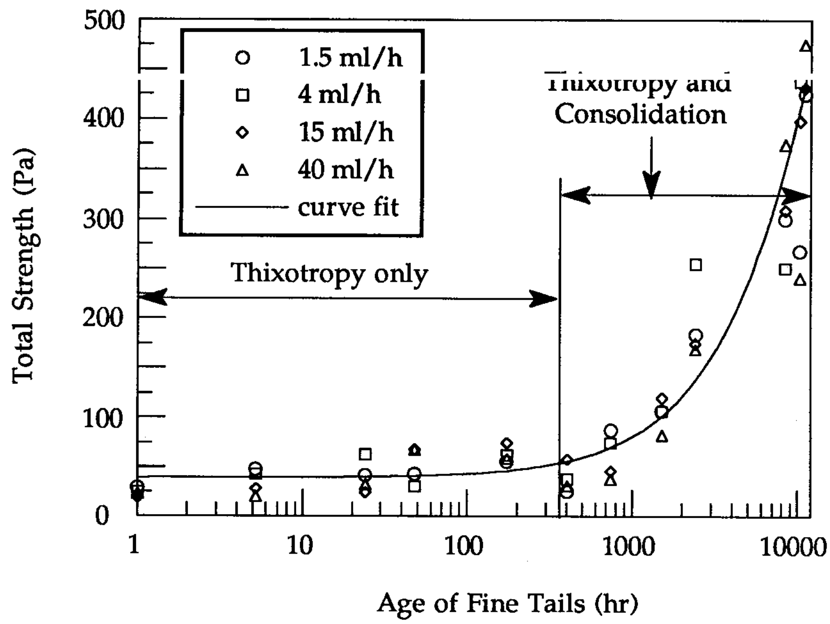
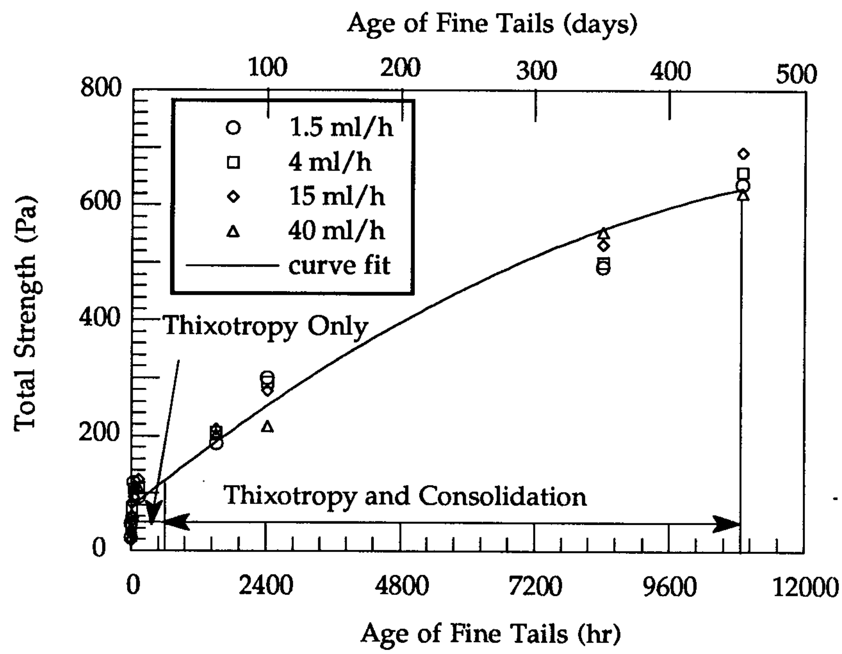


Figure B.13 Cavity Expansion Test Results
(Initial Water Content of 300%)



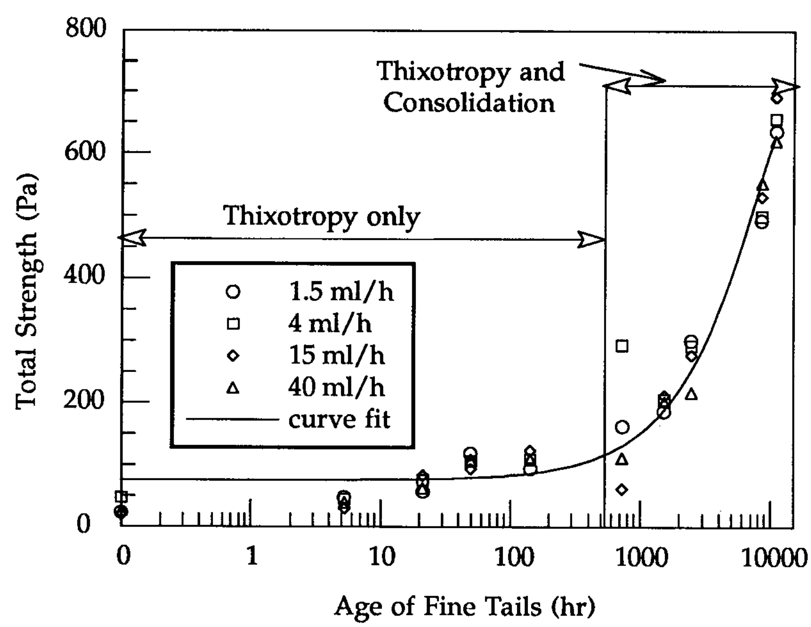


Figure B.14 Cavity Expansion Test Results
(Initial Water Content of 233%)

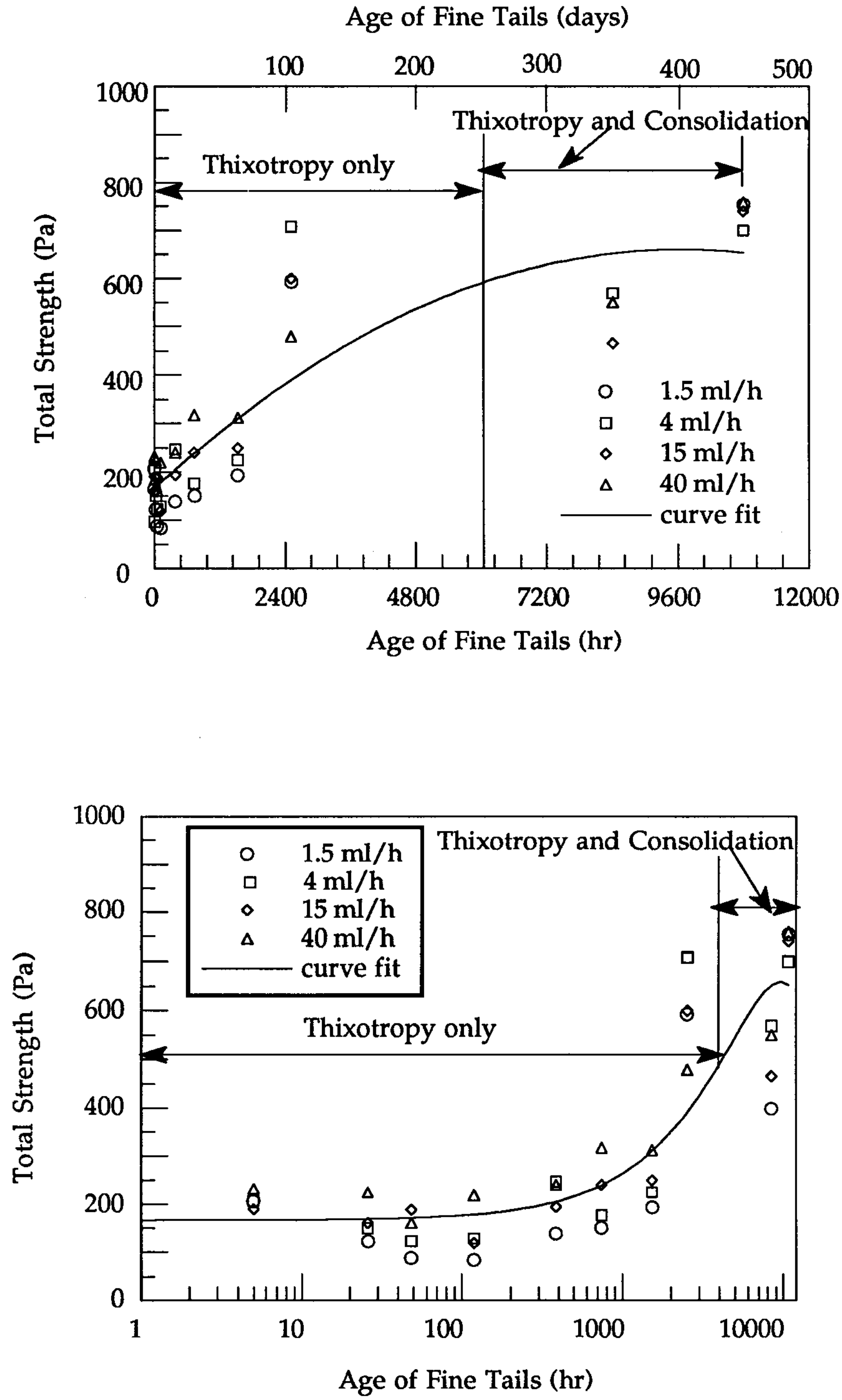
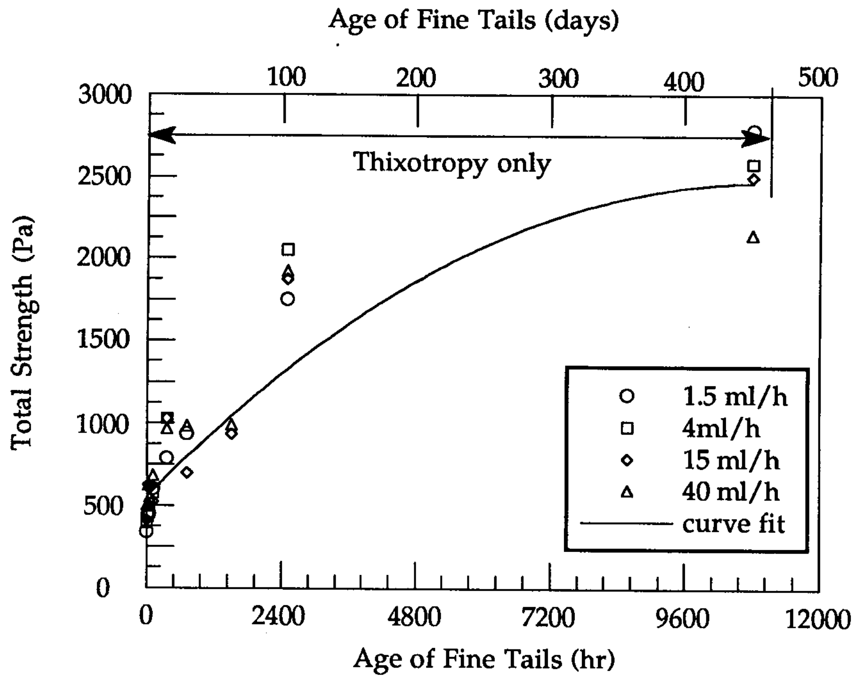


Figure B.15 Cavity Expansion Test Results
(Initial Water Content of 150%)



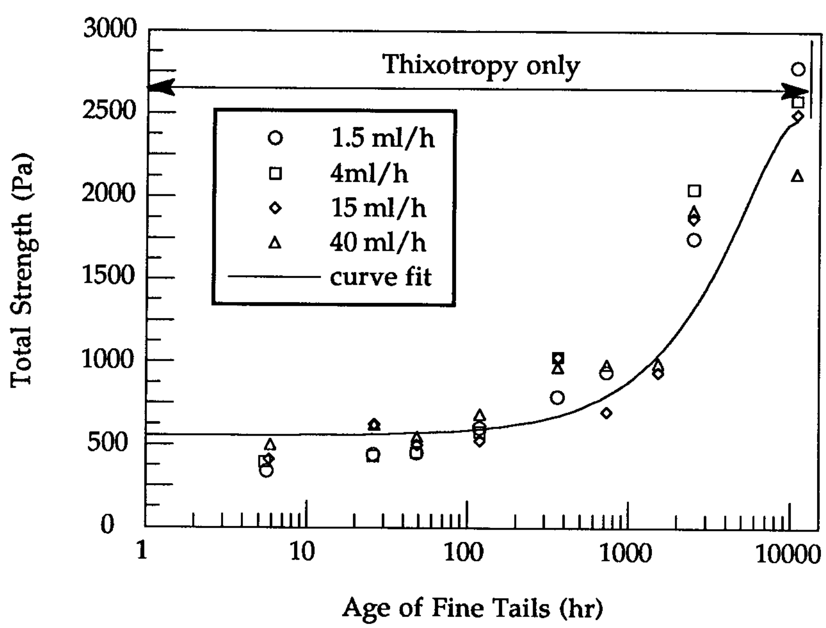


Figure B.16 Cavity Expansion Test Results
(Initial Water Content of 100%)

Two Metre Standpipe Test Results

Seven two metre high standpipe tests were performed. The following section describes the test results. The predictions were made as explained in Chapter 6. The sampling at the end of test was performed by taking the specimens using a large syringe, collecting specimens through the sample ports, and scooping the sample from the standpipe.

Table B.3 Material Properties in 2m Standpipes

Standpipe No.	Initial Height (m)	Initial Solids Content (%)	Fines Content (%)	Origin	Duration (days)
A	2	31.9	90	Syncrude	1200
1	1.993	25.67	96	Syncrude	840
2	1.996	21.38	96	Syncrude	835
3	1.995	24.88	90	Syncrude	830
4	1.985	26.4	89	Syncrude	580
5	1.997	23	86	Syncrude	580
6	1.8	27.95	89	Suncor	330

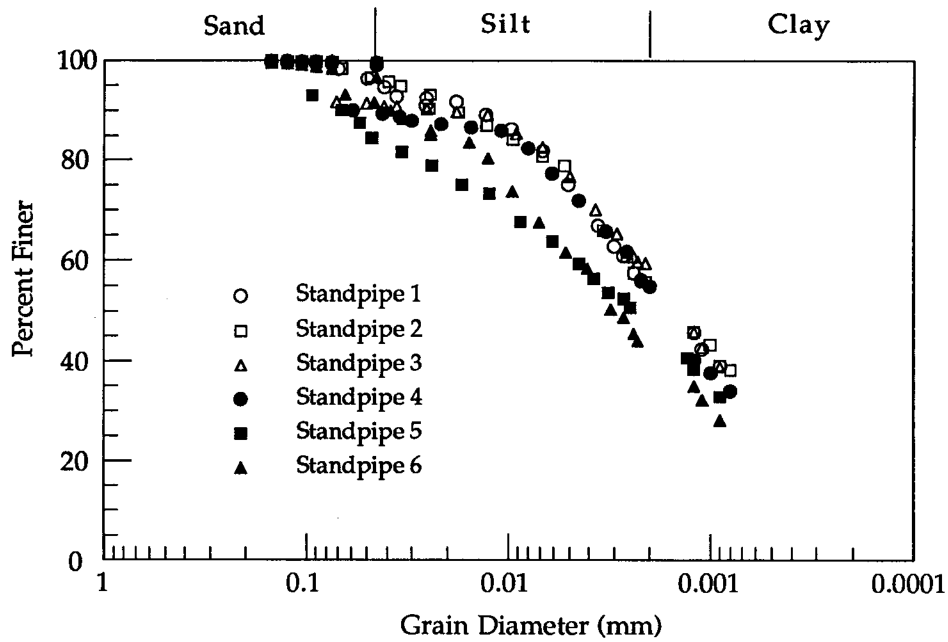


Figure B.17 Grain Size Distribution of 2m Standpipe material

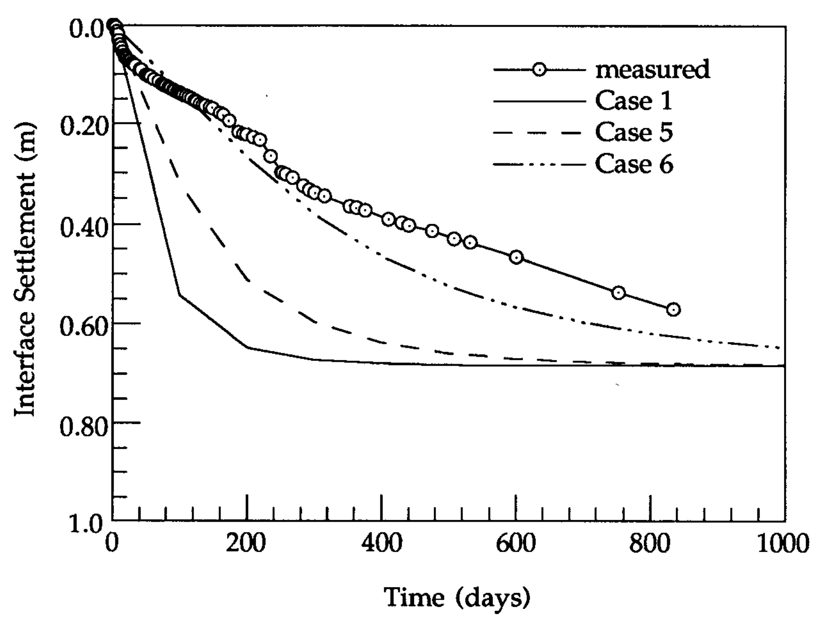


Figure B.18 Measured and Predicted Interface Settlement in Standpipe 2

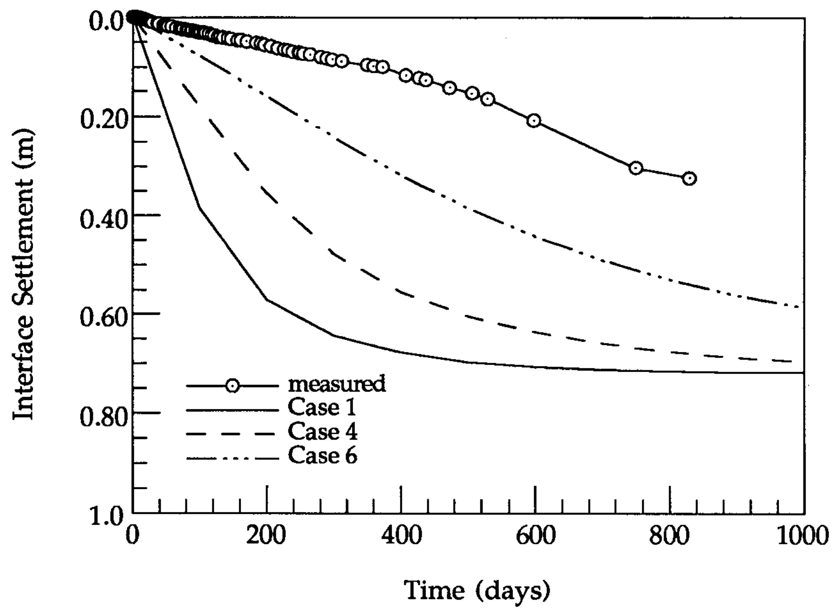


Figure B.19 Measured and Predicted Interface Settlement in Standpipe 3

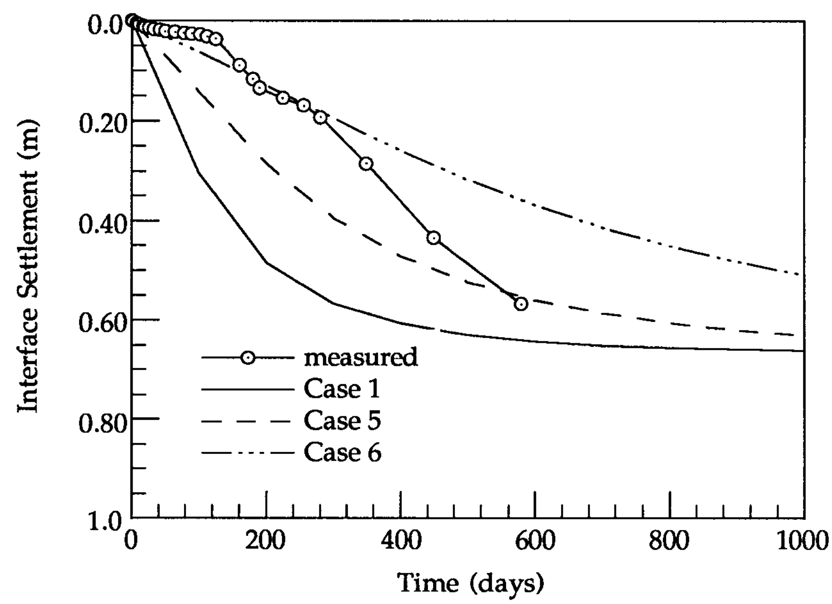


Figure B.20 Measured and Predicted Interface Settlement in Standpipe 4

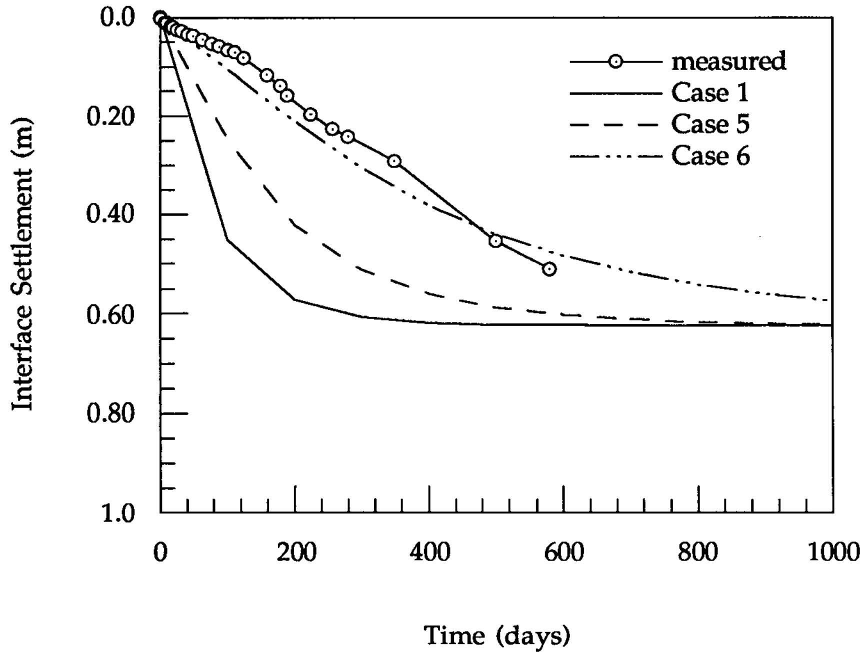


Figure B.21 Measured and Predicted Interface Settlement in Standpipe 5

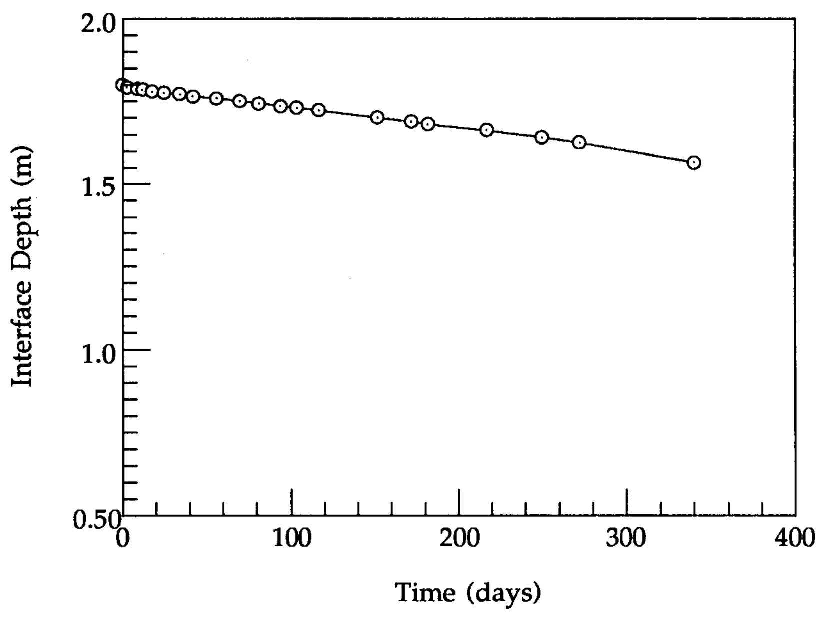


Figure B.22 Measured Interface Settlement in Standpipe 6

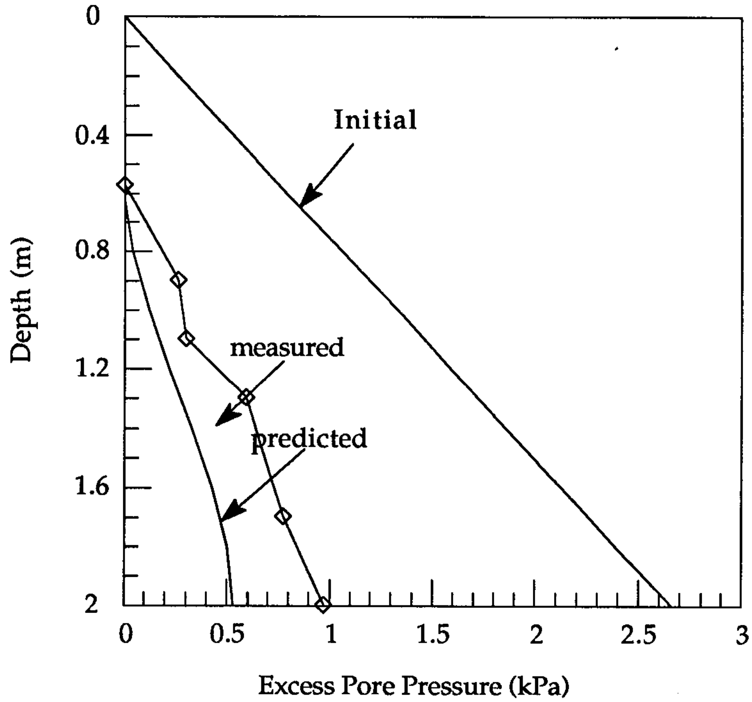


Figure B.23 Measured and Predicted Excess Pore Pressure in Standpipe 2 after 835 days

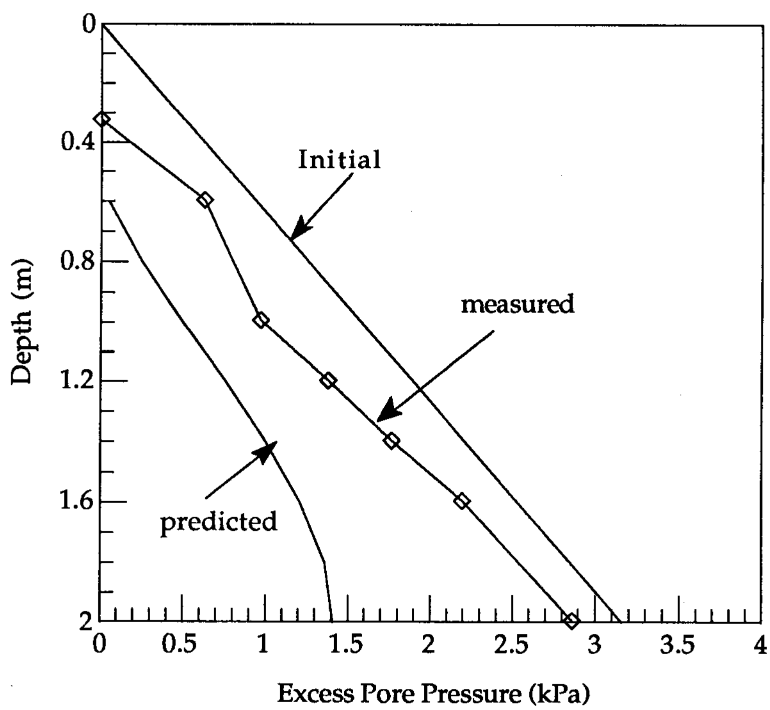


Figure B.24 Measured and Predicted Excess Pore Pressure in Standpipe 3 after 830 days

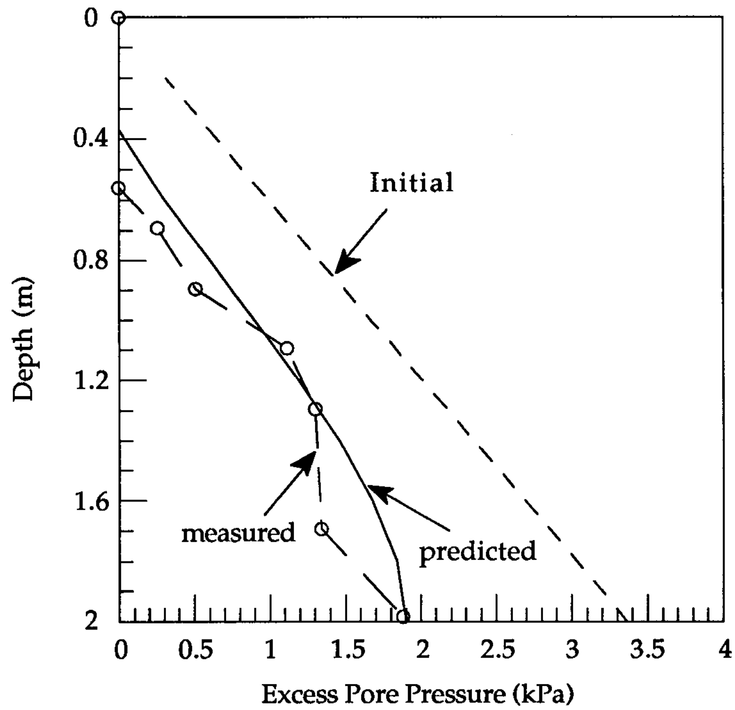


Figure B.25 Measured and Predicted Excess Pore Pressure in Standpipe 4 after 580 days

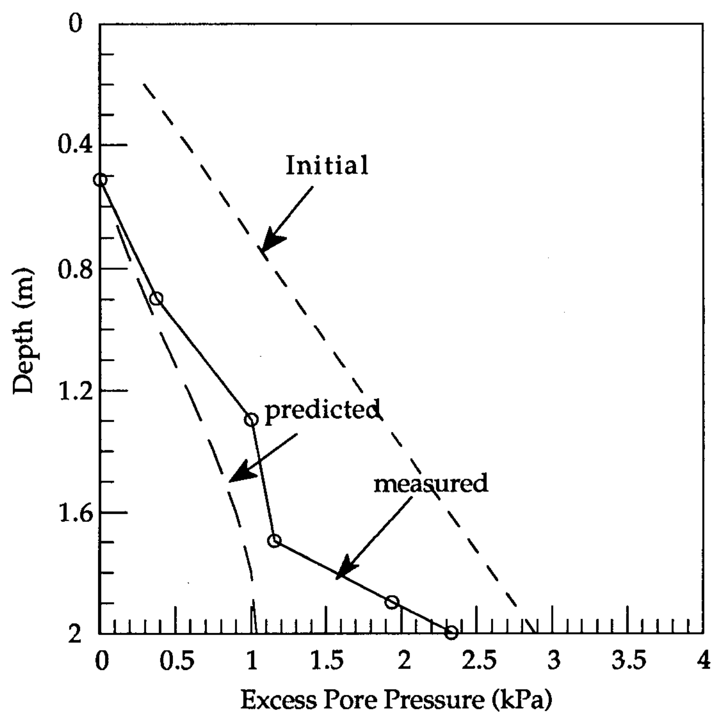


Figure B.26 Measured and Predicted Excess Pore Pressure in Standpipe 5 after 580 days

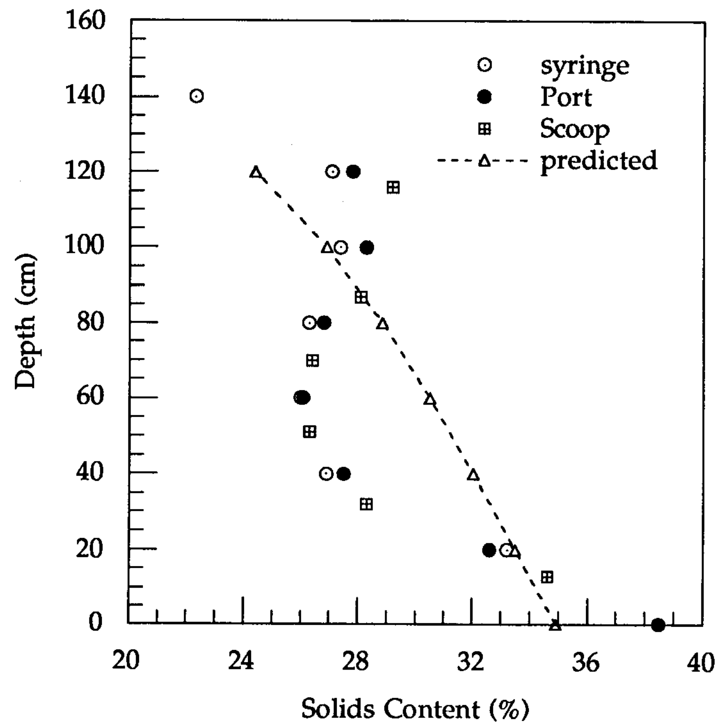


Figure B.27 Solids Content variation in Standpipe 2 After 835 days

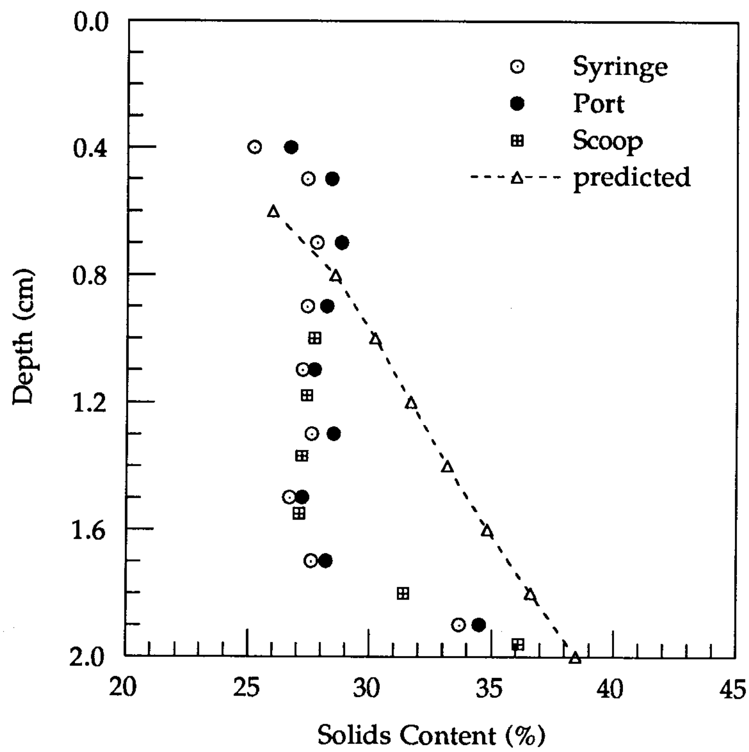


Figure B.28 Solids Content variation in Standpipe 3 After 830 days

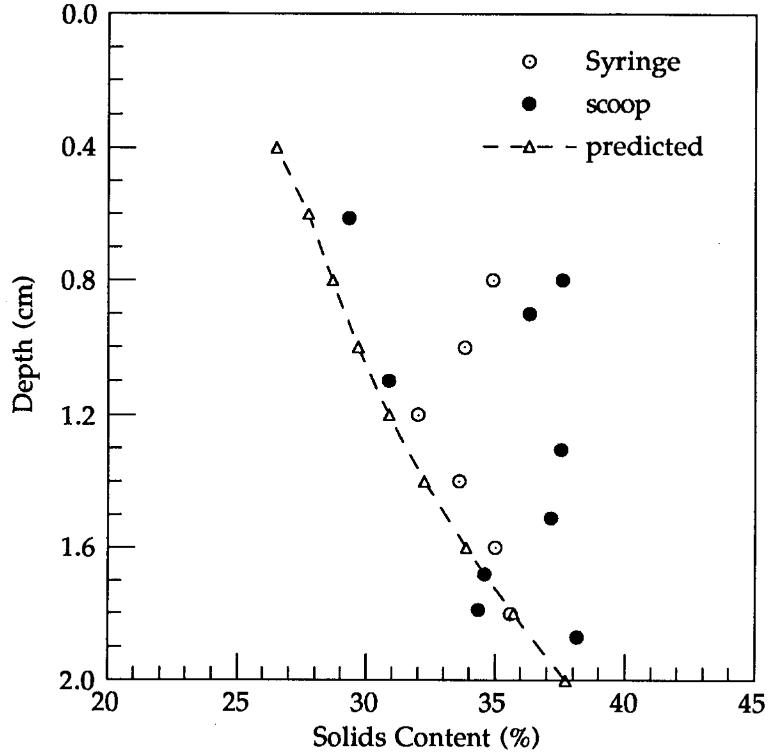


Figure B.29 Solids Content variation in Standpipe 4 After 580 days

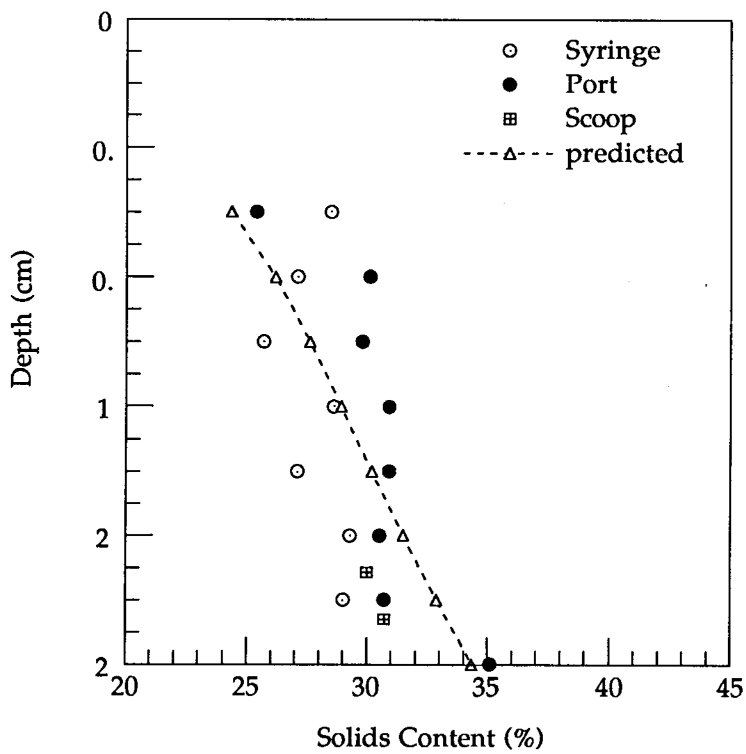


Figure B.30 Solids Content variation in Standpipe 5 After 580 days

CHECK LIST - retain for your records

To ensure your application is complete, ALL of the following items must be included in each application package.

[ ] 1. Application Form (page 5) **in triplicate**. Photocopies are acceptable.

[ ] 2. Answers to questions numbered 1 through 5 inclusive on page 6 **in triplicate**. Photocopies are acceptable.

[ ] 3. Filing fee: money order or cheque (Cdn $) made payable to Canadian Federation of University Women per application

SECTION I	$30.00
SECTION II	$25.00
SECTION III	$20.00
SECTION IV	$25.00

[ ] 4. Three letters of reference, each sealed in a separate envelope with the signature of the referee across the flap.

RETURN ALL APPLICATION PACKAGES TO:
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297 DUPUIS STREET
OTTAWA, ONTARIO K1L 7H8
Telephone (613) 747-8154

PLEASE RETAIN PAGES 1, 2, 3, 4, 6, AND 7 FOR YOUR FUTURE REFERENCE.

THE RESULTS OF THE COMPETITION WILL BE SENT BY MAY 31, 1996 TO CANDIDATES SUBMITTING APPLICATIONS.

PHOTOCOPY EXTRA FORMS IF REQUIRED