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**Permeability and volume change characteristics of bentonite-sand
mixes in a contaminant environment**

Wu, Yuan-Hsun, D.Eng.Sc.

New Jersey Institute of Technology, 1989

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PERMEABILITY AND VOLUME CHANGE CHARACTERISTICS
OF
BENTONITE-SAND MIXES
IN
A CONTAMINANT ENVIRONMENT

by
Yuan-Hsun Wu

Dissertation submitted to the Faculty of the Graduate School
of the New Jersey Institute of Technology in partial fulfillment
of the requirements for the degree of
Doctor of Engineering Science
1989

APPROVAL SHEET

Title of Thesis: Permeability and Volume Change Characteristics
of Bentonite-Sand Mixes in a Contaminant
Environment

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ABSTRACT

Title of Thesis: Permeability and Volume Change
Characteristics of Bentonite-Sand Mixes
in a Contaminant Environment

Yuan-Hsun Wu, Doctor of Engineering Science, 1989

Thesis directed by: Professor Raj P. Khera, Ph.D
Department of Civil and Environmental
Engineering

This research was conducted to study the characteristics of permeability and volume change of bentonite-sand mixes due to contaminants. The soil specimens consisted of clean sand, bentonite, and fly ash. Test liquids included seven diluted organic and inorganic chemicals and a landfill leachate.

Index tests included liquid limit, expansion, cracking, and sedimentation tests. Expansion tests proved to be the most useful for permeability evaluation in a contaminant environment.

The conventional consolidometer permeameter and flexible wall permeameter proved to be not completely satisfactory for testing soft bentonite mixes. A triple ring permeameter was developed to compensate for the limitations of these equipment and to provide a more convenient means of testing.

The test results indicated that diluted chemicals showed an adverse effect on permeability. However, when the consolidation pressure increased and the void ratio decreased, this effect tends to be minimized. Consolidation

pressure also restricted the adverse effect of cracking in a specimen and caused no permeability increase. The change of permeability generally agrees with the results found in the index tests and can be explained by the Gouy-Chapman theory.

The fly ash from one local source mixed with bentonite did not help to reduce the permeability. The expansion of the bentonite appears to be limited in the mixtures due to the effect of ion replacements.

Permeability tests conducted on bentonite-sand mixes indicated that void ratio could not be correlated well with permeability due to the effect of expanded volume of the bentonite. A four-phase soil diagram with two physical parameters, expansion ratio, e_r , and expansion function, e_f , is proposed. It can clearly express the volumetric changes of bentonite in a mix resulting from the influence of the chemical. Permeability was found to be strongly correlated to these parameters. It decreased with an increase in e_r and e_f . The concept of expansion appears to be very helpful in evaluating the permeability response of a bentonite mixture in a contaminant environment.

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CHAPTER 1
INTRODUCTION

1.1 Background

The disposal and storage of hazardous waste materials is now becoming one of the most critical environmental problems in the United States. The amount of hazardous waste generated has increased with time. In recent years, considerable attention has been given to developing safe and reliable waste containments for the disposal and storage of these problem substances.

The purpose of a waste containment is to create an impervious barrier between the waste and the surrounding ground, groundwater, and atmosphere (EPA, 1984). The general procedure is to build an impervious containment structure within which the waste is stored. Soil-bentonite slurry cut off (SB) wall has found wide application as a means of containment. This technique was originally developed for foundation construction and has been used extensively for structural lateral earth support and seepage control (Xanthakos, 1979).

Recent enactment of the Resource Conservation and Recovery Act (RCRA) resulted in stringent requirements for new disposal areas and the imposition of new requirements for upgrading and isolation of existing and abandoned hazardous waste sites. In many cases, SB wall is the only practical means to isolate the hazardous waste disposal site and prevent further contamination (IMCLAY, 1985).

Despite the increasing use of SB walls, there is insufficient information concerning the performance of SB backfill material in a waste containment structure (D'Appolonia, 1980; and Johnson et al., 1985). Many studies have shown that waste chemicals of higher concentrations tend to cause clayey soils to shrink and increase their permeability (Brown et al., 1984; Daniel, 1984; and Mitchell and Madsen, 1987). However, little investigation has been reported on the effects of dilute aqueous wastes on the properties of clayey soils (Bowder 1985). The effects of dilute chemical liquids would be more realistic than the effects of concentrated liquids since practically little pure reagent-grade chemicals are being disposed of. (Kingsbury, 1982; and Bowder, 1985).

Daniel et al. (1985) stated that the laboratory methods for determining the permeability of fine-grained materials are controversial. No permeameter has been developed to satisfy every requirement. No standard procedures have been established for permeability testing. Permeability of similar materials may deviate considerably depending on the type of apparatus and testing criteria used. Testing for the slurry material is particularly new. Therefore, it is necessary to develop a more appropriate testing technique that can be practically acceptable for all purposes.

1.2 Objective

The objective of this study is to observe the long term effects of selected chemical aqueous solutions on the

properties of bentonite-sand mix in an SB wall. To accomplish this objective, the research includes the following:

- review the available literature to ascertain the known effects of various chemicals on the engineering properties of clayey soils;
- determine the possibility of using waste products such as fly ash as a backfill component;
- evaluate the relative performance of treated and nontreated bentonite;
- evaluate the effect of various testing techniques on the values of permeability;
- develop a new permeameter which will be suitable for the testing of soft samples for SB wall design.
- develop simple testing techniques that can qualitatively indicate the soil-chemical compatibility and correlate it to the permeability.
- study the effects of chemicals on permeability and consolidation behavior; and
- use statistical techniques to facilitate data analysis and develop correlation between variables.

CHAPTER 2
LITERATURE REVIEW

2.1 Genral

This chapter presents a review of literature pertaining to the study of properties of hydraulic barriers within hazardous environments. The review consists of background descriptions and previous important findings including clay mineralogy, clay-permeant interactions and discussions related to permeability measurements.

**2.1.1 Hazardous Waste Management - Geotechnical
Considerations**

The disposal of hazardous wastes and groundwater contamination problems are rapidly becoming a national health hazard (Fung, 1980). The movement of hazardous wastes through the ground surface into the groundwater can cause human health and environmental damage. Rich (1981) illustrated typical potential sources of contaminations: (1) on-site waste disposal systems and industrial wastewater impoundments with holding ponds, (2) landfills, open dumps, abandoned dumps, (3) sewage treatment plant effluent and sludge basins, (4) sanitary sewers, (5) industrial waste discharges, (5) waste stockpiles and radioactive disposal sites, (6) spills and incidental discharges, (7) brine injection wells, and (8) overpumping.

Groundwater has little assimilation capacity compared to surface water. It flows slowly and shows virtually no rebound from pollution under natural conditions (Freeze and Cherry,

1979). Therefore, a high degree of groundwater protection must be provided by regulations. The Resource Conservation and Recovery Act (RCRA) of 1976 mandates that the Environmental Protection Agency (EPA) establish stringent requirements for new disposal areas. It also imposes new requirements for upgrading the isolation of existing and abandoned hazardous waste sites (Vardy, 1987).

Recently, the use of barrier technology to build a containment system has been recognized as an effective tool for groundwater protection (Fung, 1980). According to Evans and Fang (1982), a typical waste containment system consists of two major components:

- (1) active components including pumping wells, disposal wells, and waste fixation.
- (2) passive components including drainage systems, top seals, barrier walls, and bottom seals.

In the development of this technology, geotechnical aspects have been widely applied to collect pertinent site information required to predict the possible clay-permeant interactions (Fung, 1980; Evans and Fang, 1982). Geotechnical properties such as permeability, consolidation behavior, shear strength, and moisture-density relationships must be considered for the design of a hydraulic barrier (Mercurio, 1985). Geotechnical tests are frequently conducted to obtain these properties (Evans and Manuel, 1985). The construction of hydraulic barriers further requires some quality assurance techniques that are similar to those used during the

construction of conventional earth structures.

2.1.2 Soil-Bentonite Slurry Cut-off Wall

By far, most of the installations of vertical seepage barriers in the United States are built using the soil-bentonite slurry trench technique (Ryan, 1985; 1987). It consists of excavating a continuous trench under bentonite slurry which is subsequently backfilled with a blended mixture of soil and bentonite. Detailed design and construction procedures have been well described (Xanthakos, 1979; D'Appolonia, 1980; EPA, 1984; Ryan, 1980 and 1985; and Barvenik et al., 1985). The principal advantages of this technique are lower cost, confidence in quality, and flexibility of design and construction (EPA, 1984).

Factors that affect the performance of SB walls include: (1) design criteria, (2) backfill placement, (3) post-construction conditions, and (4) backfill composition and characteristics (Ryan, 1980; and EPA, 1984).

To produce a satisfactory vertical barrier, the material of the SB wall must contain suitable amounts of plastic fines, bentonite and water. The quantity of over size particles must be restricted to a minimum. D'Appolonia (1980) and Ryan (1985) recommended that for a low-permeability SB wall the backfill should have a minimum of fifteen to twenty percent fines. For a given bentonite content, the SB wall shows lower permeability as the fine contents are increased. In addition, bentonite content has an optimum value, beyond which its influence on permeability is reduced. In their

study Schulze et al. (1985) found that additional bentonite actually increased the permeability slightly. D'Appolonia (1980) indicated that plastic fines reduce permeability more effectively than non-plastic fines. The reasons were attributed to the clayey particles which contribute to the SB materials in the form of swelling, viscosity, gelation, and cation exchange capacity.

The water content of the backfill has a great effect on the permeability of the SB wall. It must be carefully evaluated through laboratory testing to determine the optimum water content at which the SB wall will preserve acceptable permeability and workability. For suitable placement of the backfill, a slump of four to six inches is recommended. The water contents for these slumps are typically 25 to 35 percent (D'Appolonia, 1980; EPA, 1984).

2.1.3 Permeability vs. Hydraulic Conductivity

Analyses of water flow in saturated soils are usually based on Darcy's equation as described below:

$$q = kiA \quad (2.1)$$

where q is the flow rate (L^3/T), i is the hydraulic gradient (dimensionless), A is the total cross-sectional area of flow (L^2), and k is the constant of proportionality (L/T).

k is defined as hydraulic conductivity in most disciplines but is often defined as permeability by geotechnical engineers. The American Society for Testing and Materials (ASTM D 653-80) defines the two terms as the same in terms of the flow of water. The permeability or hydraulic

conductivity defines the capacity of a porous medium to conduct a particular fluid and is a function of both the medium and the permeated fluid. The intrinsic permeability expresses the capacity of flow in terms of the properties of the porous medium only and is defined below (Freeze and Cherry, 1979):

$$k = K\tau_p/\mu_p \quad (2.2)$$

where: K = constant of proportionality or intrinsic permeability, L^2 ;

τ_p = unit weight of the permeant, F/L^3 ;

μ_p = dynamic viscosity of the permeant, FT/L^2

To simplify the terms and avoid ambiguity, this study uses permeability (cm/sec) as the term in the Darcy's equation. To account for the effect of fluid properties, the permeability obtained using demineralized water is taken as the base value. Permeabilities obtained using all other permeants are normalized into relative values by multiplying a permeant property ratio, P_r , which is defined below:

$$P_r = (\tau/\mu)_w/(\tau/\mu)_p \quad (2.3)$$

where: $(\tau/\mu)_w$ = fluid properties of demineralized water;

$(\tau/\mu)_p$ = fluid properties of permeant.

The units of the normalized permeability are cm/sec. Use of this parameter is compatible with the commonly used term in geotechnical engineering. In addition, it allows the examination of the effects of permeant properties.

2.2 Clay Mineralogy of Bentonite

2.2.1 General

Bentonite is a volcanic rock consisting mostly of montmorillonite with minor amounts of feldspar, quartz, mica, calcite, gypsum, and other minerals and rock fragments (Mitchell 1976).

The structure of montmorillonite consists of two basic types of structural sheet units that form a molecular lattice. As shown in Figure 2-1, the sheets are either made up of linked silica tetrahedra or of linked alumina octahedra. The mineral structure consists of an octahedral sheet sandwiched between two tetrahedral sheets. Each tetrahedra has a silicon atom in the center of a tetrahedral sheet arranged to be equidistant to four oxygens or hydroxyls. The tetrahedral groups are arranged to form a hexagonal network by sharing oxygen atoms with other tetrahedral sheets. The octahedral unit consists of two layers of closely packed oxygens or hydroxyls, where aluminum, magnesium, or iron are embedded so that they are equidistant from six oxygens or hydroxyls.

Montmorillonites tend to occur in equidimensional and very thin flake-shaped units. Hundreds of such flakes aggregate and form an individual thin particle. The particle has an extremely large surface area, typically 600-800 m²/gm when it is fully hydrated. The basal crystal repeat spacing of the c-axis may vary from 9.6 Angstroms to complete separation on dispersion (Mitchell, 1976). This variance is

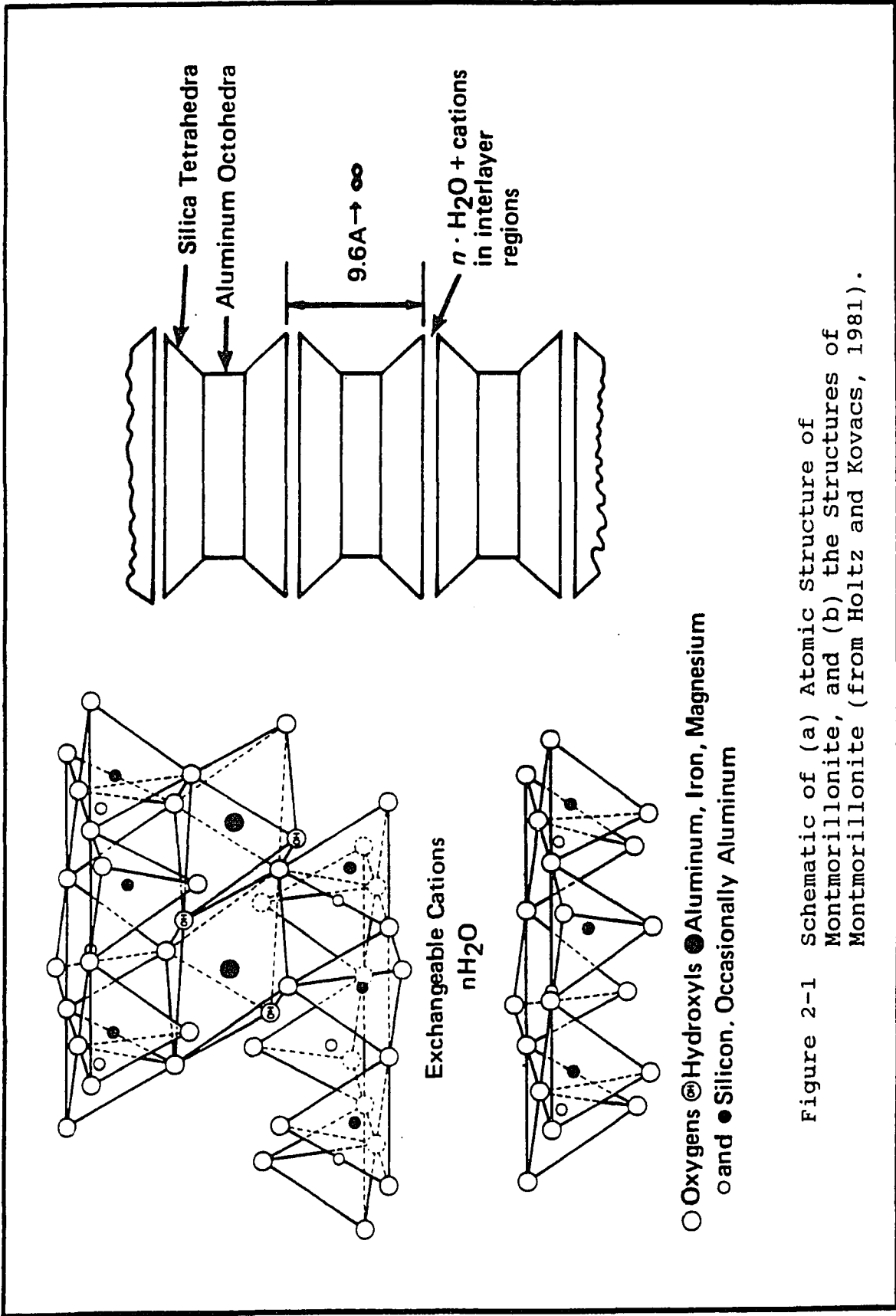


Figure 2-1 Schematic of (a) Atomic Structure of Montmorillonite, and (b) the Structures of Montmorillonite (from Holtz and Kovacs, 1981).

related to the interaction of water or other polar molecules, which are adsorbed between the adjacent silica sheets.

Montmorillonite is often unbalanced by substitutions. When substitution occurs, a negative charge deficiency results which is in part balanced by substitutions in the other sheet of the unit layer. The charge deficiency occurs mainly in the octahedral sheet, and the net charge imbalance must be satisfied at the surface of the unit. A typical example is sodium montmorillonite which has its surface charge balanced by a sodium ion. Because the sodium ions are loosely held, sodium montmorillonites have the capacity to readily exchange their sodium ions for other cations, particularly those of higher valence. The ion exchange tendency, together with their larger surface area, give this group of clay minerals a very high cationic exchange capacity of eighty to one hundred fifty meq/100gm.

2.2.2 Montmorillonite-Water Systems

A primary mechanism by which a clay interacts with its surrounding environment is by cation exchange (Mitchell, 1976). The net negative charges on the clay particle is often compensated by cations located on the layer surfaces. When the clay is in water, these compensating cations have a tendency to diffuse away from the layer surface as their concentrations are smaller in bulk solution. However, they are also attracted electrostatically to the charged layer surface. These opposing trends result in the creation of an diffuse double layer. The thickness of this layer is a

function of the properties of the pore fluid, the valency and concentration of the cations, and the surface charge on the particle. If the particle is expandable, it will also have an interlayer space occupied by the pore fluid and the cations. Factors affecting the thickness of the interlayer space are the same as those for the double layer. Under favorable conditions, the cations in the double layer and in the interlayer space can be exchanged by other cations. The molecules of the pore fluid also can be replaced by the molecules of a new fluid. These changes will affect the thickness of the double layer and alter the clay behavior.

The most common mathematical theory, used in geotechnical engineering, describing the ion distribution near the clay surface is the Gouy-Chapman model (Evans et al., 1985). van Olphen (1977) reviewed other theories that described the ion distribution. He concluded that these theories would provide the same trends with respect to pore fluid effects with variation in magnitude only.

2.2.3 Clay Structures

van Olphen (1977) discussed the clay-water interactions and developed seven modes of particle associations in clay suspensions (Figure 2-2). These vary between limits of fully-dispersed and fully-flocculated structures. The Gouy-Chapman model indicates that the tendency toward flocculation may be produced by a decrease in the double layer thickness as a result of a decrease in the interparticle repulsion force. Conversely, tendency toward dispersion increases as the

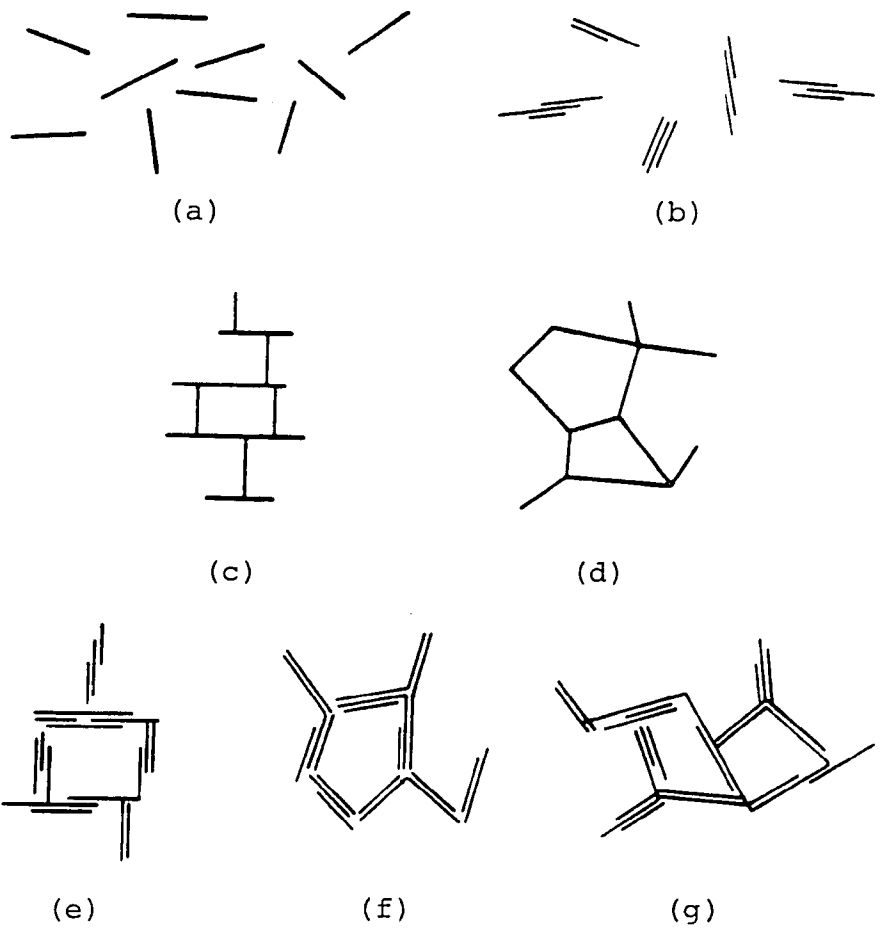


Figure 2-2 Modes of Particle Association in Clay Suspensions and Terminology: (a) dispersed and deflocculated, (b) aggregated but deflocculated, (c) edge-to-face flocculated but dispersed, (d) edge-to-edge flocculated but dispersed, (e) edge-to-face flocculated and aggregated, (f) edge-to-edge flocculated and aggregated, and (g) edge-to-face and edge-to-edge flocculated and aggregated (after van Olphen, 1977).

double layer thickness increases.

According to the Gouy-Chapman model, the thickness of the double layer is affected by many variables such as electrolyte concentration, ionic valence, dielectric constant, pH, size of the hydrated ion, anion adsorption, and temperature. Mitchell (1976) derived a quantitative indication of the thickness of a diffuse double layer based on the work of Gouy and Chapman:

$$H = (DT/h^2 v^2)^n \quad (2.4)$$

where: H = relative thickness of double layer;

D = dielectric constant of the medium;

T = temperature;

h = electrolyte concentration;

v = cation valence, and

n = a constant (to be 0.5 in most cases).

Lambe and Whitman (1969) discussed the effects of these variables on the clay-water system in light of the Gouy-Chapman theory. Evans et al, (1983) further described these parameters relative to the changes in soil structure, as illustrated in Table 2-1. They concluded that a flocculated structure will yield a higher permeability than a dispersed structure.

In addition to the primary structure, a clay mass may possess secondary structure. The principal types of secondary structure are cracks, joints, slickensides, and fissures. The cracks may be formed as a result of shrinkage due to dessication, clay permeant reactions, and differential

settlement. Secondary structure, if present and open, will govern the majority of the flow (Mitchell and Madsen, 1987).

Table 2-1
¹Effect of Pore Fluid Parameters On Double Layer Thickness and Soil Structure Tendency

Pore Fluid Parameter	Change in Parameter	Tendency in Double Layer	Tendency in Soil Structure
Electrolyte Concentration	Increase Decrease	Decrease Increase	Flocculated Dispersed
Ion Valence	Increase Decrease	Decrease Increase	Flocculated Dispersed
Dielectric Constant	Increase Decrease	Increase Decrease	Dispersed Flocculated
Temperature	Increase Decrease	Increase Decrease	Dispersed Flocculated
Size of Hydrated Ion	Increase Decrease	Increase Decrease	Dispersed Flocculated
pH	Increase Decrease	Increase Decrease	Dispersed Flocculated
Anion Adsorption	Increase Decrease	Increase Decrease	Dispersed Flocculated

1) from Evans et al., 1983.

2.3 Clay-Permeant Interaction

When the original pore fluid in a clay soil is displaced by the influx of another pore fluid, reactions between clay particles and the new permeant will likely occur. The behavior of clay on macroscopic and microscopic scales is highly related to the nature of the pore fluid. To evaluate the clay-permeant interactions in a hazardous waste environment, the characteristics of pore fluids must be

classified. The Environmental Protection Agency (EPA, 1980) divides industrial wastes into four physical classes: aqueous inorganic, aqueous organic, organic, and sludges. Evans et al, (1985) reviewed the natures of these liquids. They concluded that among the frequently encountered pore fluids in engineering practices, organic fluids are most important in the study of pore fluid effects on clays. They further discussed the effects of certain pore fluid properties on clay behavior. They found that changes in clay behavior due to the variation of pore fluid would be essentially governed by the Gouy-Chapman theory. Ion exchange reactions with inorganic and organic cations in the pore fluid will change the thickness and composition of the double layer. Organic fluids may cause volume changes in the clays by altering interlayer spacings and increasing the thickness of the double layer. They concluded that the Gouy-Chapman theory is a useful tool for qualitatively predicting the influence of pore fluid on clay behavior.

In addition to the change of clay structure, dissolution of clay minerals is an important phenomenon of clay-permeant interaction. Research (Evans et al. 1985; Anderson and Jones, 1983; and Uppot, 1984) indicated that strongly acidic and strongly basic liquids have the capacity to dissolve clay minerals. Under such conditions, the Gouy-Chapman theory will no longer govern the clay behavior.

There are other clay-permeant reactions such as gas forming reaction and precipitation reaction (Uppot, 1984).

These phenomena show little effect on the clay behavior in most circumstances and, therefore, are not discussed in this study.

2.4 Factors Affecting Permeability

Permeability has been recognized as one of the most variable properties in geotechnical engineering. It depends on many factors and is difficult to determine reliably (Mitchell and Madsen, 1987). Lambe and Whitman (1969) indicated that permeability depends on the characteristics of both the permeant and the soil. It depends on the geometric characteristics of the pore spaces for flow. It varies with geometric arrangement of soil particles and external stress conditions. In hazardous waste environments, the chemical characteristics of the permeant and its physical-chemical interaction with the soil are also important. (Dunn, 1983).

2.4.1 Effects of Soil Properties on Permeability

There are five characteristics of soil that have certain influences on the permeability. These characteristics are closely interrelated and cannot be isolated to determine their individual effect on permeability. Lambe and Whitman (1969) indicated that particle size and void ratio have a direct relationship with permeability. The smaller the particle size and the smaller the void ratio, the lower the permeability. Clay composition has a large influence on permeability. Sodium montmorillonite has the lowest permeability compared with other clays. For clayey material, the fabric component of the structure is one of the most

important characteristics affecting the permeability. The flocculated structure will have a higher permeability than the dispersed structure. In addition to the microfabric influence, further studies by others (Mitchell and Madsen, 1987; Daniel and Trautwein, 1986; and Herrmann and Elsbury, 1987) have also discussed the effects of minifabric and macrofabric on permeability. They concluded that permeability will be governed by the flow rates through the largest flow channels. Other factors such as the degree of saturation and biological processes have certain effects on the permeability. Degree of saturation controls the space available for fluid flow. Biological processes produce microorganisms in the pore space and lead to lower permeability due to pore clogging (Mitchell and Madsen, 1987).

2.4.2 Effects of Chemicals on Permeability

Chemicals preserved in the permeant may affect a soil's permeability through the physical-chemical processes. Previous discussion of clay-permeant interaction indicated that permeability will be altered if the clay structure is reconstituted or destroyed by chemicals. Although numerous studies have been conducted, a definitive theory as to how chemicals affect permeability has not been drawn. Several mechanisms have been developed in an attempt to explain the changes in permeability due to chemicals (Bowders, 1985). Mitchell and Madsen (1987) noted that the effects of clay-chemical interactions are more pronounced in the case of high

water content systems such as slurry walls. Soil particles are less constrained from movement than in compacted clays and thus are more susceptible to fabric changes. Observations made on compacted clays thus can not be directly compared to those reported on slurry walls.

For the purpose of discussion, chemicals are classified as below:

- acids and bases;
- inorganic and organic compounds;
- dilute organic compounds;
- mixed chemical compounds.

2.4.2.1 Effects of Acids D'Appoloia (1980) permeated slurry wall backfill material with five percent hydrochloric acid and observed a one-order-of-magnitude increase in the permeability. Gee, Campbell, and Opitz (1980) and Peterson and Gee (1985) used uranium mill tailings leachates (pH = 1.8 to 2.0) to permeate clays. In experiments conducted for approximately three years, the permeability showed a tendency to decrease. The decrease was attributed to the precipitation causing clogging of the pore spaces. Anderson et al. (1982) permeated four compacted clays with acetic acid. Initially all these soils showed a significant drop in permeability. However, the permeability of two of the clays began to increase later. Effluents analyses showed that soils were dissolved by the acid. The tests were terminated and, therefore, could not conclude that the other two clays would show similar results.

Lentz et al. (1985) used hydrochloric acid having pH values of one, three, and five to permeate three different clays: kaolinite, kaolinite-bentonite mixture, and magnesium montmorillonite. Six pore volumes of liquid were passed through each sample. They did not observe any increase in permeability indicating that no significant dissolution of clay minerals occurred. Gipson (1985) evaluated the effect of an acid liquor on two compacted materials: natural clay with varies plasticity and a silty sand-bentonite mixture. The acid liquor had a pH value of 2.2 and was high in calcium, calcium oxides, sodium chloride, and sulphate. Tests were conducted in different sizes of permeameters for a period of approximately one year. As compared to results using tap water, the acid liquor showed lower permeability for natural clay. However, the bentonite mixture showed an increase in permeability with time.

Based on the available information, pure acids or permeants with high concentration of acids tend to dissolve clay minerals and cause greater increase in permeability than actual acidic leachates. This is because precipitation occurs in a natural leachate as the dissolved soil buffers the acid. The precipitates tend to clog the pores of the soil and reduce its permeability. However, after a long period, the precipitates and certain soil constituents may redissolve, and the permeability of the soil may increase. Dilute acids appear not to have a major impact on permeability. Sand-bentonite mixtures may be susceptible to attack by acids due

to the relatively small amount of clay.

2.4.2.2 Effects of Bases D'Appolonia (1980) permeated a slurry wall filter cake and soil-bentonite backfill with a five percent sodium hydroxide solution. He found the permeability increased by a factor of two to ten.

Lentz et al., (1985) studied the effects of sodium hydroxide at pH values of nine, eleven, and thirteen on three clays that were used similar to their tests for acids. The permeability showed little change for pH of nine and eleven. The strongest base decreased permeability up to a factor of about 10. The magnesium montmorillonite showed the greatest decrease among the three clays tested. The reduction in permeability was attributed to the precipitation of magnesium in the clay voids.

According to the limited studies, the effects of bases on permeability are not clear. Pure, concentrated bases have the potential for altering the permeability of a clay. However, both decreases and increases in permeability have been observed. The difference in the test results may be attributed to the specimen preparation and testing apparatus (Mitchell and Madsen, 1987). In addition, other factors such as electrolyte concentration, and valence of electrolyte may show more effects on permeability than the pH values (Bowders, 1985).

2.4.2.3 Effects of Inorganic Compounds Dunn (1983) indicated that beryllium, cadmium, chromium, copper, lead, mercury, and zinc are the inorganics of most concern in waste

management. Dunn and Mitchell (1984) evaluated the effect of synthetic lead-zinc tailing leachate on two types of compacted clay. Results of both soils showed a higher permeability when permeated with leachate compared to tap water.

Alther et al. (1985) studied the effects of several inorganic aqueous solutions on permeability of both contaminant resistant (polymerized) bentonite and untreated bentonite. They found that of the inorganic solutions tested, those with potassium cations or chloride anions or both induced the largest permeability increases with increasing electrolyte concentration. The increase of cation valence has a greater initial effect on the bentonite permeability. There is a saturation limit, beyond which any further addition of the soluble salts had only a limited impact on permeability. Pierce and Witter (1985) tested three clays with three inorganic liquids: FeCl_3 , $\text{Ni}(\text{NO}_3)_2$, and $\text{Ni}(\text{NO}_3)_2$ (1M). Test results indicated that these permeants do not affect permeability compared to standard water (CaSO_4 0.01 M).

Daniel and Liljestr and (1984) concluded that soils tend to be most permeable to water that is rich in multi-valent dissolved cations. It is least permeable to water containing dilute solutions of sodium or other monovalent cations. Soils tend to be more permeable to tap water compared to distilled water (Dunn and Mitchell, 1984).

2.4.2.4 Effects of Organic Compounds Effects of organic compounds on the permeability of clayey materials have been extensively studied for the past few years (Green et al., 1979; Anderson and Brown, 1981; and Bowders, 1985). Mitchell and Madsen (1987) reviewed recent literature and found most of the results are conflicting and confusing. However, there are still some conclusions that can be drawn. The permeability of clayey material is invariably influenced when pure organic chemicals are permeated through the soil. Hydrocarbons have no effect on permeability. Water soluble organics such as alcohols and ketones showed no effect on permeability even at concentrations as high as seventy-five percent. Organic acids can dissolve carbonates and iron oxides. Precipitation and pore clogging may occur due to the buffering of acid. Over a long period and if all carbonates and oxides are removed, the permeability may increase. Organic acids tested did not cause large scale dissolution of clay particles. Weak organic bases appear to have no effect on the clay particles. The mechanisms which cause the change in permeability are not clearly defined. Daniel and Liljestr and (1984) indicated that collapse of the double layer and shrinkage cracking may be the most probable controlling mechanisms for the permeability increase.

2.4.2.5 Effects of Dilute Organic Compounds Acar et al. (1985) described a study in which dilute nitrobenzene, acetone, phenol, and benzene were passed through compacted kaolinite. All tests resulted in a slight decrease of

permeability.

Brown et al. (1984) permeated three sandy clay mixtures with permeants containing various percentages of acetone. They observed that the higher permeability to polar organic liquids diminished as the organics were diluted with water. A solution diluted with more than fifty percent water would behave like water.

Bowders and Daniel (1987) used dilute organic chemical permeants with compacted kaolinite and illite-chlorite. The chemicals used included methanol, acetic acid, heptane and trichloroethylene. The permeants were prepared by mixing these chemicals with water at various concentrations. The results of the investigation showed that dilute organic chemicals have little effect on the permeability of compacted clay if their concentration was less than 80% by volume.

The limited information indicates that dilute solutions do not alter permeability significantly. In some cases, the clay may show less permeability compared to those measured with water (Mitchell and Madsen, 1987).

2.4.2.6 Effects of Mixed Chemical Compounds Leachates generated from hazardous waste sites often consist of more than one chemical. Little information is available about the effect of mixed chemical solutions on permeability (Mitchell and Madsen, 1987). Brown et al. (1984) used mixtures of xylene and acetone to permeate compacted clay. The results indicated that at 100% xylene, permeability was four orders of magnitude greater than the value found with water. When

12.5 percent of acetone was added, the permeability was reduced to about 3.3 orders of magnitude lower than with xylene alone but was still greater than for water. As the concentration of acetone increased, permeability gradually increased. It showed a final value of 1.5 orders of magnitude greater than water when 100% acetone was used. They explained that concentrated solvents have a greater impact on the shrinkage of the clay than the mixtures of chemicals.

Daniel and Liljestr nd (1984) studied the effects of landfill leachates on natural clay liner materials. Six leachates, either actually collected from the sites or laboratory manufactured, were permeated through five types of clays. None of these leachates produced an increase in permeability of the clay tested. In fact, some of the clays appeared to be less permeable to the leachates than to water.

2.5 Laboratory Permeability Measurement

Laboratory permeability measurement depends on many variables (Dunn, 1983). In this section, a review of current testing techniques, equipment, and criteria for permeability measurement are presented.

2.5.1 Test Equipment

There are three laboratory apparatus commonly used for determining the permeability of fine-grained soils. These devices are: (1) rigid-wall permeameter (RP), (2) flexible-wall permeameter (FP), and (3) consolidometer permeameter (CP). None of these has been adopted as standard method by ASTM. Each of these devices has advantages and limitations.

Controversies have developed over which type of device is best (Zimmel, 1981; Daniel et al., 1984; Daniel et al., 1985; Edil and Erickson, 1985; and Mitchell and Madsen, 1987). The rigid-wall permeameter (RP) is essentially assembled by clamping a standard compaction mold between two end plates. Dependent upon the purpose of testing, soil is placed into the mold either by compaction or deposited to a controlled density. A collar is attached as a reservoir for the permeant. Bowders (1985) concluded that RP is the simplest and most economical device for testing compacted clay. It is normally operated without backpressure saturation.

Olson and Daniel (1981) reported that permeability measurements on an unsaturated specimen are often difficult as the permeability varies with changes in the degree of saturation and suction. Therefore, small gradients must be used to minimize the alteration of these parameters. Otherwise, permeant flow-gradient-suctions must be measured simultaneously at a point. Backpressure can be applied in RP to saturate the specimen. However, Edil and Erickson (1985) indicated that backpressure appears to have a detrimental effect as it facilitates the formation of channels and side flow. Thus, use of backpressure with RP may lead to erroneous results.

Another limitation of RP is that the device does not allow application of any vertical or confining stress to the specimen. In the field, there will usually be a vertical

stress applied either from overburden soil or from solid waste. Daniel and Liljestr nd (1984) reported the importance of controlling vertical stress when measuring permeability of a soil specimen. Their results showed three orders of magnitude changes in the measured permeability when vertical stress was changed by an amount equivalent to just a few meters of overburden soil. The most criticism of RP is that sidewall leakage may occur in cases where chemical-clay interactions lead to shrinkage and cracking (Daniel et al., 1985).

To overcome this limitation, Anderson et al. (1985) developed a modified version of RP known as a double-ring permeameter. The base plate contains an additional ring that separates flow near the sidewalls from flow through the central portion of the soil core. If there is significant sidewall leakage, the rates of flow into the two rings will not agree. One could either use the outflow of the inner ring to develop permeability or reject the test.

The consolidometer permeameter (CP) is a conventional consolidation cell modified with drainage lines to permit permeability measurements. The soil specimen is usually trimmed into the specimen ring. The applied vertical stresses can minimize the chance of potential sidewall leakage. Mitchell and Madsen (1987) concluded that CP is potentially the most useful apparatus among the three types of permeameters. It allows the application of a wide range of vertical stresses. The change in specimen thickness due to

the influence of seepage forces and chemical alteration can be accurately monitored. The relatively thinner specimen offers shorter testing time in comparison with other permeameters.

The flexible-wall permeameter (FP) is a triaxial cell or a modified triaxial cell. The soil specimen is enclosed in a flexible membrane and confining stresses are applied for consolidation. Many designs have been described in the literature (Olson and Daniel, 1981; Dunn, 1983; Evans and Fang, 1984; Daniel et al., 1984).

FP has several advantages over the other permeameters. Undisturbed specimens are easily tested because minimal trimming is required and irregular surfaces of the specimen can be accommodated. Backpressure is normally used to facilitate saturation. Confirmation of specimen saturation and measurements of volumetric deformations can be obtained by using conventional triaxial testing techniques (Head, 1982, and 1986). The chance of sidewall leakage are negligible.

There are three disadvantages of FP. The commercial latex membrane is vulnerable to chemical attacks, particularly for organic fluids with dielectric constants less than 10 (Acar et al., 1985). Daniel et al. (1984) recommended that the problem can be eliminated by wrapping the specimen with a Teflon sheet or Teflon tape. The membrane is then placed over the wrapped specimen. However, even with such provisions, significant diffusion of organic

constituents through latex membranes has been observed (Acar et al., 1985). Neoprene and butyl rubber have more chemical compatibility than latex. However, they are too rigid to seal the specimen completely. Bulk flows were observed around the specimen and caused significant erroneous measurements (Dunn, 1983). Evans and Manuel (1985) successfully used a modified polyvinyl alcohol (PVA) membrane to prevent the breakthrough of perchloroethylene (PCE).

Permeability measurements in FP frequently need to apply higher cell pressure to accommodate the greater hydraulic gradients. The minimum cell pressure must be equal to the influent hydraulic pressure. The resulting higher effective stress at the effluent end tends to close the seepage paths or any possible cracks in the soil specimen yielding erroneous permeability measurements (Daniel et al., 1985). Lower confining pressure can only be applied if lower hydraulic gradients are used. This would be impractical as the clays are relatively impermeable and thus require considerable time to complete the permeation of one or more pore volumes.

To prepare slurry specimens for FP testing is difficult and time-consuming (Schulze et al., 1985). The conventional means of applying a vacuum or negative stress to the sample is practically useless because of the impermeable nature of the specimen. Schulze et al. (1985) used a piece of aluminum foil, placed around the circumference of the former, to facilitate the mounting of the specimen. Other disadvantages of FP

reported are higher costs and the difficult nature of the test compared with other types of permeameter (Daniel et al. 1985).

2.5.2 Test Methods

There are a number of methods available to determine laboratory permeability. They are constant head method, falling head method, and some special techniques such as constant rate of flow method. Most of these methods can be used with the permeameters described in the previous section.

Constant head and falling head methods are widely used. Testing procedures and data reductions have been well described in the literature (Lambe, 1975; Head, 1982; Bowles, 1986; and ASTM, 1987). Where backpressure is used for a falling head measurement, the test would essentially become a constant head test provided that the reduction in applied head is small compared to the applied pressure.

Permeability can be calculated from the coefficients of consolidation. It is obtained based on the results of conventional one-dimensional consolidation tests and uses the following equation:

$$k = c_v a_v \gamma_w / (1+e) \quad (2.5)$$

where: c_v = coefficient of consolidation,
 a_v = coefficient of compressibility,
 γ_w = unit weight of water,
 e = void ratio

Olson and Daniel (1981) concluded that the ratio of permeability measured in conventional tests to that

determined from consolidation tests can vary from 0.9 to 1000. This ratio is quite variable and dependent on soil type. Zimmie et al. (1981) indicated that values determined from consolidation tests are almost always less than those from conventional tests, and thus not conservative for the design. They recommended that this method not be used to determine permeability for a hazardous waste site.

In the constant rate of flow test, a known flow rate is applied with a flow pump (Olsen et al., 1985). The induced head difference across the specimen is measured with a sensitive differential pressure transducer. The permeability is calculated similarly to the constant head method. Olsen et al. (1985) and Zyl (1986) concluded that this technique offers a rapid permeability measurement at substantially smaller gradients.

2.5.3 Hydraulic Gradient

In-situ hydraulic gradients are generally on the order of 0.5 to 10 (Dunn, 1983). This would be impractical in the laboratory for the testing of a clayey specimen because of the long testing time required. Uppot (1984) reviewed many recent practices and reported that hydraulic gradients used in the laboratory generally range from twenty to five hundred.

Dunn and Mitchell (1984) studied the effect of hydraulic gradients on permeability measurements. They found permeability decreased with increasing gradient and that the decrease was irreversible with decreasing gradient. Many

factors may cause these decreases. Clogging of pore spaces resulting from the migration of soil particles is the most probable reason. Other possible causes include: (1) growth of microorganisms, (2) dispersion of the soil fabric due to the soil-chemical interaction, (3) consolidation at the effluent end of the sample in response to the increased effective stress. They concluded that the hydraulic gradient should be as close as possible to that expected in the field, still allowing reasonable time to complete the test.

Haji-Djafari and Wright (1982) suggested that the applied gradient should be maintained at a level where laminar flow and Darcy's law are not violated. The pressure difference should not significantly alter the physical characteristics of the specimen. The applied gradient also should provide sufficient time to allow for soil-chemical interaction.

Carpenter (1982) performed similar tests on various dimensions of specimens. He found the tendency for the increase of permeability was less for larger diameter and smaller height samples. He recommended a minimum diameter of 2.8 inches and a height to diameter ratio of 0.5 to 1.0 be used for the test.

2.5.4 Termination Criteria

Permeability tests on fine-grained soils are time-consuming. There is no consensus on termination criteria for these tests (Peirce and Witter, 1986). Evans and Manuel (1985) recommended that permeability test be conducted with

permeation of at least two to three pore volumes of the specimen. Daniel et al. (1984) suggested that the test should be continued until the permeability becomes steady or the inflow and outflow rates are equal. For permeants other than water, the chemical characteristics of the effluent should be similar to those of the influent. Peirce and Witter (1986) used linear regression and related statistical techniques to determine the termination criteria. They concluded that the test can be terminated when the slope of permeability versus pore volumes is close to zero.

2.6 Consolidation of Bentonite-Sand Mix

Consolidation of a clayey soil is highly dependent upon the permeability of the material. Terzaghi's theory has been used to predict the consolidation behavior of clay. For SB backfill mix that is predominantly coarse-grained material, Terzaghi's theory may not be valid. Fukue et al. (1986) investigated consolidation properties of bentonite-sand mixtures in an oedometer. They found a threshold void ratio of nonclay fraction at which the frictional resistance becomes dominant during consolidation. Based on this value with the clay content and the dry density, the soil can be easily classified as sandy or clayey. The classification is based on the mechanical properties of the soil. It is different from the conventional technique that is based on the physical properties of the material. In addition, they observed that salt infiltration into the specimens caused rapid settlement and greatly reduced the swelling. They

concluded that as long as the void ratio of the nonclay fraction is less than the threshold value, the influence of salt is negligible.

2.7 Soil Index Tests

Soil index tests have been developed based on the physical properties of the soil. They may provide qualitative information on the changes that occur in soils within a contaminant environment (Dunn and Mitchell, 1984). Atterberg limits, hydrometer analysis, and swelling observation are frequently used to evaluate soil-liquid compatibility. The results indicate the change of soil for water adsorption and the type of structure (dispersed or flocculated). This section contains the findings of recent research conducted on these tests with water and chemical liquids.

2.7.1 Atterberg Limit Tests

Liquid limit and plastic limit with plasticity index are found convenient to express the plastic properties of soils. Many investigations have been conducted to obtain a detailed understanding of the factors and mechanisms controlling the liquid limit. Sridharan et al.(1986) reviewed these studies and concluded that the liquid limit of natural soil should be controlled by the thickness of a diffuse double layer. However, the influence of other composite factors, such as the percent of clay, size, surface area, and concentration of exchangeable cations is not clear. Acar et al. (1985) performed liquid limit tests on montmorillonite with a variety of organic fluids. The liquid limit increased with an

increase in the dielectric constant of the pore fluid. However, tests on kaolinite did not follow the same trend. The results were explained by using the Gouy-Chapman theory. The higher the dielectric constant, the thicker the double layer. The liquid limit thus increases with increasing adsorbed water. For kaolinite, the results may be related to the unaccounted charges at the broken edges of the soil particles. These charges have a major effect on the particle interactive forces. Acar et al. (1985) further found that changes in intrinsic permeability correlate well with the change in liquid limits for the kaolinite. Permeability was found to decrease as the liquid limit of kaolinite increased.

Daniel and Liljestr nd (1984) conducted similar tests on five clays with six different liquids. Most of the soil showed negligible alteration of the limits when tested with dilute solutions. However, when mixed with pure methanol or xylene, all soils showed a decrease in the liquid limit and loss of plasticity. Foreman and Daniel (1986) observed the same results when they tested three clays with methanol and heptane. Bowders and Daniel (1987) concluded that Atterberg limits can be affected by the soil-chemical interaction. They can provide an excellent qualitative indication of the potential of an organic liquid that can affect the permeability of the compacted clay.

2.7.2 Sedimentation Tests

Hydrometer analysis is useful in studying the tendency of various liquids to flocculate clays. Tests are essentially

done in accordance with ASTM D-421 and D-422. Foreman and Daniel (1986) performed sedimentation tests with water, methanol, and heptane. A dispersant was added, and then the sedimentation of the clays was observed. The clay fraction remained in suspension with water for more than twenty-four hours but flocculated and settled out of suspension shortly after being mixed with methanol or heptane.

Dunn and Mitchell (1984) performed twelve hydrometer tests on two soils using water and waste liquid. For only half of the tests, a dispersant was used. The waste liquid caused more flocculation than water for both types of soil suspensions. Further observations made on the permeability tests showed both soils to be more permeable with waste liquid than water. They concluded that hydrometer tests may be a rapid method to qualitatively determine if a permeant has the potential to cause flocculation and increase permeability.

Bowers and Daniel (1987) conducted sedimentation tests on two clays with four organic liquids without using a hydrometer. The change of distance from the surface of the liquid to the top of the soil-liquid suspension was recorded with time. In concentrated liquids, they found that all soil particles were flocculated and settled out of the suspension rapidly. When these liquids were diluted, no noticeable changes were observed. The flocculation was attributed to the lower dielectric constant of the tested liquids. They concluded that mixtures having a dielectric constant greater

than sixty to seventy will not cause particles to flocculate. They further correlated these results with the findings from their permeability tests. It was hypothesised that if an organic liquid does not cause soil particles to flocculate in a suspension, it will not cause an increase in its permeability.

2.7.3 Swelling Tests

When certain soil particles are mixed with water, their volumes may increase due to the electrochemical interaction with water. This behavior is affected by the thickness of the diffuse double layer. The swelling potential varies with the mineral composition of the clay. It can be classified by using a plasticity index and clay activity (Mitchell, 1976). The higher the plasticity index, the greater the swelling potential. Kaolinite and illite show minor swelling. Sodium bentonite can swell as much as ten to fifteen times its dry volume when it is fully hydrated (American Colloid Co., 1982; EPA, 1984).

Acar et al. (1985) studied the swelling behavior of compacted soils in a consolidation cell and found sodium-montmorillonite showing the highest value. When mixed with organic liquids, the swelling potential dropped for all clays tested. It generally decreased with the decreasing dielectric constant of the liquid. They concluded that the changing volume can be attributed to the contraction of the diffuse double layer.

2.7.4 Cracking Observations

Clay will shrink significantly due to loss of its moisture content. Cracking is a result of the clay undergoing three dimensional shrinkage. Clayey structures may shrink, crack, and lose their integrity due to seasonal desiccation. Therefore, clay liners are susceptible to damage from desiccation cracking. In many cases, cracks and fissures were reported to be responsible for the failure of clay liners (Daniel, 1984b).

Kleppe and Olson (1985) studied the cracking tendency of various compacted clay/sand mixtures. The study showed that desiccation shrinkage increased linearly with compaction water content and was unaffected by density. Shrinkage strains greater than ten percent will cause serious problems in the field. They concluded that to reduce cracking, the sample should consist of fifty to seventy-five percent sand, a wide range in grain sizes, and enough silt and clay to fill the voids.

Alther et al. (1985) mixed bentonite with KCl solutions and observed their cracking patterns. Their results could be explained by the Gouy-Chapman theory. With increasing flocculation of soil structure, cracking occurred more frequently and crack density was high. Ryan (1987) used a cracking test to determine the compatibility of bentonite with a specific permeant. Bentonite was mixed with leachate and allowed to dry without any disturbance. Cracking was observed which indicated that the bentonite was vulnerable to

chemical attack. Additional tests conducted on attapulgite showed no cracking. He attributed the difference to the effect of distinct soil structures. Attapulgite is composed of needle-like particles that do not rely on a swelling phenomenon to form a slurry. The chemicals thus exhibited only a minor effect on the overall performance of the slurry. Permeability tests conducted on bentonite and attapulgite showed agreement with the results from cracking observation.

2.8 Properties of Fly Ash

Fly ash is the finer portion of the residue obtained from the combustion of pulverized coal in electricity generating boiler units. It is produced in vast quantities and yet underutilized in the United States (DiGioia et al., 1979). Fly ash has been widely used in concrete technology. However, its use in waste containment has not been understood. (Bowders et al., 1987). Parker et al. (1977) conducted permeability tests on mixtures of soil and fly ash. They obtained permeabilities in the range of 10^{-6} cm/sec. Vesperman et al. (1985) performed permeability tests on two pozzolanic fly ashes, mixed with quartz sand, and compacted to various densities. The study indicated that the addition of fly ash to a high permeability soil will result in a dramatic reduction in permeability. However, there is a limitation for the fly ash content in the mixture, beyond which the increase of fly ash showed no effect on permeability. This limitation is approximately equal to the initial pore volume of the tested soil. Permeability is

highly dependent upon the type of fly ash. The self-cementing fly ash greatly reduced permeability to a value of 10^{-7} cm/sec and is considered to be suitable for liner application.

Bowders et al. (1987) stabilized two class F fly ashes by adding various amounts of lime or cement. Test specimens were subjected to compaction, strength, durability, and permeability tests. Their results showed that permeability decreased with increasing amounts of stabilizer, and that fly ash could be an alternative material for waste containment.

2.9 Summary of Literature Review

Based on the literature review, the significant findings are summarized as follows:

- The disposal of hazardous wastes and groundwater contamination problems present an ever-increasing challenge to geotechnical engineering professionals. The design of a waste containment becomes one of the primary works for the waste management business.
- The performance criteria for an installed barrier are focused on soil-waste compatibility and the preservation of satisfactory permeability.
- The soil-bentonite slurry wall has been increasingly used as a waste containment. However, its long-term performance has not been evaluated.
- Clay-permeant interactions may result in certain effects on the soil mass include altering clay structure, dissolution of clay minerals, gas forming reaction, and precipitation

reactions.

- The thickness of the double layer of clay particles has a direct relationship to soil permeability. In general, the expansion of the double layer causes soil to swell, promotes a dispersed arrangement of soil particles, and tends to reduce permeability. Contraction of the double layer encourages soil to shrink, promotes a flocculated arrangement of soil particles, generates cracks or macropores, and tends to increase permeability.
- The most common mathematical theory used to describe soil-chemical interactions is the Gouy-Chapman model. Based on this, the alternation of double layer thickness is controlled by the dielectric constant, electrolyte concentration, valence of electrolytes in the pore fluid, and absolute temperature.
- Concentrated acids and bases can significantly dissolve clay minerals. Removal of the dissolved material will increase permeability while precipitation will tend to clog the pores of the soil and reduce its permeability.
- When permeated with inorganic compounds, permeability tends to increase as the cations in the permeant are varied from monovalent to multivalent. Permeability decreases if the ionic concentration in the free pore water decreases.
- Organic compounds are more important as these materials are the major substances encountered in hazardous waste sites.
- Concentrated, pure organic solvents will interact adversely with clay and increase permeability. Collapse of the double

layer of the adsorbed cations and shrinkage cracking appear to be the mechanisms controlling the increase.

- Limited data indicate that dilute organic compounds have essentially no effect on the permeability of clays.
- Laboratory permeability determination of fine-grained materials is controversial. No permeameter appears to satisfy every requirement.
- The termination criteria for permeability tests conducted on fine-grained soil with waste permeants are not established.
- Index tests such as Atterberg limits, sedimentation, swelling, and cracking observations can provide qualitative indication of soil-chemical compatibility.
- Fly ash could be an alternative material for waste containment. However, there is a limitation to the fly ash content in the mixture beyond which the increase of fly ash shows no effect on permeability.

CHAPTER 3

RESEARCH PROGRAM

The purpose of this research is to study the effect of chemicals of various concentrations on the long term behavior of a soil-bentonite backfill mix. The experimental program included index properties determinations, permeability system development, and permeability/compatibility measurements.

The experimental program consisted of two phases. Phase I included permeability system development, index properties determinations, permeability measurements by using consolidometer permeameter and flexible wall permeameter, and the evaluation of the suitability of using fly ash as a backfill component. The works were conducted at the geotechnical laboratory of the New Jersey Institute of Technology, Newark, New Jersey.

Phase II consisted of additional tests based on the techniques developed in Phase I to further verify the experimental hypotheses. The triple ring permeameter which was developed in the first phase was modified and improved for better performance. The works were conducted at the geotechnical testing center of Dames & Moore, Cranford, New Jersey.

This chapter describes the materials, equipment, and techniques used to perform the experiments for the study.

3.1 Materials

3.1.1 Soils

Three types of soils were used. A commercially available

clean sand was selected as the coarse-grained material. It was supplied with controlled quality and contained virtually no fines. Commercially-processed materials were used because they are produced with certain standards and, therefore, ensure the consistency of the specimen.

Fine-grained materials consisted of commercial sodium bentonite and nonclay fly ash. Two types of bentonites were used, a custom sealant 200 (CS-200) and a saline seal 100 (SS-100), both supplied by American Colloid Company, Illinois. CS-200 is a powdered-type bentonite. It contains a minimum of 80 percent by weight of dry fines passing the 200 mesh. SS-100 is a granular type bentonite. It contains no more than 20 percent by weight of dry fines passing the 200 mesh. For wet particle size, both materials contain at least 94 percent by weight of fines passing 200 mesh.

According to the manufacturer, CS-200 is specifically designed for use in the slurry trench construction to reduce the flow of water. SS-100 is a chemically treated bentonite which is formulated as a "contaminant-resistant" material for use in containing wastes with high levels of dissolved salts, acids, or alkalis (American Colloid Co., 1984). These materials were selected because they have been used at many sites (Kingsbury, 1985). Table 3-1 presents typical results of chemical analyses for these materials.

The fly ash was a typical by-product of the combustion process for a power station. It was from the Mercer Generating Station and was provided by Public Service

Electric and Gas Company , New Jersey. This material was selected because of its non-plastic nature and for economic considerations and use of a waste product.

¹Table 3-1 Chemical Analyses of Sodium Bentonite

Chemical	Silica	63.02% as SiO ₂
Composition:	Alumina	21.08% as Al ₂ O ₃
	Iron (Ferric)	3.25% as Fe ₂ O ₃
	Iron (Ferrous)	0.35% as FeO
	Magnesium	2.67% as MgO
	Sodium & Potassium	2.57% as Na ₂ O
	Calcium	0.65% as CaO
	Crystal Water	5.64% as H ₂ O
	Trace Element	0.72%
	Chemical Formula:	A tri-layer expanding mineral structure of approximately: $(Al, Fe_{1.67}, Mg_{0.33})Si_4O_{10}(OH)_2Na_{+}Ca_{++0.33}$
pH:	5% Solids suspension 8.0 - 10.5	
Viscosity:	1 part bentonite to 15 parts deionized water (6.25% solids) dispersed on high speed mixer. Fann viscosimeter 7 cps. typical	
Specific Gravity	2.5	

1) Data provided by American Colloid Company, IL.

Soil specimens were prepared by mixing the above materials in various proportions. Compositions of these

admixtures are presented in Table 3-2. Figure 3-1 shows the gradation curves of the specimens.

Table 3-2 Soil Compositions for Specimen Preparation

¹ Designation	Soil Description	Specific Gravity
10SS	10% SS-100 + 90% Sand	2.62
10SS30F	10% SS-100 + 30% Fly Ash + 60% Sand	2.57
5SS30F	5% SS-100 + 30% Fly Ash + 65% Sand	2.58
5SS	5% SS-100 + 95% Sand	2.66
10CS	10% CS-200 + 90% Sand	2.62
30F	30% Fly Ash + 70% Sand	2.61
Sand	100 % Sand	2.70

1) SS: Bentonite SS-100, CS: Bentonite CS-200, F: Fly Ash

3.1.2 Chemical Liquids

Eight chemicals were selected for this study. They were diluted to a variety of concentrations to examine their effect on bentonite behavior. The concentrations selected are generally comparable to the conditions reported at various disposal sites (Keith and Telliard, 1979; Cheremisinoff and Gigliello, 1983). Table 3-3 presents the physical and chemical properties of these chemicals.

Organic chemicals and chromium were selected because they have been frequently found at various hazardous waste sites and classified as priority pollutants or toxic substances by government agencies (CFR, 1986; NJDEP, 1986).

Table 3-3 Physical and Chemical Properties of the Liquids

Liquid	Molecular Formula	Molecular Weight	Concentrations (by Volume)	Unit Weight (gm/cm ³)	Viscosity (cp)	Pr ¹	Dielectric Constant	Conductivity (µmho)
Aniline	C ₆ H ₅ NH ₂	93	500 ppm	0.975	0.934	1.14	80.06	1.3
			1000 ppm	0.973	0.954	1.17	79.73	1.4
			10000 ppm	0.969	0.974	1.20	74.11	2.1
Phenol	C ₆ H ₅ OH	94	500 ppm	0.969	0.906	1.11	80.22	1.0
			1000 ppm	0.973	0.927	1.14	80.00	1.0
			10000 ppm	0.982	0.993	1.22	76.89	2.1
			30000 ppm	0.983	1.015	1.23	70.70	4.2
			60000 ppm	0.985	1.048	1.27	63.05	6.5
Toluene	C ₇ H ₈	92	100 ppm	0.980	1.011	1.23	80.25	3.7
			500 ppm	0.978	1.020	1.24	79.67	3.2
			1000 ppm	0.978	1.031	1.25	78.96	0.7
1,1,1-Trichloroethane	C ₂ H ₃ Cl ₃	132	100 ppm	0.978	1.007	1.23	80.33	-
			500 ppm	0.978	1.011	1.23	80.07	-
			1000 ppm	0.979	1.050	1.28	79.74	-
Chromium	K ₂ CrO ₄	194	10 ppm	0.987	0.927	1.12	-	55.0
			50 ppm	1.008	0.993	1.17	-	340.0
Sodium Hydroxide	NaOH	40	0.4 %	0.987	0.930	1.12	-	-
			2.0 %	0.998	1.064	1.24	-	>20,000.0
			4.0 %	1.033	1.095	1.26	-	>20,000.0
Hydrochloric Acid	Hcl	36	0.4 %	0.999	-	-	-	-
			2.0 %	1.003	-	-	-	>20,000.0
Landfill Leachate	-	-	-	1.066	1.102	1.23	-	-
Water	H ₂ O	18	-	0.997	0.837	1.00	80.37	1.5

1) Permeant property ratio calculated in accordance with equation 2.3.

2) Values calculated based on Clausius-Mosotti Relation as recommended by Bowders (1985).

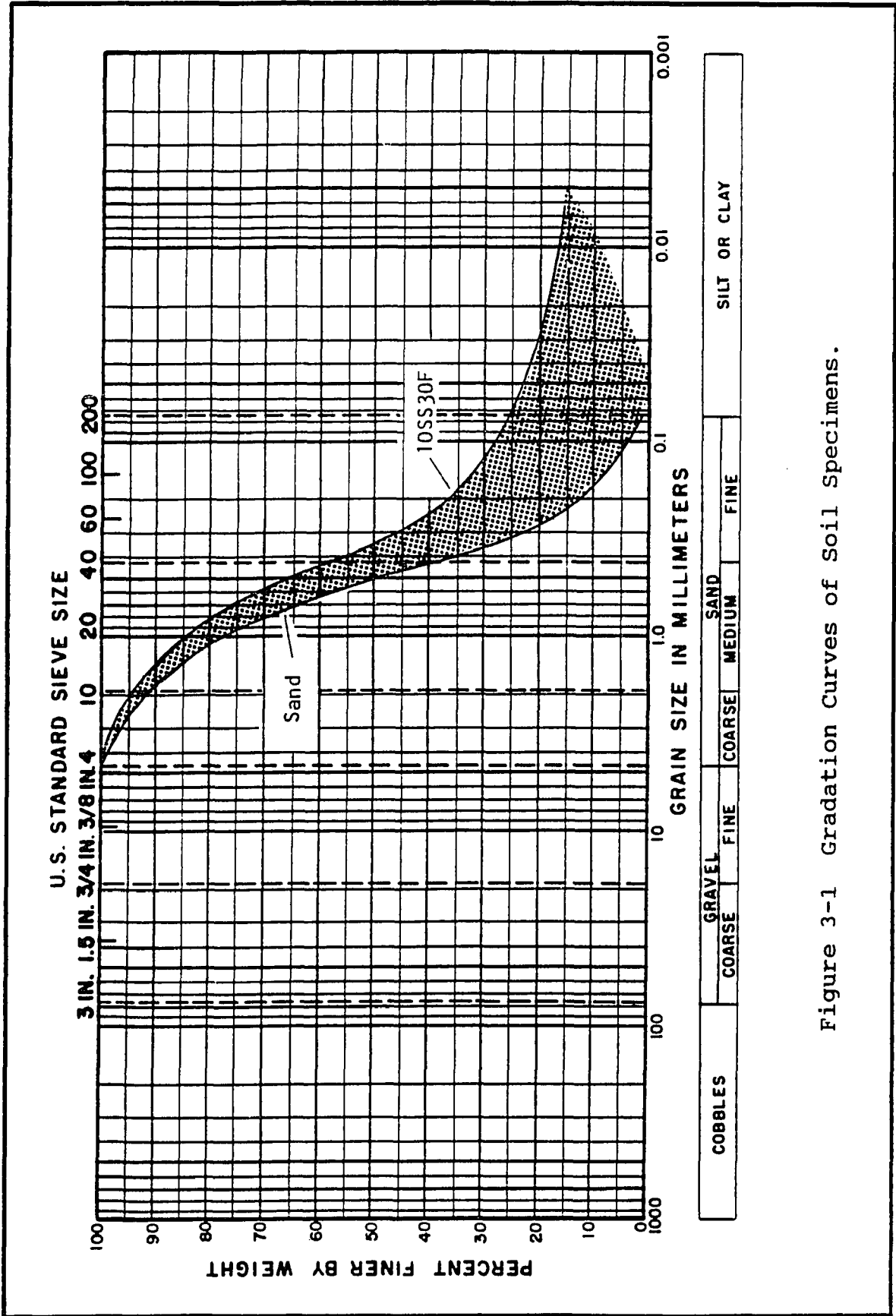


Figure 3-1 Gradation Curves of Soil Specimens.

Sodium hydroxide and hydrochloric acid were selected to observe the effects of an acid and a base on these materials.

A leachate sample collected from a municipal landfill in New Jersey was tested to evaluate the effect of mixed chemicals. The leachate contained organic compounds of a concentration of 90 ppm with methylene chloride and toluene predominantly.

Demineralized water was used for baseline values and for preparing the dilute chemicals.

3.2 Test Equipment

The permeability tests were conducted with three types of permeameter, a flexible wall permeameter (FP), a consolidometer permeameter (CP), and a triple ring permeameter (TP). This section describes each scheme of this equipment.

3.2.1 Flexible Wall Permeameter

Two types of triaxial permeameters were used. One is a conventional mercury pressurized system, originally developed by Bishop and Henkel (1953 and 1962). It is manufactured by Wykehan and Farrance Company and designed for non-hazardous soil testing only. A schematic diagram of this system is shown in Figures 3-2A and 3-2B. For detailed descriptions of this system see Head (1986) and Wu (1983).

As shown in Figure 3-3 , the other equipment is a compressed air operated system, manufactured by Brainard and Kilman. Figures 3-4A and 3-4B show the schematic diagram of this system. It is specially designed to handle the testing

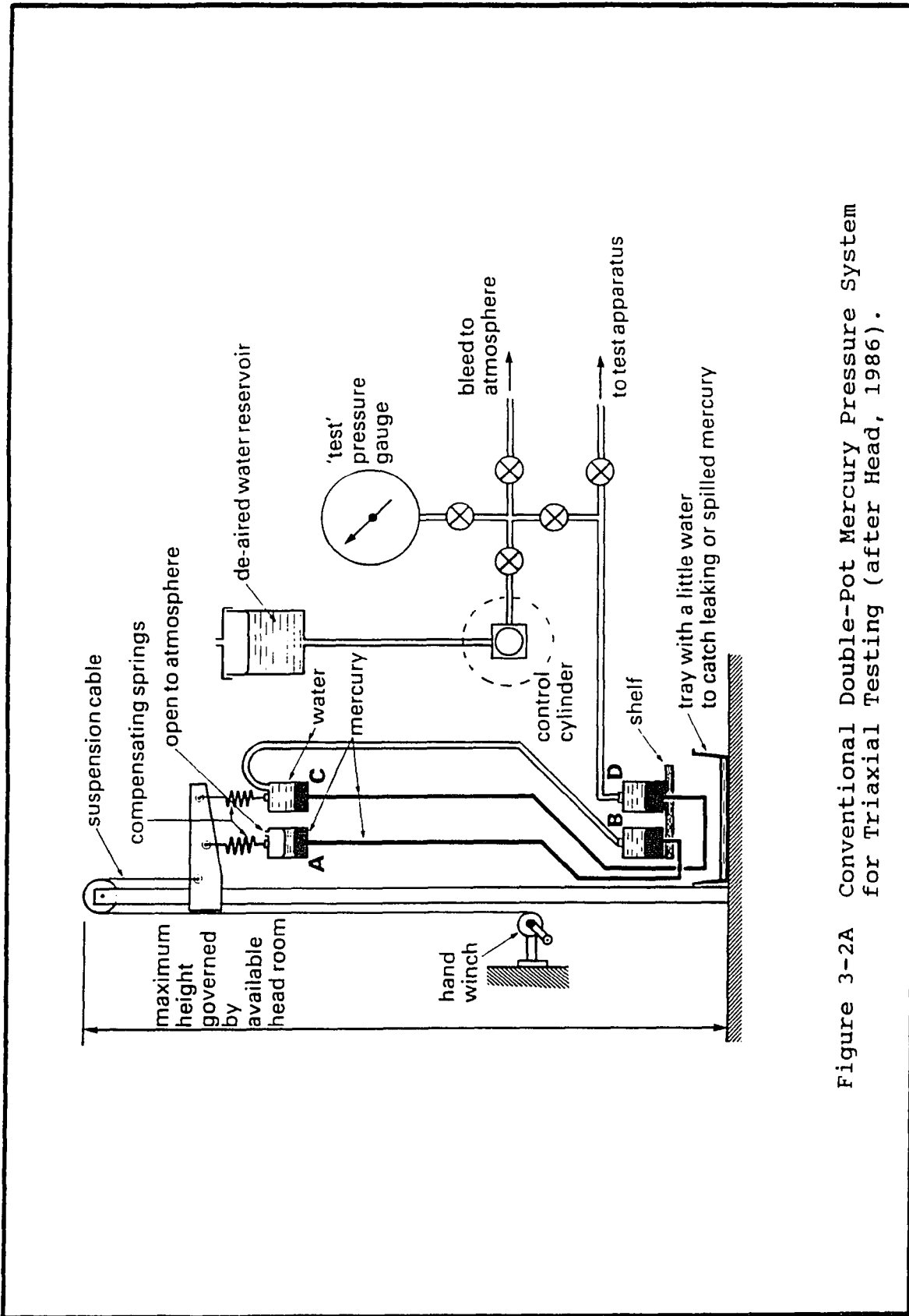


Figure 3-2A Conventional Double-Pot Mercury Pressure System for Triaxial Testing (after Head, 1986).

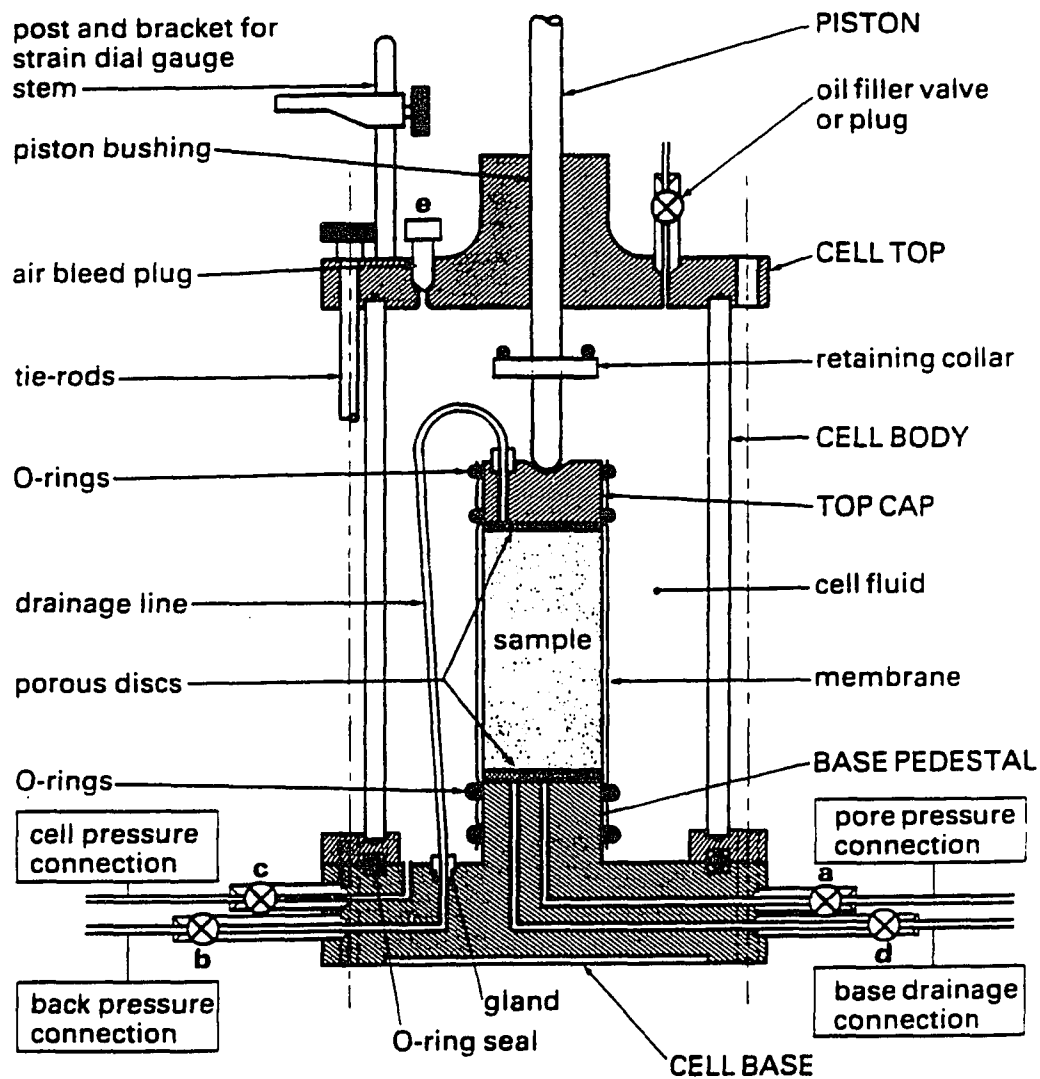


Figure 3-2B Schematic of a Conventional Triaxial Cell for Permeability Test (after Head, 1986).

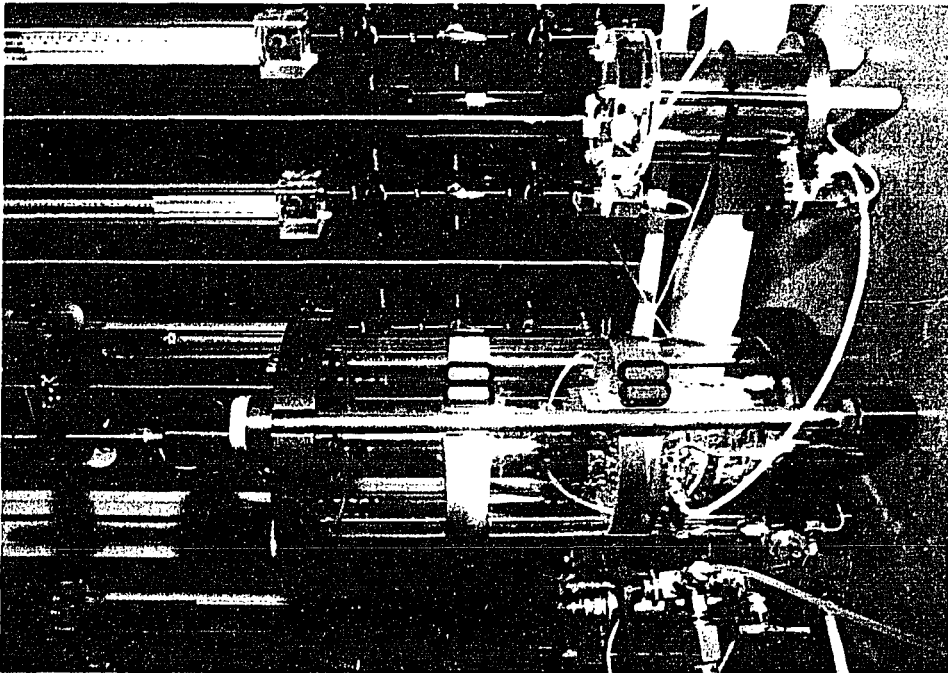
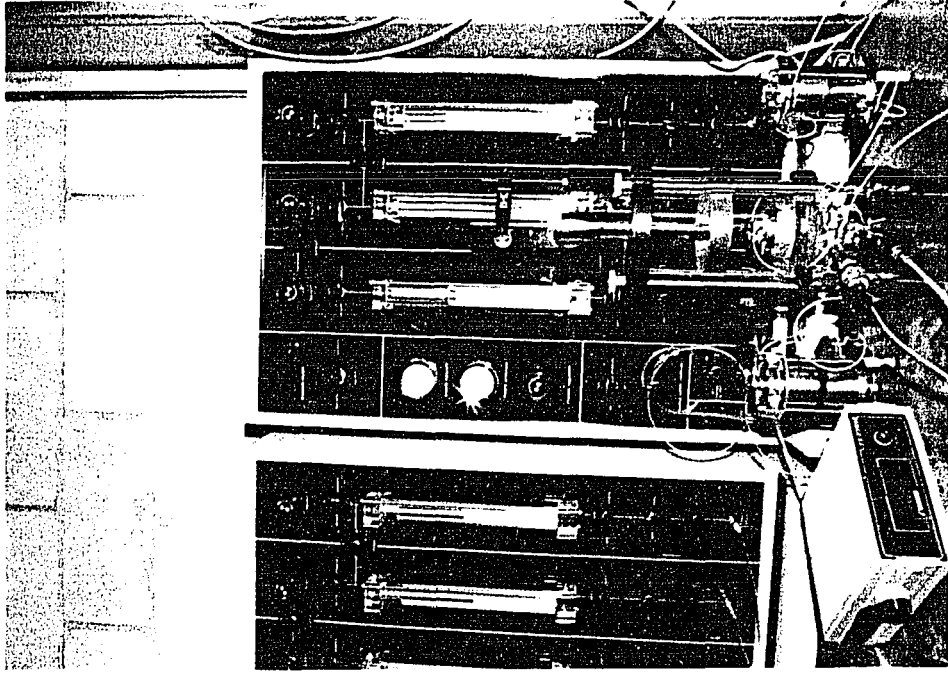
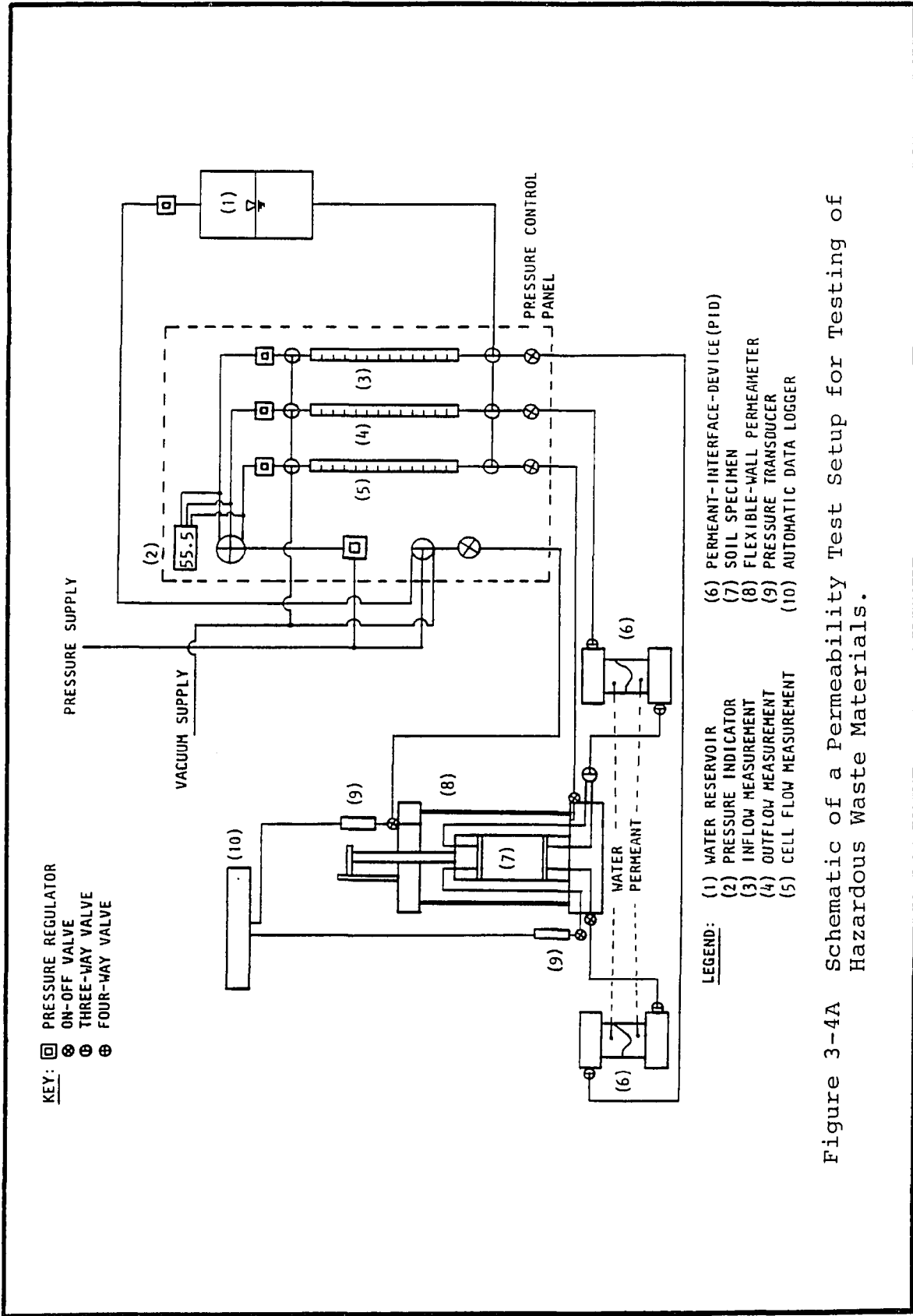


Figure 3-3 Air-operated Flexible Wall Permeability Testing System.



KEY: □ PRESSURE REGULATOR
 ⊕ ON-OFF VALVE
 ⊕ THREE-WAY VALVE
 ⊕ FOUR-WAY VALVE

LEGEND: (1) WATER RESERVOIR
 (2) PRESSURE INDICATOR
 (3) INFLOW MEASUREMENT
 (4) OUTFLOW MEASUREMENT
 (5) CELL FLOW MEASUREMENT
 (6) PERMEANT-INTERFACE-DEVICE (PID)
 (7) SOIL SPECIMEN
 (8) FLEXIBLE-WALL PERMEAMETER
 (9) PRESSURE TRANSDUCER
 (10) AUTOMATIC DATA LOGGER

Figure 3-4A Schematic of a Permeability Test Setup for Testing of Hazardous Waste Materials.

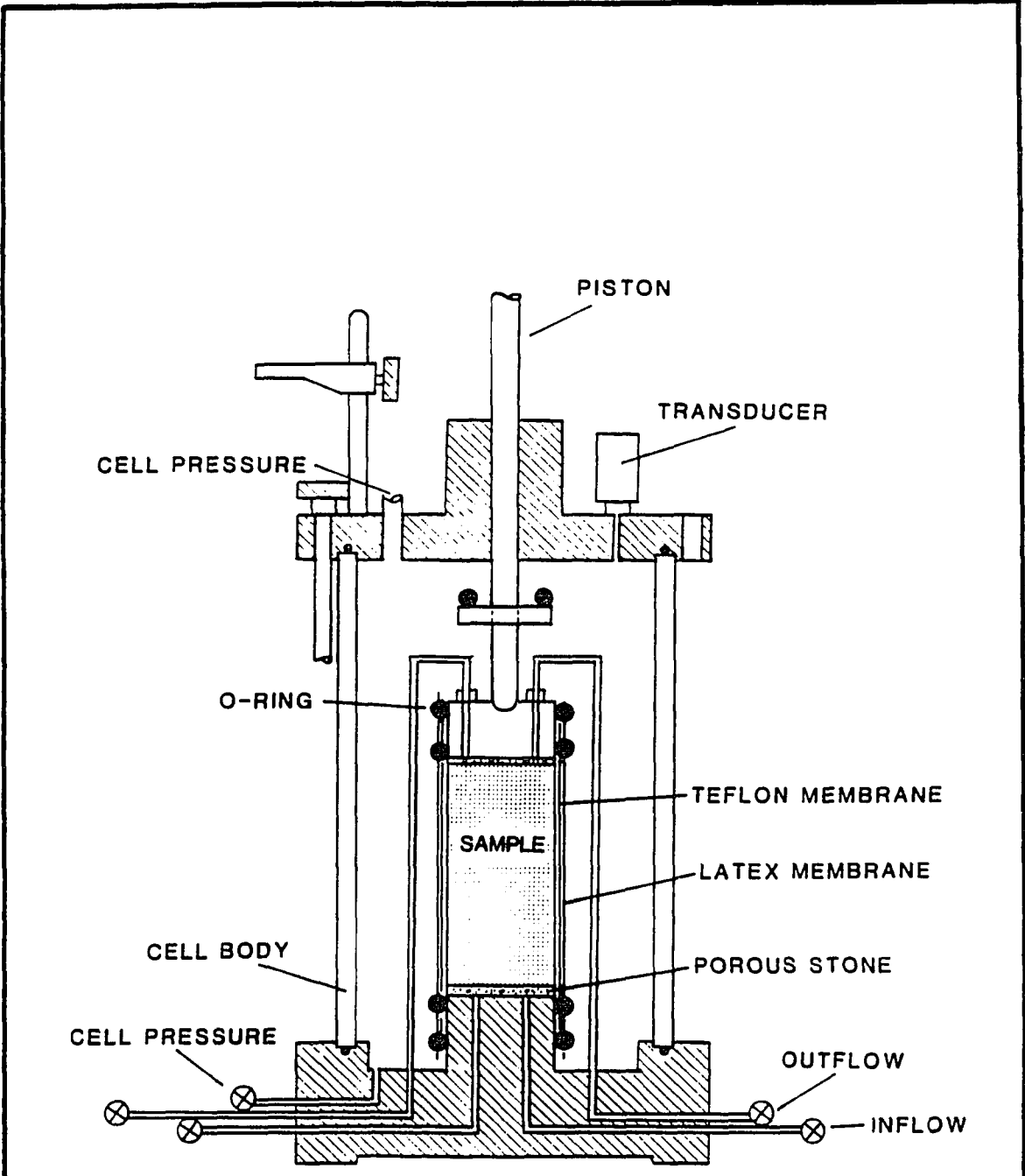


Figure 3-4B Details of a Flexible Wall Permeameter for Testing of Hazardous Waste Materials.

of contaminated materials. The components of the apparatus that come in contact with soil and permeant consist of noncorrosive materials such as stainless steel and Teflon. In addition, two bladder accumulators are installed in-line between the control panel and the permeameter to retain the toxic permeant. Either a Viton or a Buna N membrane is used to interface the permeant with water. Both materials are compatible with a variety of corrosive and toxic fluids. This in-line design prevents corrosive fluids or vapors from entering the control panel and the environments, thus minimizing the health and safety problems.

The control panel consists of pressure and vacuum supply lines, bleed-type regulators, coaxial measuring burettes, control valves, and a digital pressure readout. The system allows for vacuum saturation or backpressure saturation. Permeability can be obtained with standard triaxial testing techniques. Both inflow and outflow can be measured accurately.

3.2.2 Consolidometer Permeameter

The consolidometer permeameter used in this study is modified from a conventional fixed-ring type oedometer, manufactured by the Soil Test Company. A schematic diagram of this device is shown in Figure 3-5. The base of the cell consists of two drainage ports, conventionally used to saturate the base. By attaching a burette to one port while simultaneously plugging the other, falling head technique can be used.

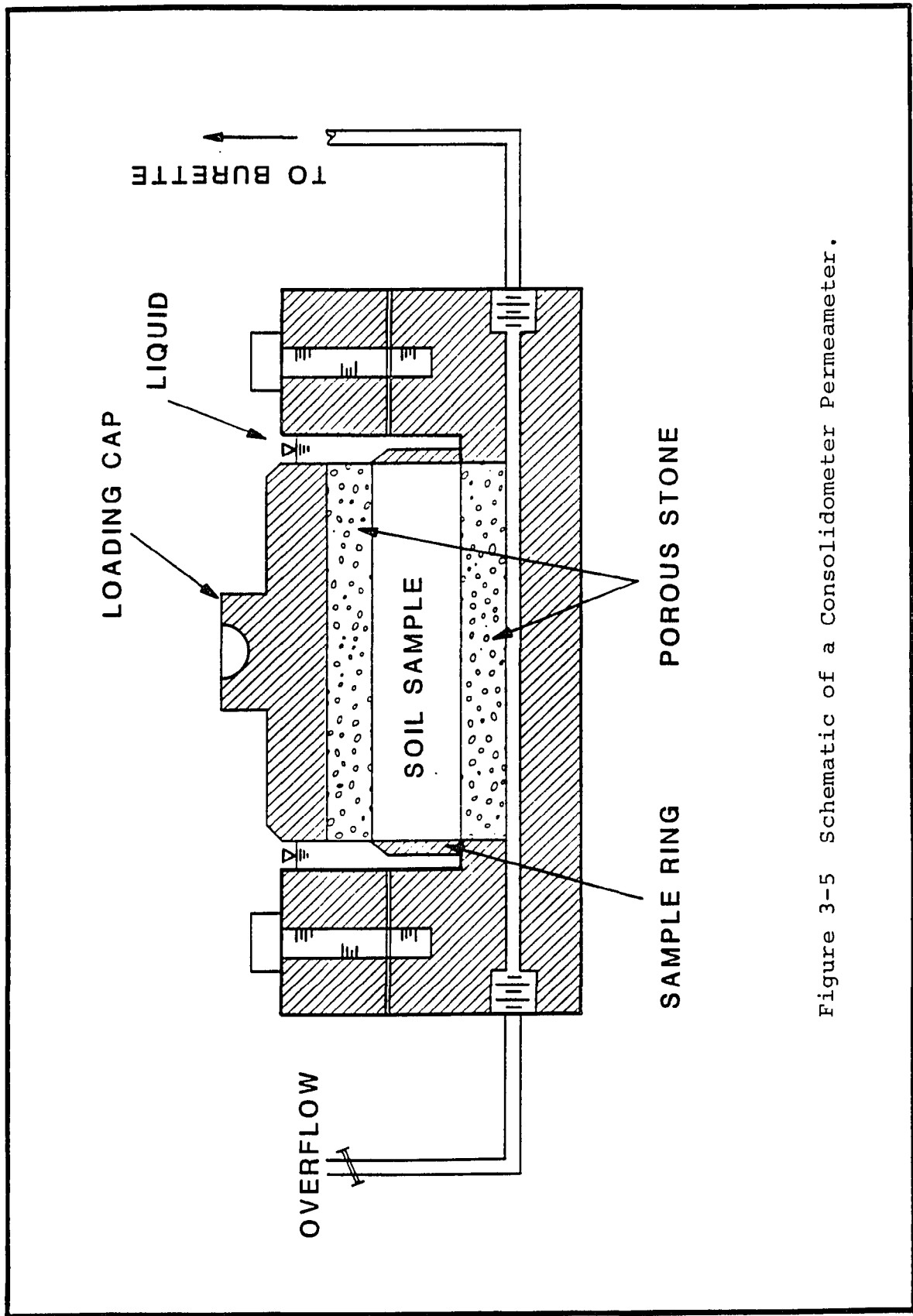


Figure 3-5 Schematic of a Consolidometer Permeameter.

3.2.3 Triple Ring Permeameter

Figure 3-6 illustrates the triple ring permeameter developed in this study. The schematic diagram of this device is shown in Figure 3-7. It is a modified version of the double ring permeameter with several improvements. All components of the permeameter that come in contact with the specimen and the permeant are made of either stainless steel or Teflon. It is similar to the Rowe consolidation cell in which loads are applied to the sample hydraulically. The hydraulic loading system gives more accurate control of applied loads over a wide range as compared with the conventional oedometer. The base has three portions with equal areas. Volume changes are measured by burettes. Five drainage lines lead to the soil specimen. Two are installed on the top cap and used for inflow. The other three lines are installed at the base pedestal to provide independent outflow measurements. If there is any leakage along the side walls, it can be detected. Furthermore, measurements from the two inner portions provide a check on the accuracy of the data.

Multiple drainage lines at both ends of the specimen provide flexibility in controlling drainage and measuring porewater pressure during the test. The extra drainage line can also be used to flush the trapped air from the system or for differential pressure monitoring.

3.2.4 Data Acquisition System

An HP 3497A data acquisition system was used to monitor the porewater pressure variation during the stages of

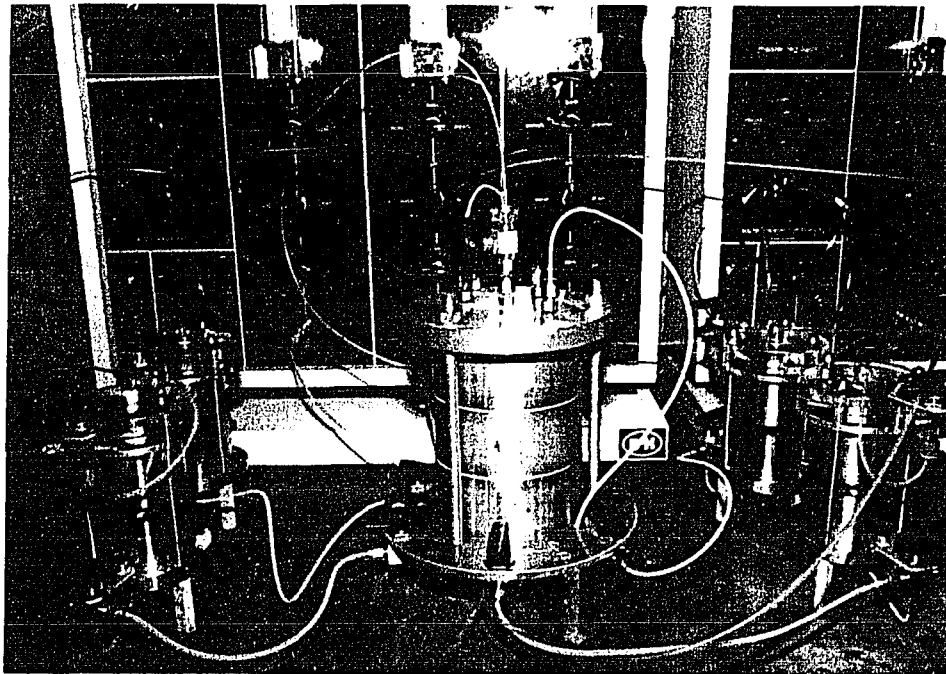
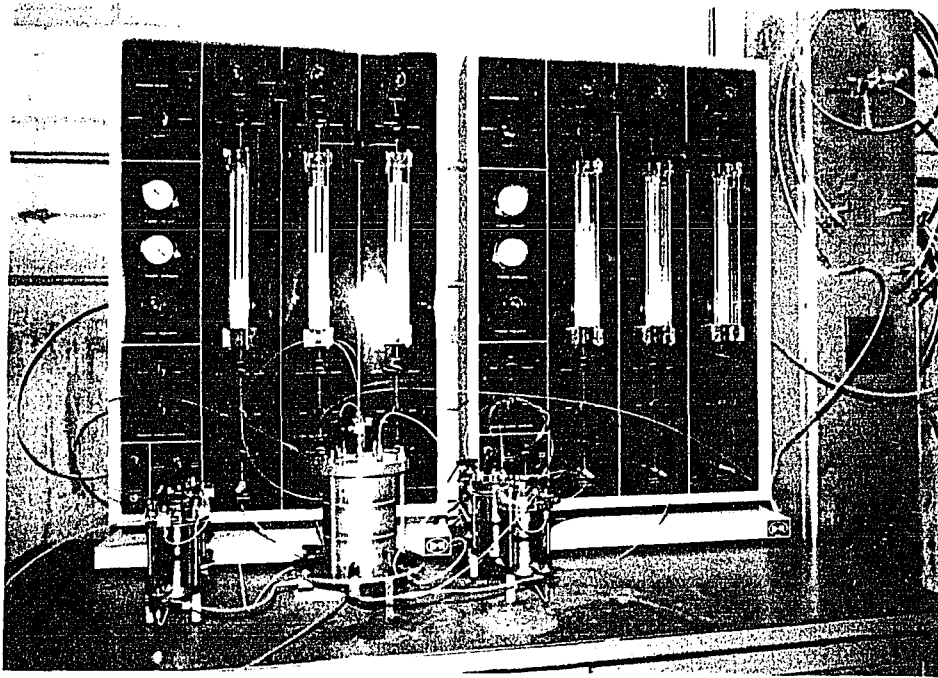


Figure 3-6 System Setup for a Triple Ring Permeability Test.

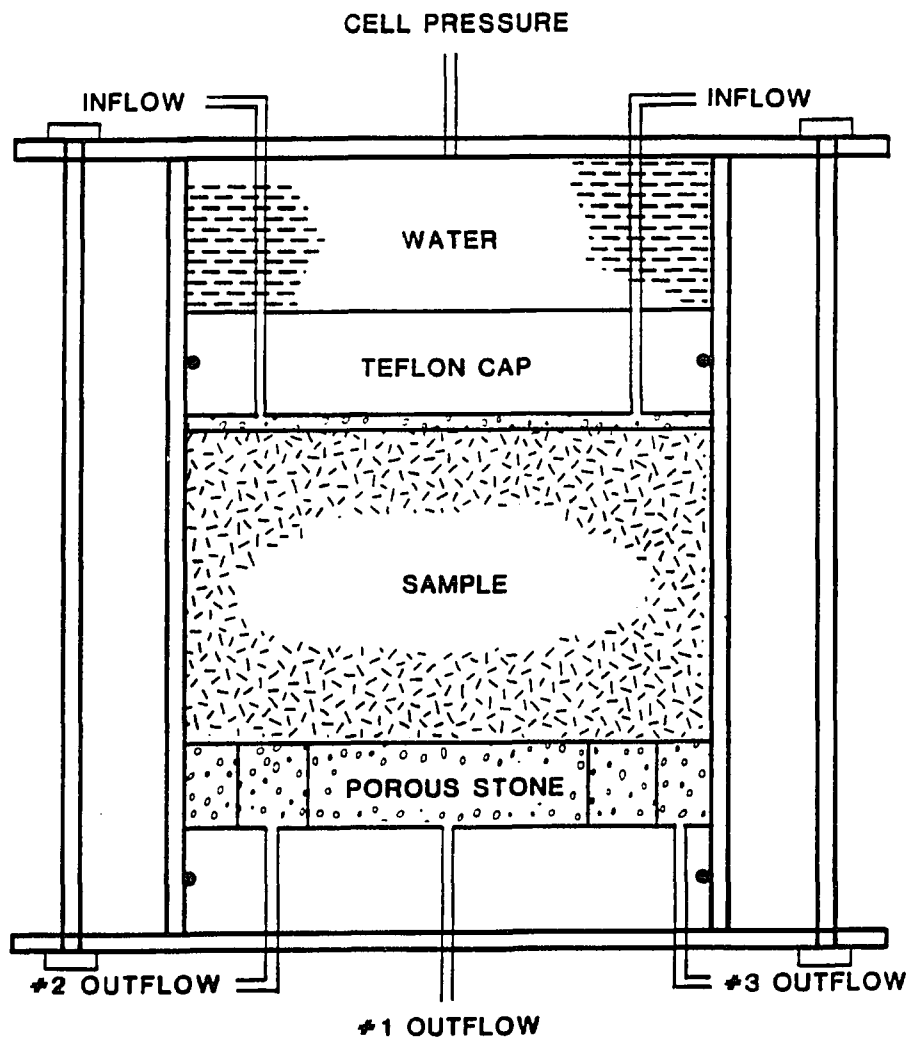


Figure 3-7 Schematic of a Triple Ring Permeameter.

saturation and consolidation. The system consists of a 20-channel multiplexer, voltmeter, and timer. Analog pressure signals are transmitted through the transducers to the system and are converted to the physical unit by the built-in program. Detailed system operations are described elsewhere and not presented here (Wu, 1983).

3.3 Experimental Program

Many researchers have conducted permeability tests using different types of permeameters. However, there are no standard testing procedures that have been established for slurry materials (Dunn, 1984; Forman, 1984). This study generally followed the procedures described by Head (1982 and 1986) with certain revised techniques developed in this study to meet the requirements of the experiment.

3.3.1 Specimen Preparation

Soil specimens were prepared by simulating the field construction. Bentonite slurry was prepared to have a viscosity of 40 sec-Marsh (15 cp). Figure 3-8 depicts that the viscosity of the slurry increases with the bentonite content for SS-100. A 40 sec-Marsh needs about 8 percent of bentonite by weight of the slurry.

The backfill mix was prepared by sluicing the soil with the slurry. Generally, it will be uniformly blended to become a paste having a slump of 2 to 6 inches. This consistency is required to ensure proper placement of the material in the trench (D'Appolonia, 1980). Dry bentonite is often added to the backfill mix prior to blending with slurry to futher

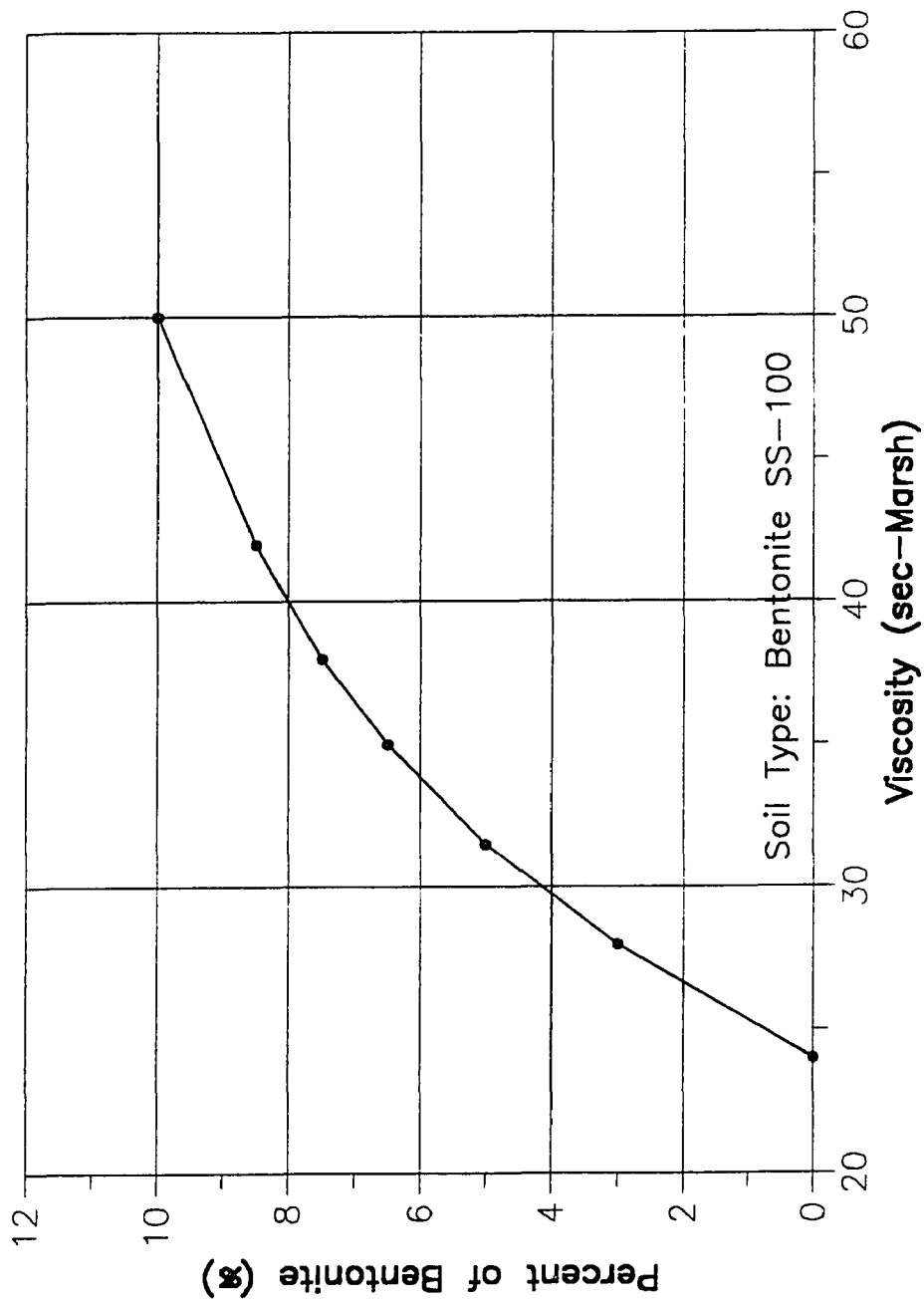


Figure 3-8 Variation of Viscosity with Bentonite Content in a Bentonite slurry.

lower the permeability. The more bentonite within a mix, the higher the water content needed to obtain the required consistency. Figure 3-9 shows the relationship between slump and water content for a backfill mix containing 6 percent, by weight, of dry bentonite. Schulze et al. (1985) reported that higher water content generally shows greater potential of permeability increase. Increased bentonite content, therefore, will not warrant a lower permeability. To achieve the desired permeability, the proportions of the backfill mix must be determined by trial batches. EPA requires that the design criteria for a waste containment is to have a field permeability of less than 10^{-7} cm/sec. To account for the uncertainties in the field, laboratory values should be at least one order of magnitude lower than those specified in the field (Dames & Moore, 1987).

Based on the results of several trial batches, a total bentonite content of 10 percent was determined for the backfill mix. This requires that an additional 6 percent dry bentonite be added to the soil before introducing the slurry. Initial study indicated that the prepared mixture showed an approximate permeability of 10^{-9} cm/sec. For a backfill mix with little fines, bentonite content as high as 10 percent is common (Schulze et al., 1985).

Slump measurement requires a much greater amount of material than needed to make a specimen. Since the slump of a backfill mix shows a linear relationship with water content at a given bentonite content, consistency dictated by the

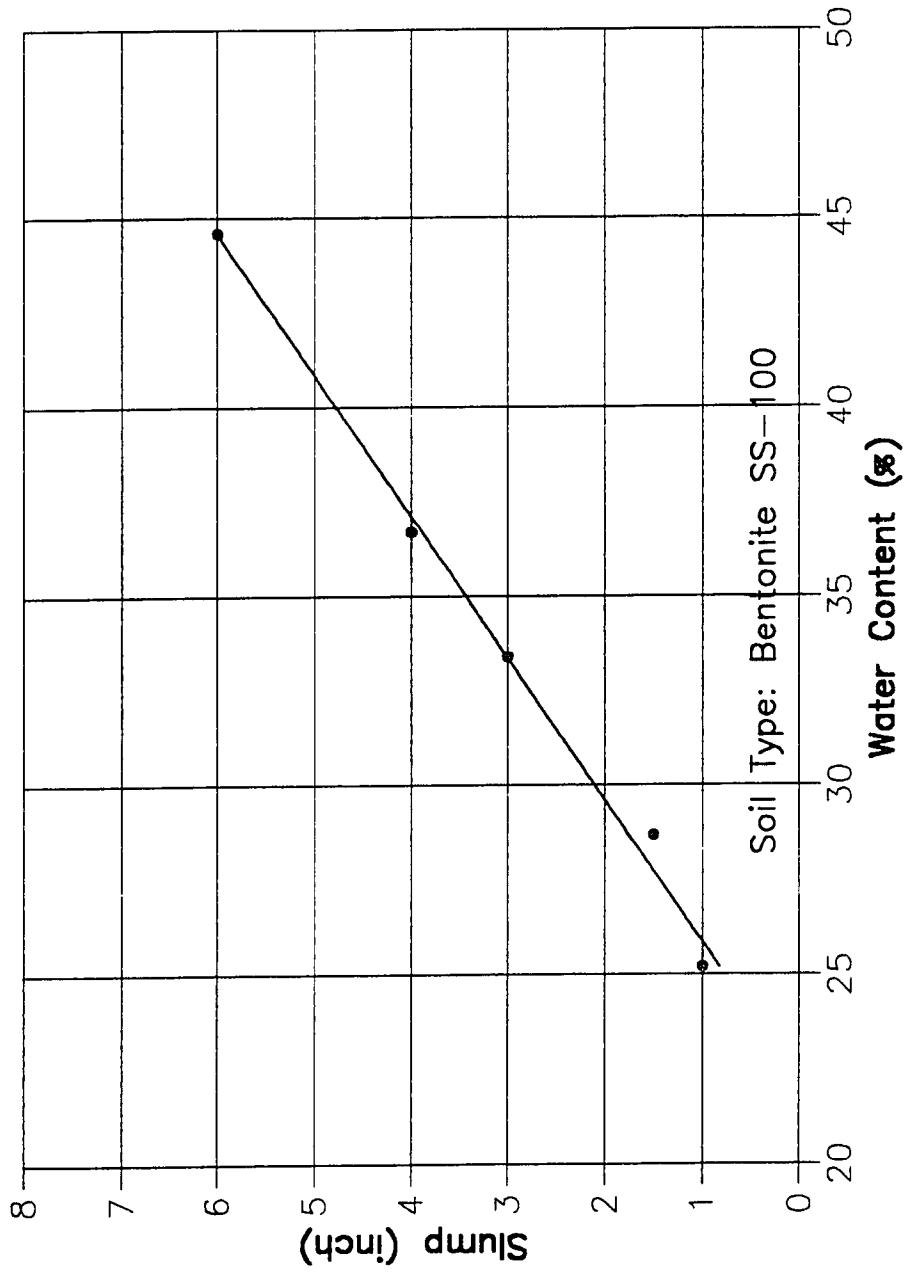


Figure 3-9 Relationship Between Water Content and Slump for a 10 Percent Bentonite Backfill Mix.

water content will be more convenient (D'Appolonia, 1980). To facilitate the specimen assembly, a 2-inch slump for the specimen was selected. This is equivalent to a water content of 30 percent. Based on the soil phase composition, the specimen should be saturated at a void ratio of 0.7 to 0.8 for this amount of water content. The bulk density of the specimen at such condition is 1.6 gm/cm^3 (100 pcf).

To stress the effect of chemicals and simulate the long term conditions, test specimens were prepared by directly saturating the soil with chemical fluids of the desired concentration. In the field, over a long period of exposure to the hazardous environment, soil is likely to be saturated with the chemicals. Use of this technique has several advantages over conventional methods. The initial pore volume in the specimen is occupied by the permeant. Soil-chemical interaction may readily occur as soon as the specimen is made. The time for permeating two or more pore volumes of flow as required in the conventional practices is thus not necessary. The specimen represents the worst condition in the field and yields a conservative test result.

3.3.2 Permeability Measurements

3.3.2.1 Flexible Wall Permeameter Flexible wall permeability tests were performed in a standard triaxial cell. The prepared specimen was encased in a flexible membrane and sealed with a rubber O-ring to the top cap and bottom pedestal. The cell was filled with water, and the

specimen was subjected to backpressure saturation in small increments. The Skempton porewater parameter B was constantly monitored. The specimen was saturated when the B value had reached 0.98 or greater. Three stages of consolidation pressures, typically 50, 100, 200 kPa were applied. At the completion of each consolidation, the specimen was permeated with an upward gradient. The gradient was created by raising the pressure at the influent end and simultaneously lowering the pressure equally at the effluent end. The typical pressure difference is about 25 to 50 kPa. For a typical specimen height of 5 to 7 cm, such pressure will have a hydraulic gradient less than 100. The volume changes of inflow and outflow were recorded periodically.

The permeability for water was determined when the flow quantity versus the time curve became linear (Head, 1986). For chemical permeants, the test was terminated when the rates of inflow and outflow became equal (Daniel et al., 1984). The specimens were initially saturated with chemical fluids, and bentonite had been hydrated for at least 48 hours before permeability was measured. Therefore, the conventional termination criteria for at least one to two pore volumes displacement as suggested by Peirce and Witter (1985) were not necessary.

The latex membrane used to confine the specimen is vulnerable to chemical attack (Acar et al., 1984). As neoprene has more chemical compatibilities than latex (NY DEC, 1984), neoprene membrane was used in an attempt to replace

the latex membrane. Preliminary trial indicated that the neoprene membrane is too stiff to seal around the specimen for the relatively low confining stresses applied. Bulk flows occurred and led to erroneous permeability measurements. To protect the latex membrane, a Teflon sheet, 6-inch wide and 0.002-inch thick, was used to separate the membrane from the specimen as recommended by Daniel et al. (1984). The Teflon sheet has some rigidity, and it may show problems similar to those shown by the neoprene membrane.

Preliminary trials on specimen wrapped with one to two revolutions of Teflon sheet indicated that the Teflon sheet was flexible enough to remain pressed against the soil for the given confining stresses. When the Teflon sheet was dismantled at the end of the test, ridges were observed around the specimen surface that matched exactly the wrinkles of the Teflon sheet. These ridges prevents the wrinkles from acting as flow channels and mislead the permeability measurement.

To assemble a slurry specimen in a triaxial cell requires considerable effort. The conventional technique of using a vacuum to maintain the shape of a granular specimen is not applicable because of the impervious nature of the bentonite material. A plastic former combined with a perforated sheet of aluminum foil were used to facilitate the specimen assembly. As the slurry mix was filling in, the former was gradually withdrawn. The aluminum foil around the specimen, outside the membrane, supports the specimen and

prevents it from distorting.

3.3.2.2 Consolidometer Permeameter In the consolidometer permeability test, a predetermined amount of slurry mix was placed into the sample ring. The material was slightly tamped to reach the required density. The consolidation cell was then filled with testing liquid to keep the specimen saturated. A seating pressure of 2 kPa was applied to remove the possible trapped air. Consolidation pressures were applied generally with double increments. The typical pressures used were 5, 10, 25, 50, 100 and 200 kPa. During consolidation, time-deformation data were recorded. Coefficients of consolidation were calculated based on the logarithmic time fitting method. Indirect permeability values were obtained according to Terzaghi's theory. Upon completion of each consolidation, permeant was introduced through the attached burette. Permeability was determined by the falling head techniques.

3.3.2.3 Triple Ring Permeameter The specimen assembly for the triple ring permeability test was similar to that for the consolidometer permeability test. To saturate the specimen, backpressure was gradually increased to a maximum pressure of 210 kPa, and porewater pressure parameter B was constantly monitored. A slight gradient was applied to promote saturation. Initial tests were conducted with an effective pressure of 50 kPa applied to the specimen. However, later studies indicated that the consolidation pressure tends to render lower permeability than may be found in the field. An

effective pressure of 10 kPa was then used to keep the sample stable during the test. Permeant was introduced with a typical gradient of less than 100. The outflows were collected at the three outlets at the bottom of the cell. The cumulative volume of each outflow was monitored to observe if there was any sidewall leakage. The tests were terminated when the rates of inflow and outflow became equal.

3.3.3 Index Tests

3.3.3.1 Atterberg Liquid Limit Tests Atterberg liquid limit tests were performed on bentonite SS-100 in accordance with ASTM D-4318. Liquids used in the tests included water and various concentrations of aniline, toluene, and phenol. Teflon spray was used to protect the apparatus.

Dry bentonite was initially mixed with liquid at an amount approximately equals to liquid limit. One group of samples was tested immediately. Other groups of samples were sealed and cured for a period of one day, two days, and one week before the tests were conducted.

3.3.3.2 Sedimentation Tests Sedimentation tests were conducted similar to a standard hydrometer test as outlined in ASTM D-421 and D-422. A 151H hydrometer was used for measurements. Air dry bentonite, SS-100, was dispersed thoroughly in the designated testing liquid. A dispersing agent was not used to disclose the effect of chemicals on the bentonite. The suspension behavior of bentonite was observed, and hydrometer readings were taken periodically for 60 days. Liquids used in the tests included water and different

concentrations of aniline, phenol, 1,1,1-trichloroethane, sodium hydroxide, and hydrochloric acid.

3.3.3.3 Expansion Tests The expansion behavior of bentonite is highly dependent upon its chemical environment. When saturated with water, bentonite could expand its volume to 10-to-15-fold of its dry bulk (American Colloid Co., 1984). Expansion behavior was observed by placing an initial dry volume of 5 ml of bentonite into a graduated cylinder and adding liquid to a volume of 100 ml. Because of the impervious nature of the bentonite, soil and liquid must be dropped into the cylinder alternatively to ensure complete saturation. The soaked samples were sealed and the volume changes were recorded for a period of about one year. Bentonite SS-100 and CS-200 were tested. Liquids used in the tests included different concentrations of aniline, phenol, 1,1,1-trichloroethane, toluene, chromium, sodium hydroxide, hydrochloric acid, and landfill leachate.

3.3.3.4 Cracking Observations Cracking observations were made on bentonite SS-100 with water and various concentrations of sodium hydroxide. Tests were conducted by placing one gram of dry bentonite into an evaporating dish and filling with sufficient amount of liquid to submerge the soil. The specimen was then allowed to dry at room temperature and cracking conditions were observed for about four weeks.

3.4 Statistical Data Analysis

Statistical correlation is a useful technique in data

analysis when one variable tends to influence or affect another variable (Gulezian, 1979). To analyze the correlation between two variables, x and y , Pearson's correlation coefficient, $r(x,y)$, is helpful. It can be interpreted as a measure of the strength of the linear relationship of two variables. It is a number between 1 and -1. If $r(x,y)$ is equal to 1, the two variables have perfect positive correlation. This implies that small values of x are associated with small values of y . When $r(x,y)$ is equal to -1, it indicates that a perfect negative correlation exists between the two variables. This means that small values of one variable are associated with large values of another and vice versa. If $r(x,y)$ is close to zero, the two variables are uncorrelated (Milton and Arnold, 1986).

To determine the relationship between a sample correlation coefficient and the true population coefficient, the student's t -distribution statistic is used for a small sample size of n and t is defined as follows:

$$t = (r)(n - 2)^{0.5} / (1-r^2)^{0.5} \quad (3.1)$$

For a chosen sample size and level of significance, α , t may be calculated and compared with the critical values (CV) listed in the standard table. The critical values are defined as follows:

$$CV_1 = -t_{\alpha/2} \quad \text{and} \quad CV_2 = t_{\alpha/2} \quad (3.2)$$

The null hypothesis, H_0 , that there is no correlation between x and y may now be tested at a particular confidence interval to determine if it is true. The following decision-

rule is used for the testing:

Accept H_0 when $CV_1 < t < CV_2$

Reject H_0 when $t < CV_1$ or $t > CV_2$ (3.3)

Based on the decision rule, the null hypothesis of no correlation can be accepted when the t-statistic falls between the tabulated critical values. It can be rejected and concluded that correlation exists when the t-statistic is either less than or equal to the lower critical value, $-t_{\alpha/2}$, or greater than or equal to the upper critical value, $t_{\alpha/2}$.

The statistical correlation technique is applied to analyze the relationships of void ratio, consolidation pressure, and permeability values. Permeability is known to be strongly correlated with void ratio for most of the soils, and void ratio is highly related to consolidation pressure (Mesri and Olson, 1971 and Samarasinghe et al., 1982). Use of statistical technique allows further delineation of their relationships.

In addition to Pearson's correlation method, the Spearman rank test was also used for the analysis. The Spearman rank correlation coefficient, R , is a nonparametric alternative to the Pearson correlation coefficient. It can be used regardless of the shape of the population, and thus can be applied under much more general conditions (Siegel, 1956 and Dowdy and Wearden, 1983). The procedures to determine the relationship between the sample correlation coefficient and the true population coefficient are similar to those described for Pearson's method. The R value may be directly

compared with critical values given in the table and eliminate the need to perform the t-test.

To perform the statistical data analysis, this study used a computer statistical package, STAT+. The program was developed by StatSoft, Inc. For detailed description of this program see STAT+ (1988).

CHAPTER 4

PRESENTATION AND DISCUSSION OF TEST RESULTS

This chapter presents the test results for all the experiments performed in this study. It consists primarily of summary tables and plots. Detailed results of permeability tests and statistical analysis are presented in Appendices A and B, respectively. Discussions of the test results and an interpretation of various observations are included.

The first section evaluates the findings of the index tests. These tests included Atterberg liquid limit, sedimentation, expansion, and cracking observations. The second section presents the results and discussion of the permeability measurements. Tests performed included consolidometer permeability tests, flexible wall permeability tests, and triple ring permeability tests.

4.1 Index Tests

The index tests were conducted to evaluate the possibility of using simple indices to qualitatively and quantitatively predict the behavior of bentonite in the contaminated liquids. The results of the evaluation are used to propose criteria which are then used to evaluate the influence of contaminated liquids on the permeability of bentonite mixtures.

4.1.1 Atterberg Liquid Limit Tests

The liquid limit tests were performed on bentonite SS-100 with diluted aniline, phenol, toluene, and water. The test results are presented in Table 4-1 and plotted in

Figures 4-1A through 4-1D. Comparisons of the liquid limit obtained with organic liquids to that obtained with water were used as the basis for classifying the effect of a liquid on the soil.

Table 4-1 Summary of Liquid Limit Test Results¹

Type of Liquid	Concentration (ppm)	Instant	1-Day	2-Day	7-Day
Water	-	459	-	-	-
Aniline	500	388	348	463	372
	1000	384	372	367	359
	10000	293	315	315	304
Phenol	500	369	360	363	347
	1000	369	354	356	354
	10000	366	354	356	349
Toluene ²	100	398	389	424	378
	500	380	373	376	351
	1000	393	374	378	354
Toluene ³	100	346	364	358	338
	500	350	361	352	344
	1000	351	344	359	327

1) Tests were conducted on bentonite SS-100 only.

2) First run of the test.

3) Second run of the test.

Most of the liquids tested have shown some reductions in the liquid limit when compared with water. The dielectric constants of all the liquids are less than that of water. In accordance with the Gouy-Chapman theory, a lower dielectric constant may cause the soil structure to be flocculated. A more flocculated soil structure tends to have a smaller double layer and thus lesser amount of liquid will be held by the soil particles. In general, the higher the concentration,

LIQUID LIMIT TEST

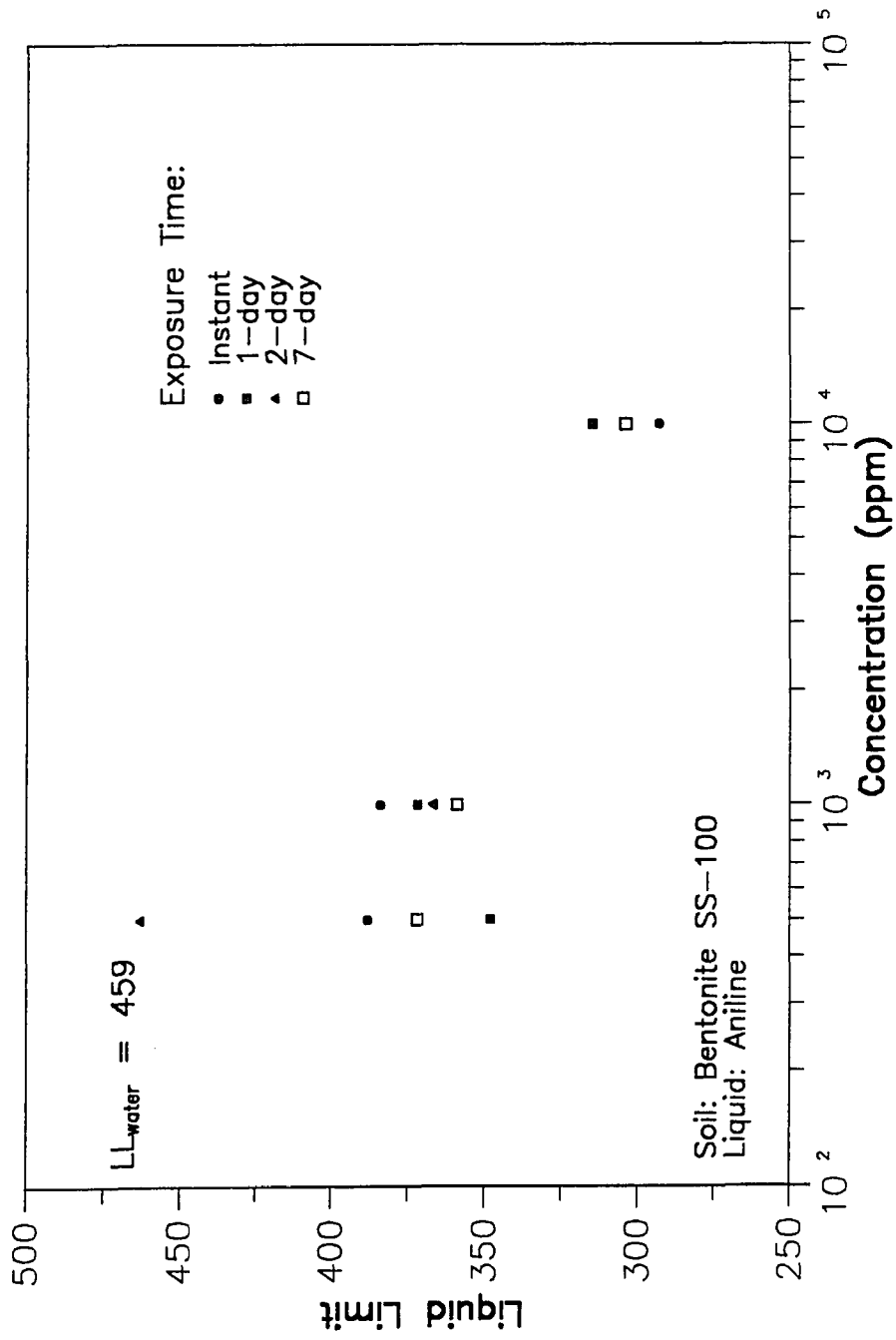


Figure 4-1A Liquid Limit of Bentonite SS-100 with Aniline.

LIQUID LIMIT TEST

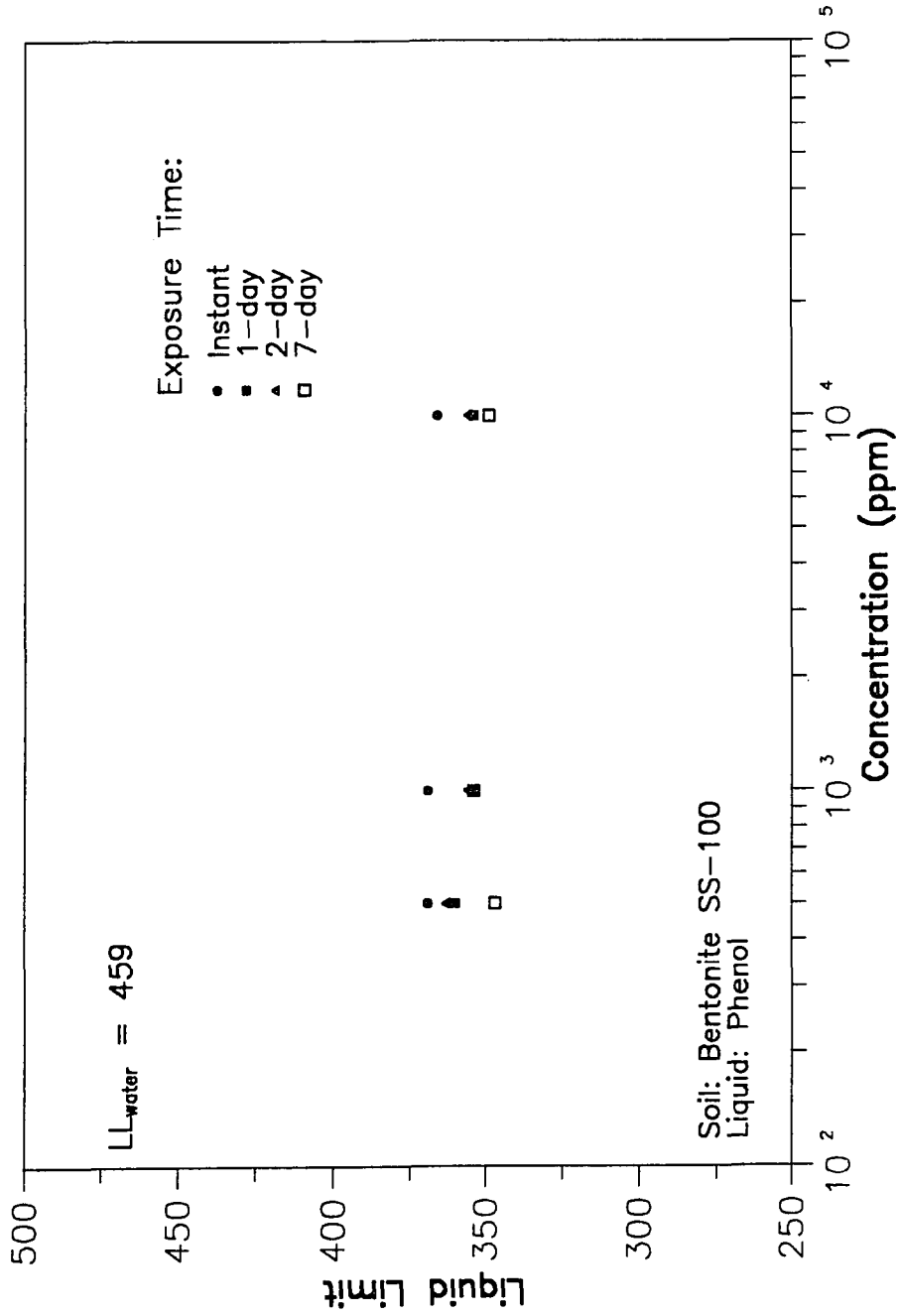


Figure 4-1B Liquid Limit of Bentonite SS-100 with Phenol.

LIQUID LIMIT TEST

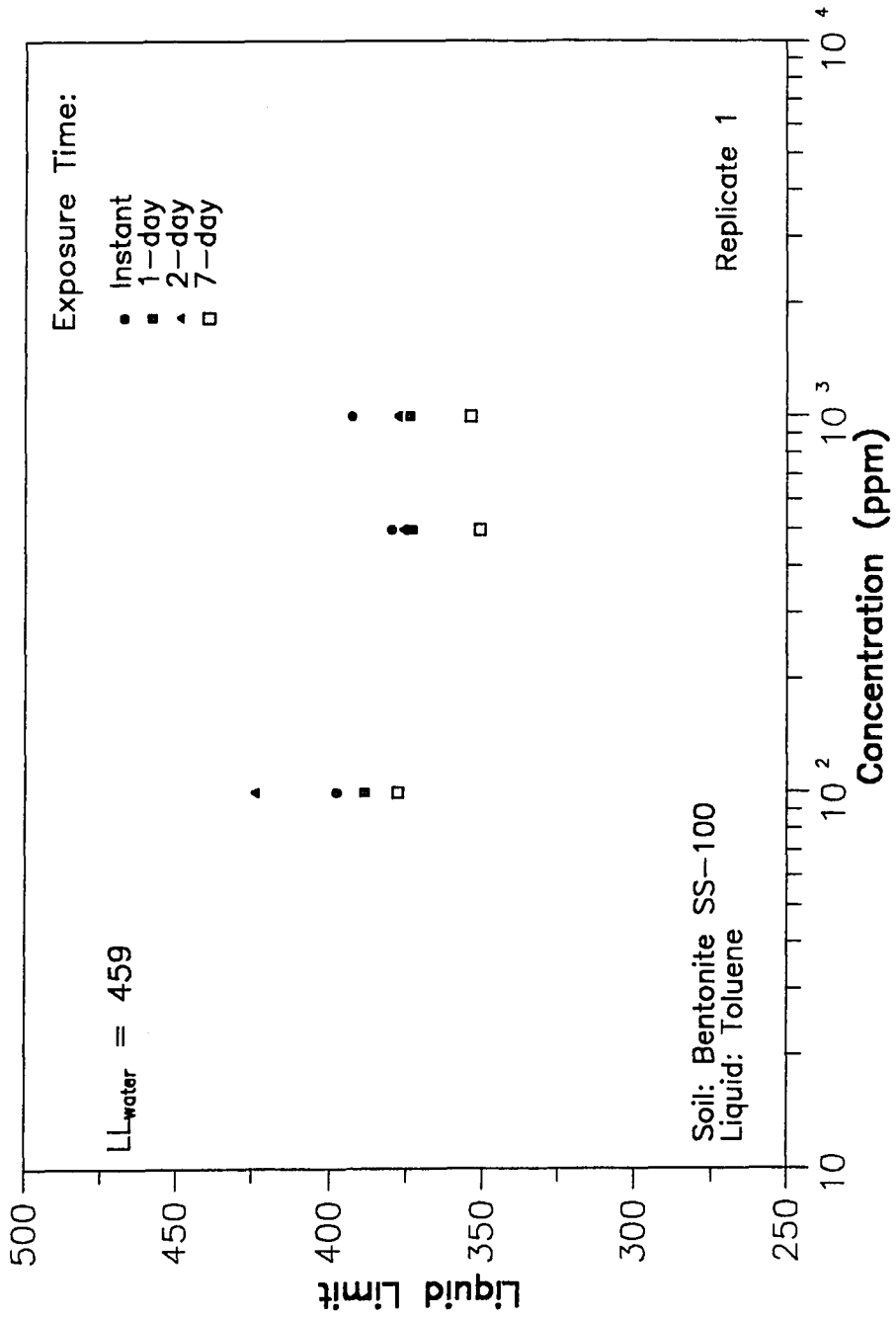


Figure 4-1C Liquid Limit of Bentonite SS-100 with Toluene (Replicate 1).

LIQUID LIMIT TEST

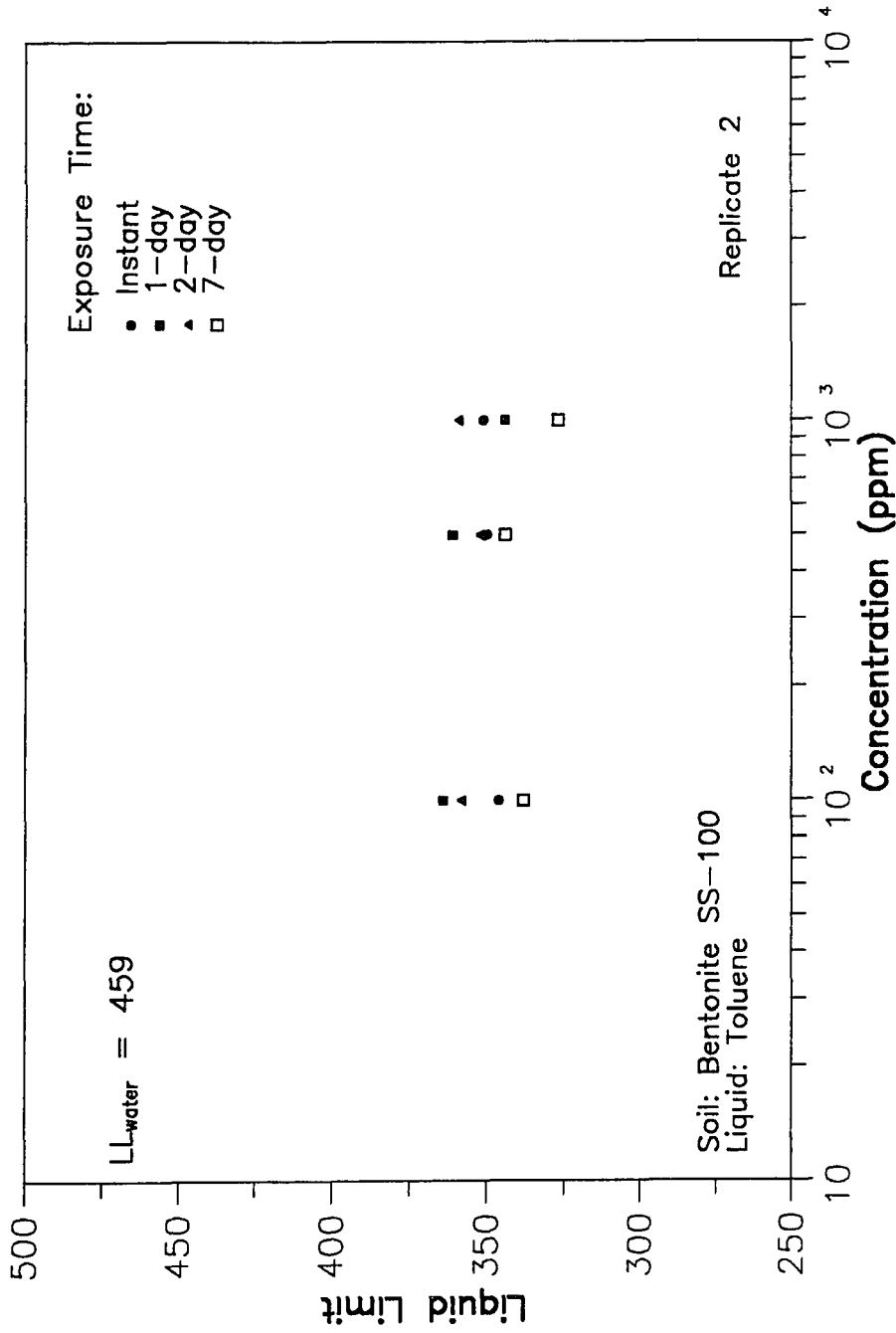


Figure 4-1D Liquid Limit of Bentonite SS-100 with Toluene (Replicate 2).

the more the reduction in the double layer. For aniline, the liquid limit decreases clearly as the concentration increases. However, this is not true for toluene and phenol. The dielectric constants of these chemicals range from 74 to 80. Considering the small difference, the results appear to be reasonable.

Dielectric constant was reported to be a useful index to correlate the effect of organic solvents on the clayey materials. Acar et al. (1985) presented the test results of liquids of various dielectric constants on the liquid limit of sodium montmorillonite. They found the liquid limit was about 400 when tested with water for a dielectric constant of 80. The liquid limit dropped with the decrease of dielectric constants. It was only about 68 when tested with pure phenol for a dielectric constant of 13 and was about 60 when tested with aniline for a dielectric constant of 7.

To observe the effect of exposure time, specimens were soaked in the liquids for one day, two days, and one week. The test results are erratic and show no definite correlation between the soaking period and the liquid limit. To confirm the results, additional observation was made for toluene. The results again show considerable spread. The scatter in data may be due to the physical properties of the chemicals and the test methodology. Though no conclusion can be drawn for the effect of exposure time on the liquid limit, the test values are not substantially different from the values with water. Some researchers performed liquid limit tests on

various clay soils with different diluted liquids (Daniel and Liljestrand, 1984; and Bowders, 1985). The results indicated that there was negligible alteration of the limits caused by the diluted liquids. Bowders (1987) concluded that chemicals which do not change the soil liquid limit may show little potential to alter the structure of the soil or cause an increase in its permeability.

The performance of liquid limit tests requires certain practice and effort to ensure the accuracy of the test results. Oven drying the soil at 110°C may lead to erroneous moisture determinations as the boiling point of each liquid may be significantly different (Bowders, 1985). Therefore, it cannot be used with relative convenience and reliability to predict the effect of chemicals on bentonite. Additional liquid limit tests were not performed for this reason.

4.1.2 Expansion Tests

Free expansion is a modified term, originally defined as free swell by Gibbs and Holtz (1956) and stated by Head (1984) as:

$$\text{Free expansion, } F_e = (V_f - V_i) / V_i \text{ in percent} \quad (4.1)$$

where: V_i = initial dry volume of the soil corresponding to the loosest state,

V_f = final volume of the expanded soil in liquid.

The term swelling is generally used in relation to the soil deformation that resulted from pressure reduction (Mitchell, 1976). It was considered inappropriate to be used herein as no vertical stress was applied during the expansion

measurements.

To facilitate comparisons of the test results, the free expansion value of soil for each liquid was normalized by its expansion value with water at corresponding time intervals. Specific expansion, S_e , is defined as the normalized value.

$$(S_e)_t = (F_{ec}/F_{ew})_t \quad (4.2)$$

where: F_{ec} = soil free expansion with the chemical;
 F_{ew} = soil free expansion with demineralized water,
 t = elapsed time since the start of the expansion test.

4.1.2.1 Free Expansion The expansion behavior of bentonite with various chemicals is presented in Figures 4-2A through 4-2H for SS-100 and Figures 4-3A through 4-3F for CS-200. Bentonite SS-100 with water expanded gradually and its F_e reached a constant value of 1,080 percent at about 100 days. When saturated with the chemicals, F_e generally dropped as the concentration increased.

As shown in Figure 4-2A, F_e of aniline 500 ppm showed a value of 1,040 percent. The value dropped by about two-thirds as the concentration increased to 10,000 ppm; that is about 56 percent of the value of water. In Figure 4-2B through 4-2D, toluene, phenol, and 1,1,1-Trichloroethane demonstrated some effect on the soil expansion but their changes were not as significant as that for aniline. Note that even when the concentration of phenol was 10,000 ppm, there was not much change in F_e .

EXPANSION TEST

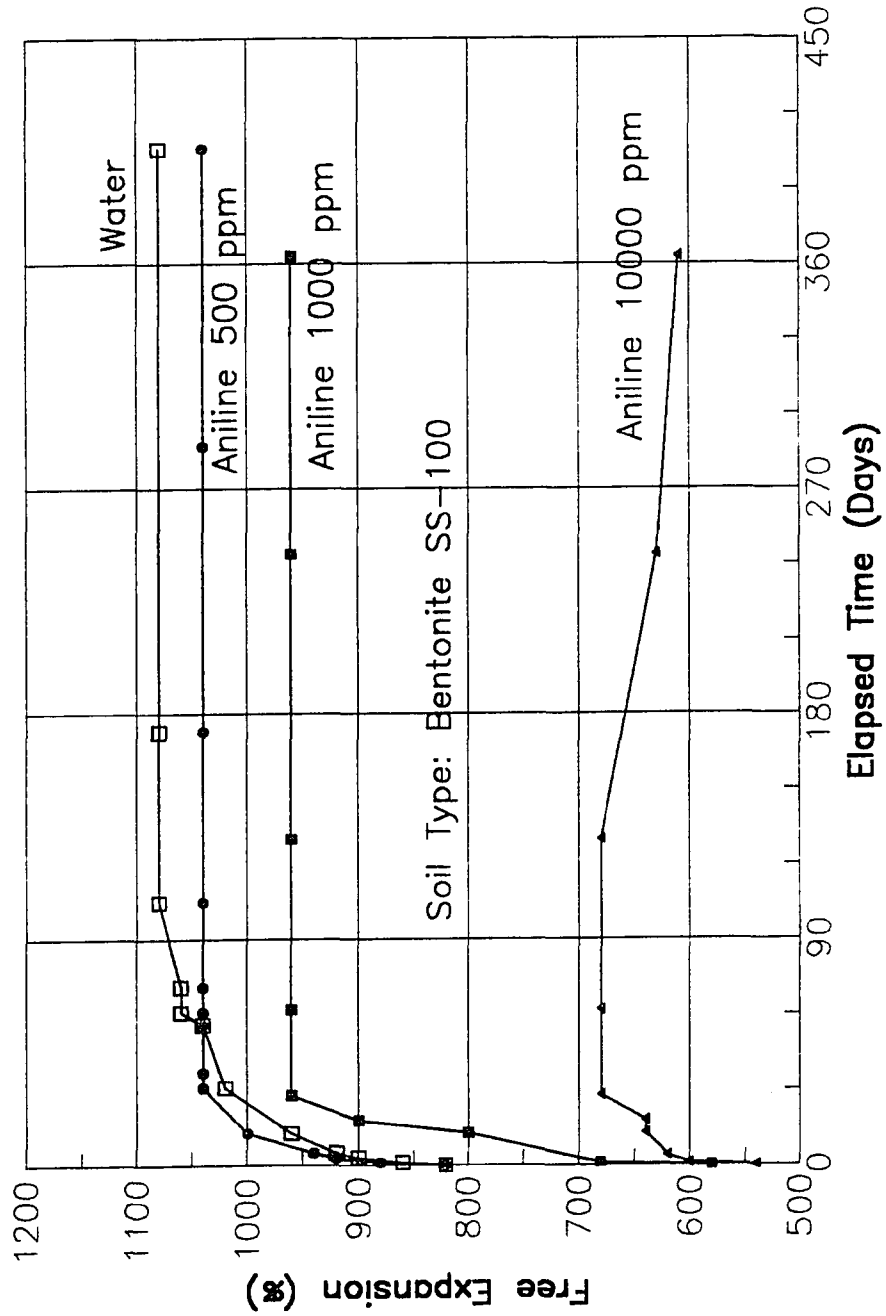


Figure 4-2A Free Expansion of Bentonite SS-100 with Aniline.

EXPANSION TEST

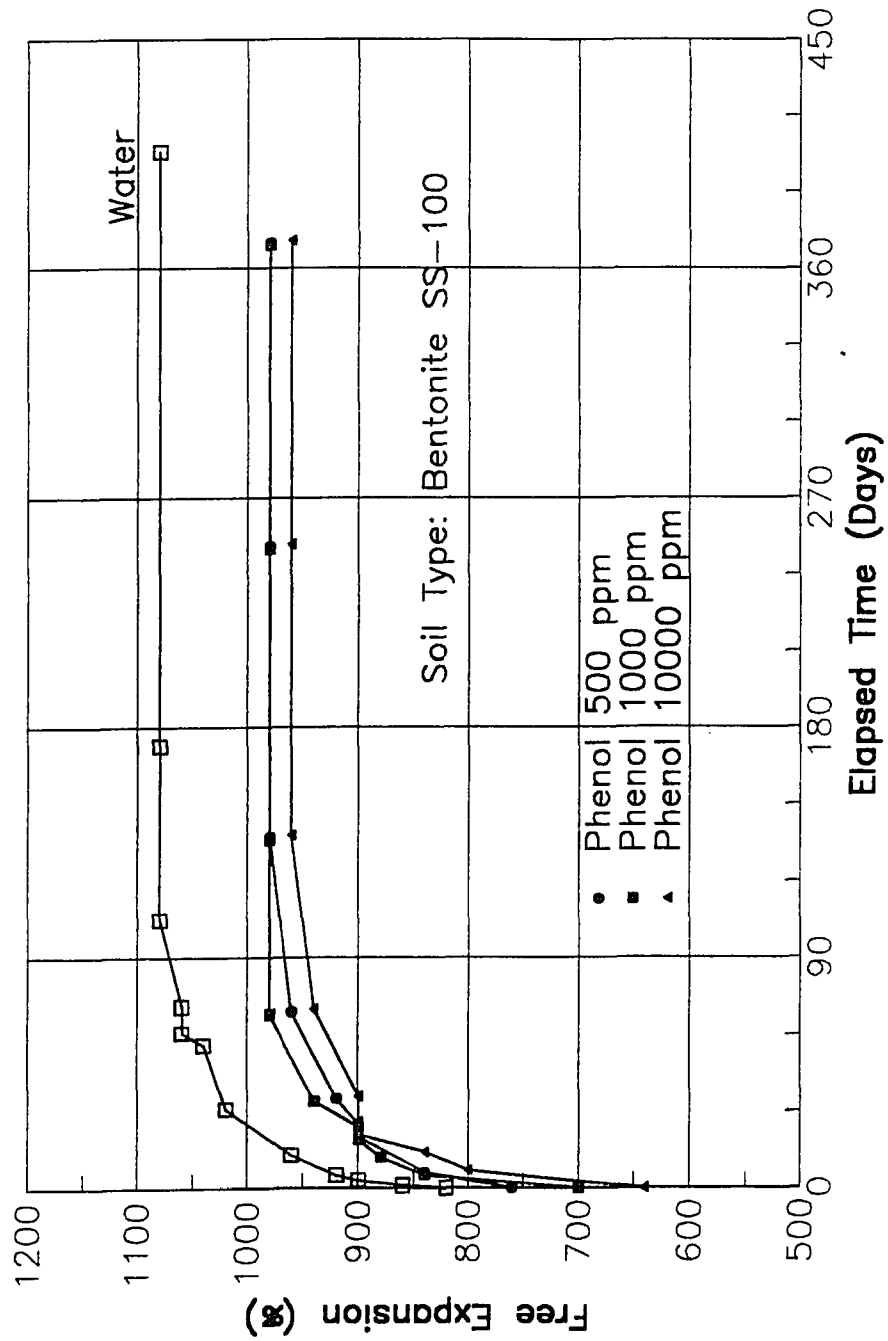


Figure 4-2B Free Expansion of Bentonite SS-100 with Phenol.

EXPANSION TEST

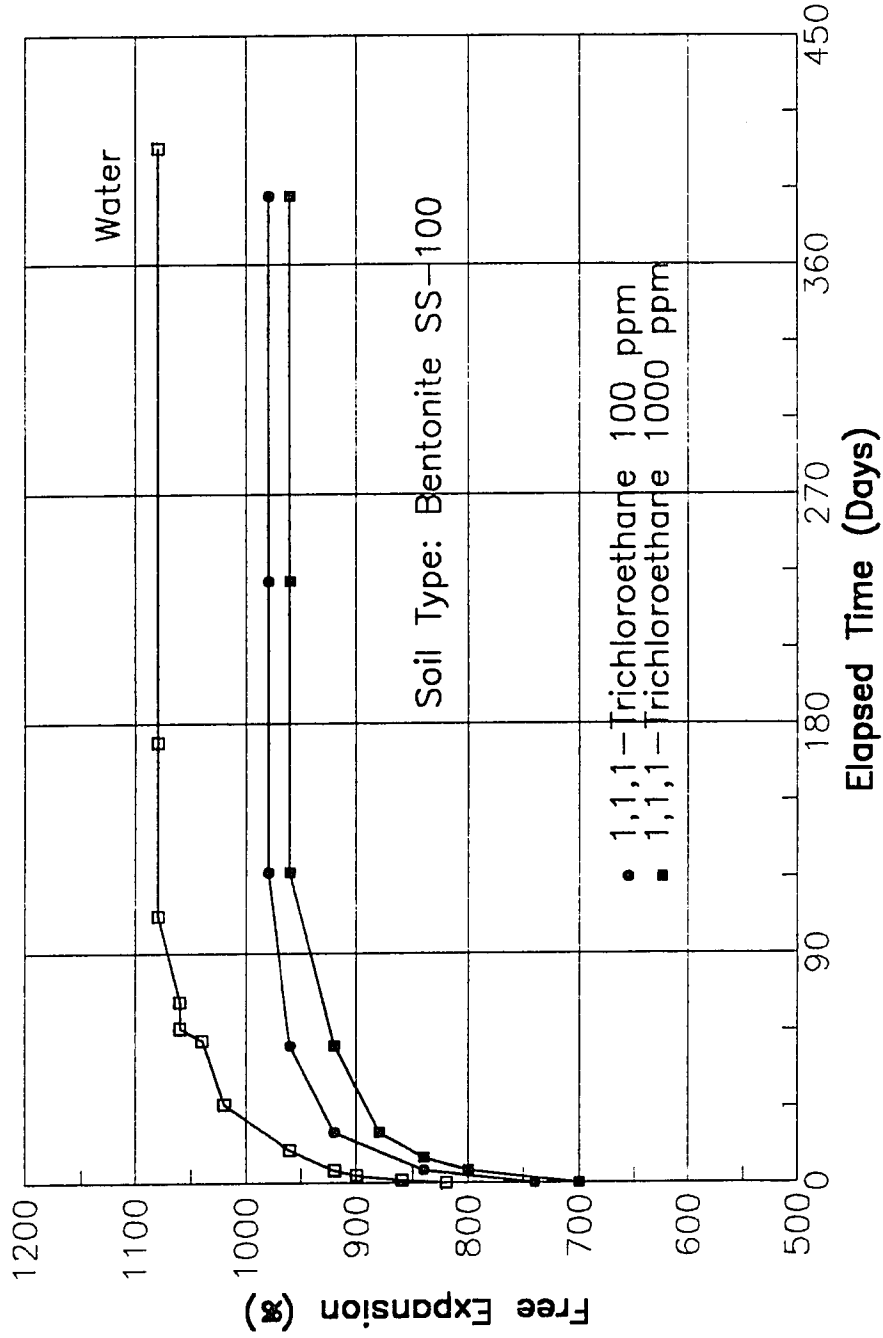


Figure 4-2C Free Expansion of Bentonite SS-100 with 1,1,1-Trichloroethane.

EXPANSION TEST

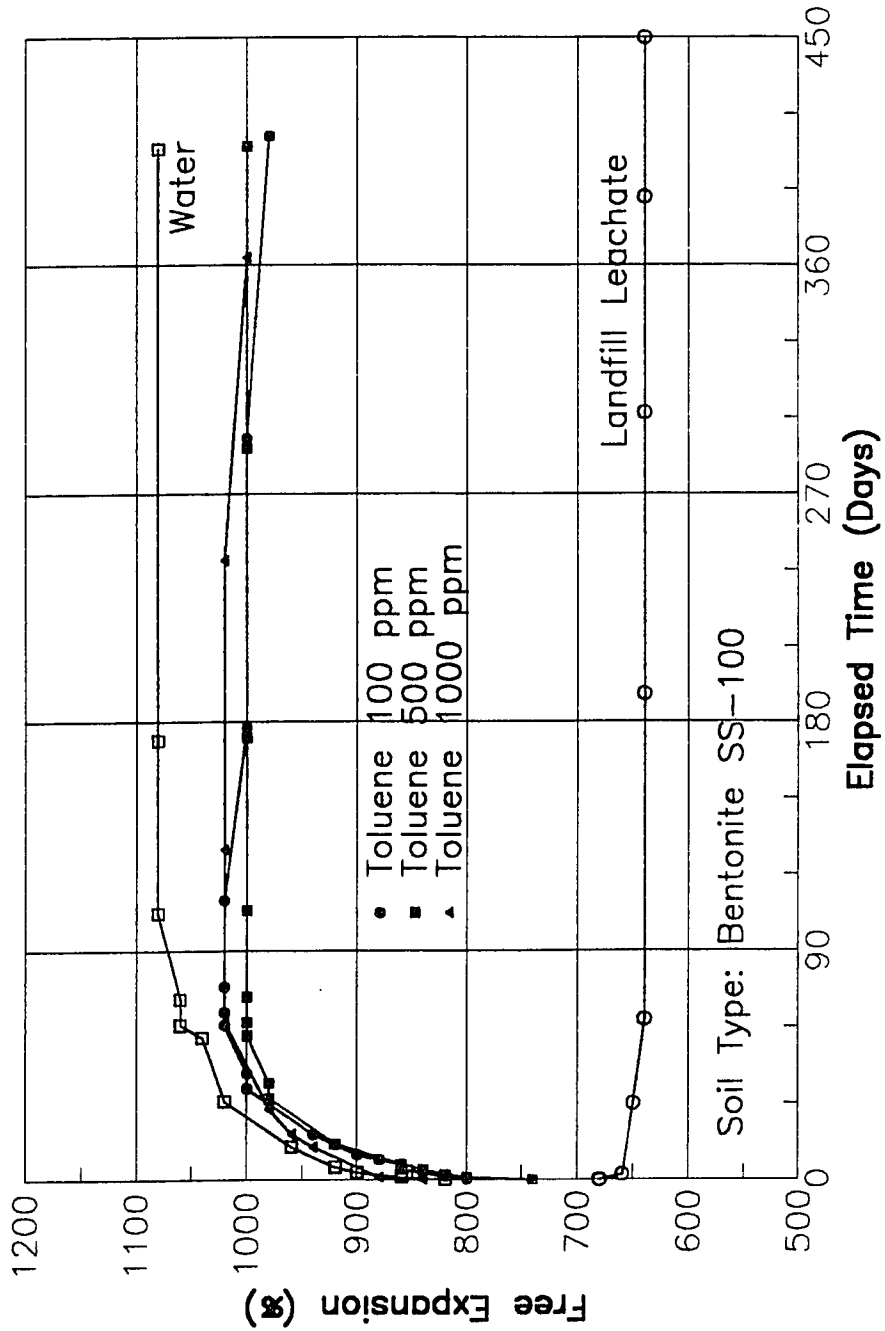


Figure 4-2D Free Expansion of Bentonite SS-100 with Toluene and Landfill Leachate.

As noted earlier, the dielectric constants of these chemicals decrease with increasing concentration. Therefore, the lesser of the expansion could be attributed to the reduction of the dielectric constant. Acar et al. (1985) performed similar observations on the expansion of sodium montmorillonite with pure organic fluids. They found that soil expansion generally increased with the dielectric constant of the pore fluid. The test results agree well with their findings.

The expansion behavior of SS-100 with a municipal landfill leachate is presented in Figure 4-2D. The leachate contains predominantly methylene chloride and toluene with a total organic content of 90 ppm. The pH value is 5.0. In this liquid, the expansion of the bentonite was considerably inhibited. The final value of F_e was only 640 percent which was about 64 percent of the value of toluene 1000 ppm. This is believed to be the result of the composite effects of the mixed chemicals. Mitchell and Madsen (1987) stated that there is very little information available about the behavior of mixed chemical solutions and that their effects on the clay properties should be studied case by case.

Figure 4-2E shows the expansion behaviors of SS-100 with all the organic liquids at a concentration of 1000 ppm. Though there are some variations in the test results, their differences are not significant. As noted earlier, the dielectric constants of these chemicals are close to each other; therefore, the results are reasonable.

EXPANSION TEST

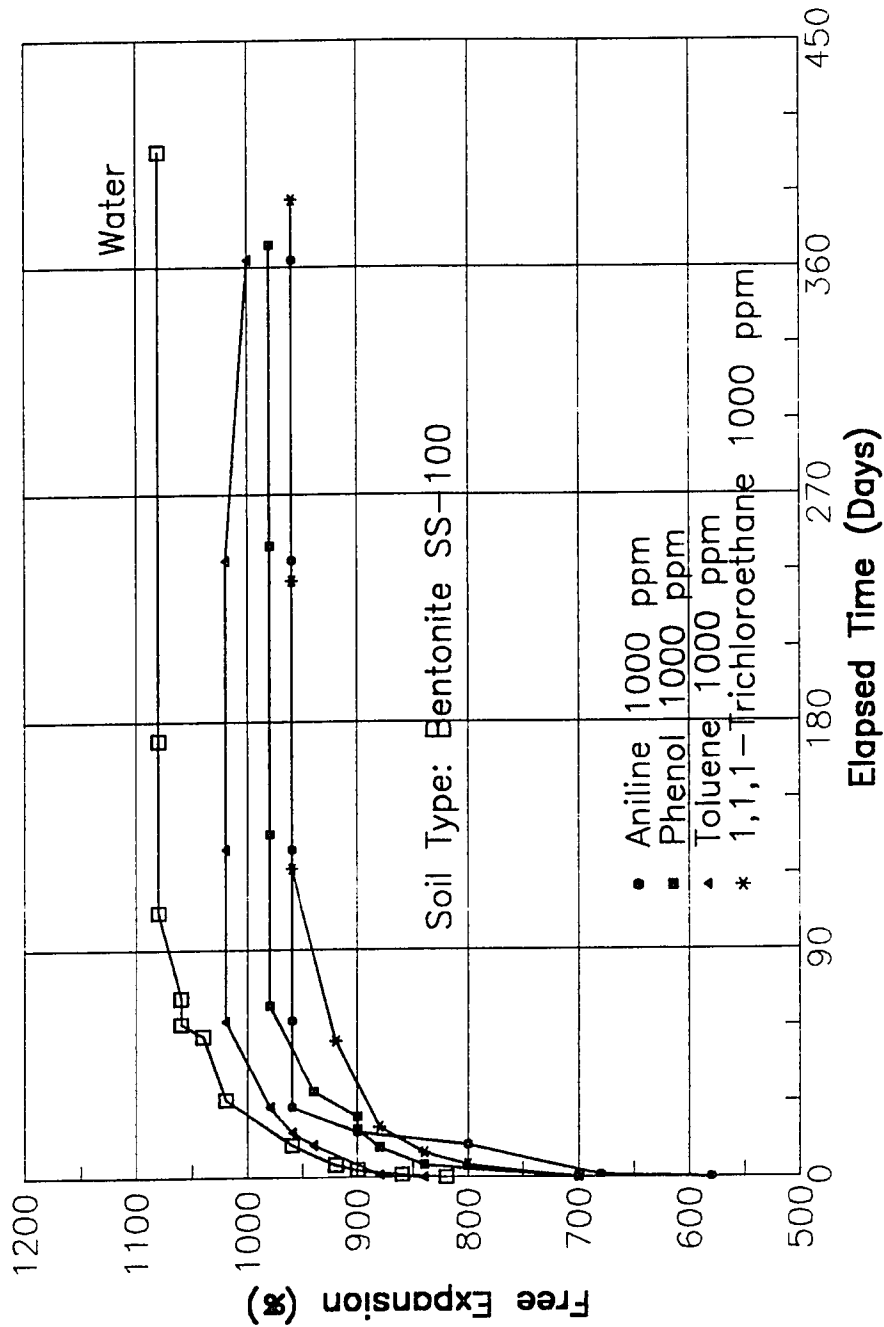


Figure 4-2E Free Expansion of Bentonite SS-100 with Various Organic Chemicals at 1000 ppm.

As shown in Figure 4-2F, the F_e of chromium 50 ppm from potassium chromate (K_2CrO_4) was about 880 percent. It exhibited more effect on SS-100 though its concentration is much lower than that of the organic liquids. The higher charge on the chromium ion is believed to inhibit expansion. The Gouy-Chapman theory describes that the higher the ion valence, the smaller the double layer thickness, thus the lesser the expansion.

The effects of an acid and a base on the expansion are shown in Figures 4-2G and 4-2H, respectively. The hydrochloric acid 0.4 percent showed an F_e of 520 percent. When the concentration increased to two percent, F_e dropped to only 270 percent.

For sodium hydroxide, F_e of 0.4 percent solution showed a value of 830 percent which is about two-thirds more than that of the hydrochloric acid 0.4 percent. In accordance with the Gouy-Chapman theory, decreasing the pH tends to reduce the thickness of the double layer and thus has less expansion. The test result agrees well with this hypothesis. However, when the concentration further increased to 2 percent, the expansion became considerably less than that in the hydrochloric acid solution. When the concentration increased to 4 percent, SS-100 showed only 44 percent of volume expansion. Soils with free expansion values of less than 50 percent are considered to be non-expansive material (Head, 1984). This appears to be an anomaly, as the sodium-rich environment should favor the sodium bentonite expansion.

EXPANSION TEST

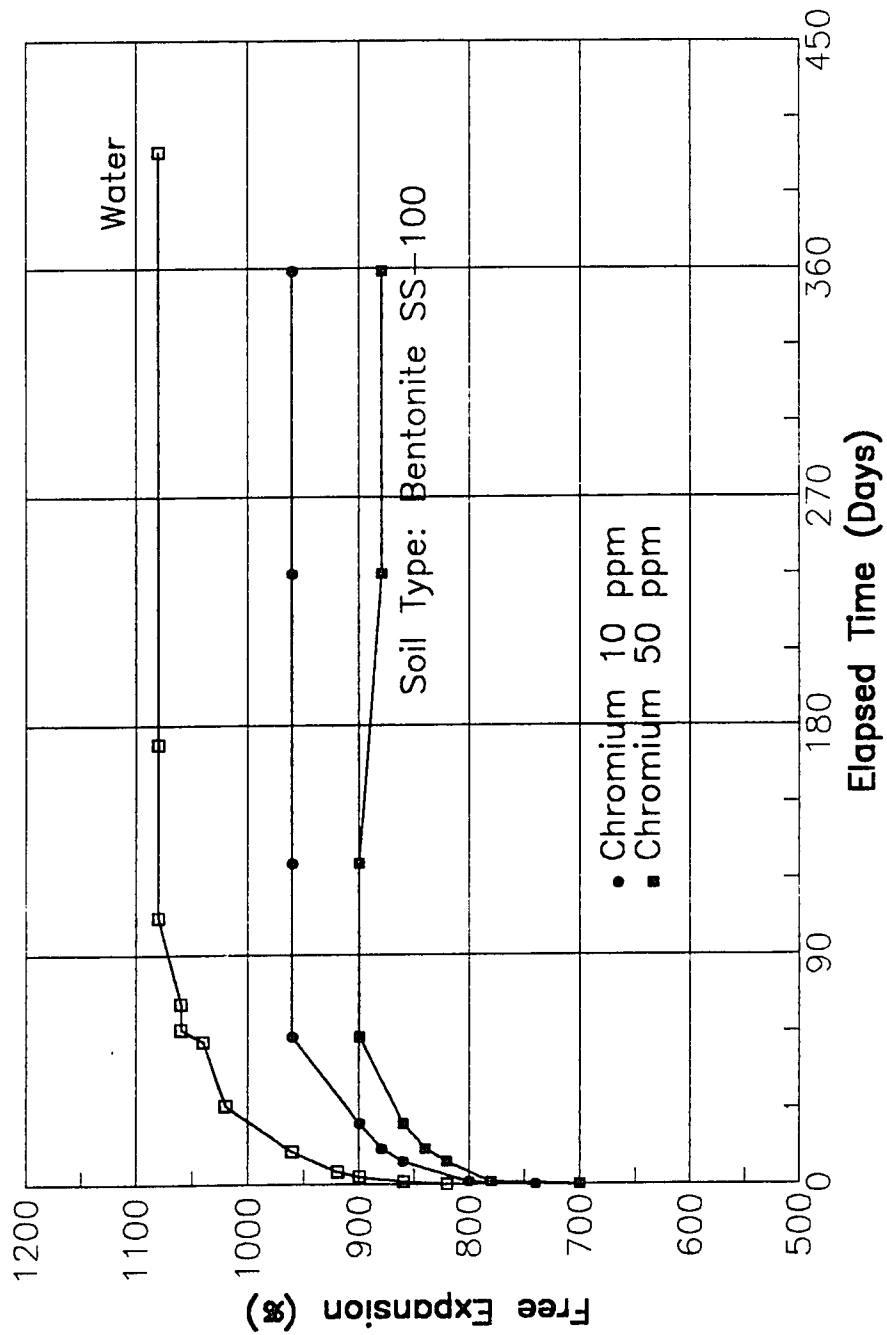


Figure 4-2F Free Expansion of Bentonite SS-100 with Chromium from Potassium Chromate.

EXPANSION TEST

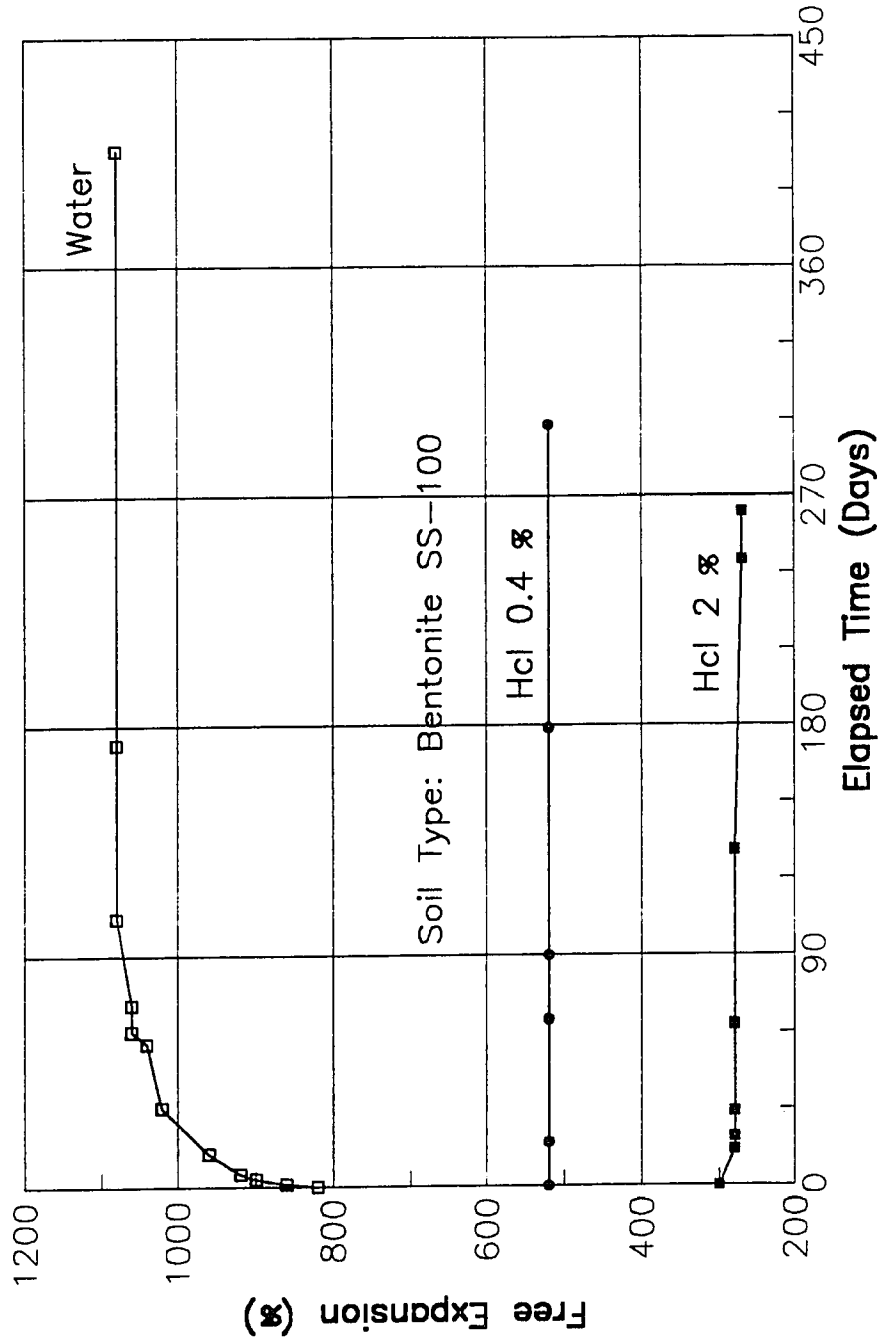


Figure 4-2G Free Expansion of Bentonite SS-100 with Hydrochloric Acid.

EXPANSION TEST

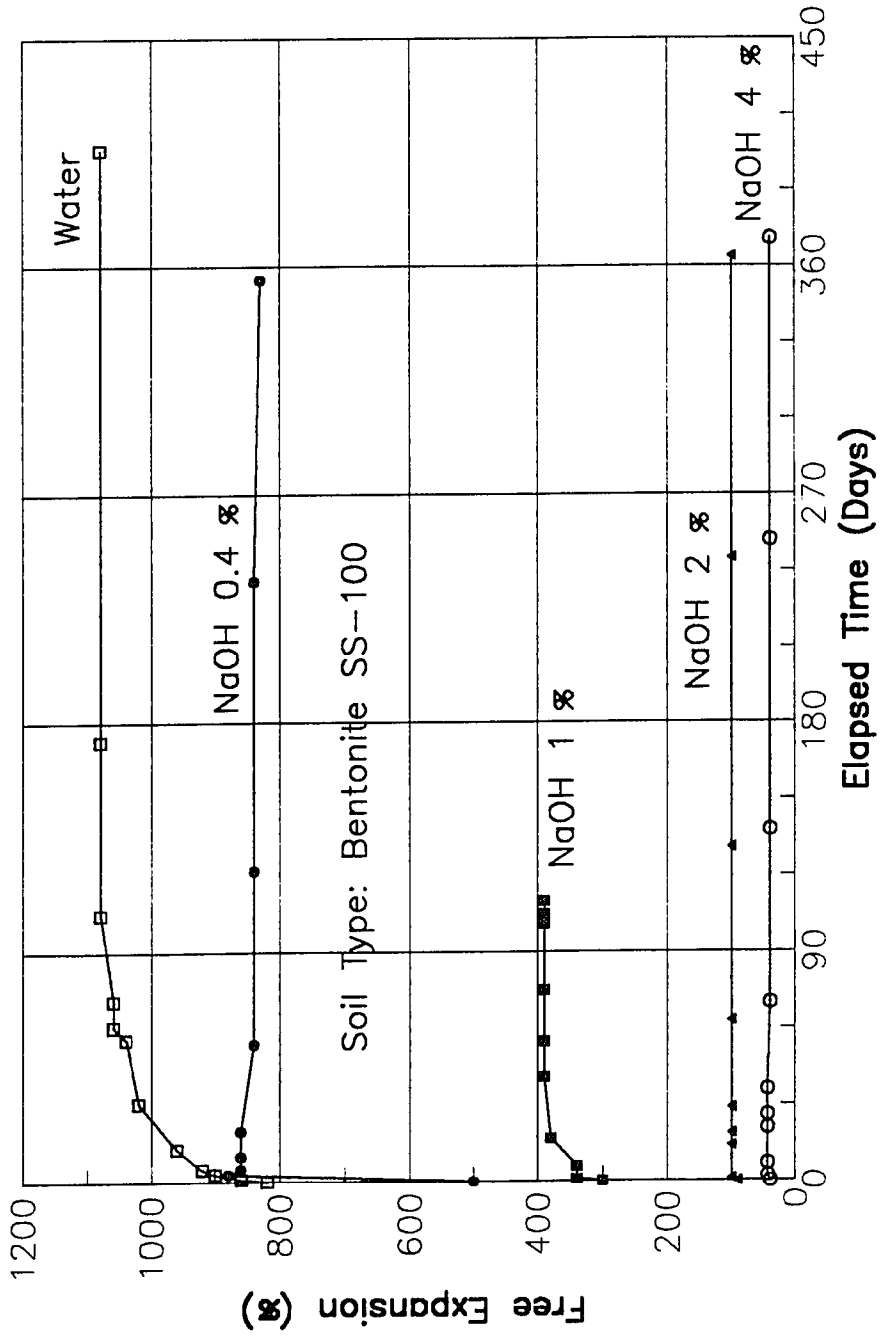


Figure 4-2H Free Expansion of Bentonite SS-100 with Sodium Hydroxide.

Hughes (1977) observed that the expansion of sodium bentonite was severely inhibited due to the presence of other sodium salts. He found that if additional sodium is added to the system a decrease in the water absorption occurs after a certain concentration. The expansion of bentonite is related to the surface charge density and ionic distribution of the sodium. This sodium attracts water molecules and ionically orients them in the swollen system, and thereby causes the expansion. He concluded that this is a result of competition among the excess sodium ions for the same orienting effect on the water molecules. The net effect of this ionic crowding is to reduce the expansion of the bentonite.

Daniel and Liljestr nd (1984) also reported that acid and base of higher concentrations tend to dissolve the clay minerals and cause a reduction of expansion.

As shown in Figure 4-3A, the expansion of bentonite CS-200 was about 15 percent more than the value of SS-100. CS-200 also expanded faster than SS-100. As noted earlier, the dry particle size of CS-200 is smaller than that of SS-100. The smaller the particle, the greater the active surface area with its surrounding environment; therefore, the faster the expansion. Furthermore, SS-100 is chemically treated and formulated with a dispersing agent. It is more dispersive than CS-200. Lambe (1969) reported that a dispersed sediment contains parallel soil particles and tends to have a denser volume than a flocculated sediment. Though the bentonite was not sedimenting in the liquid during the expansion test, the

EXPANSION TEST

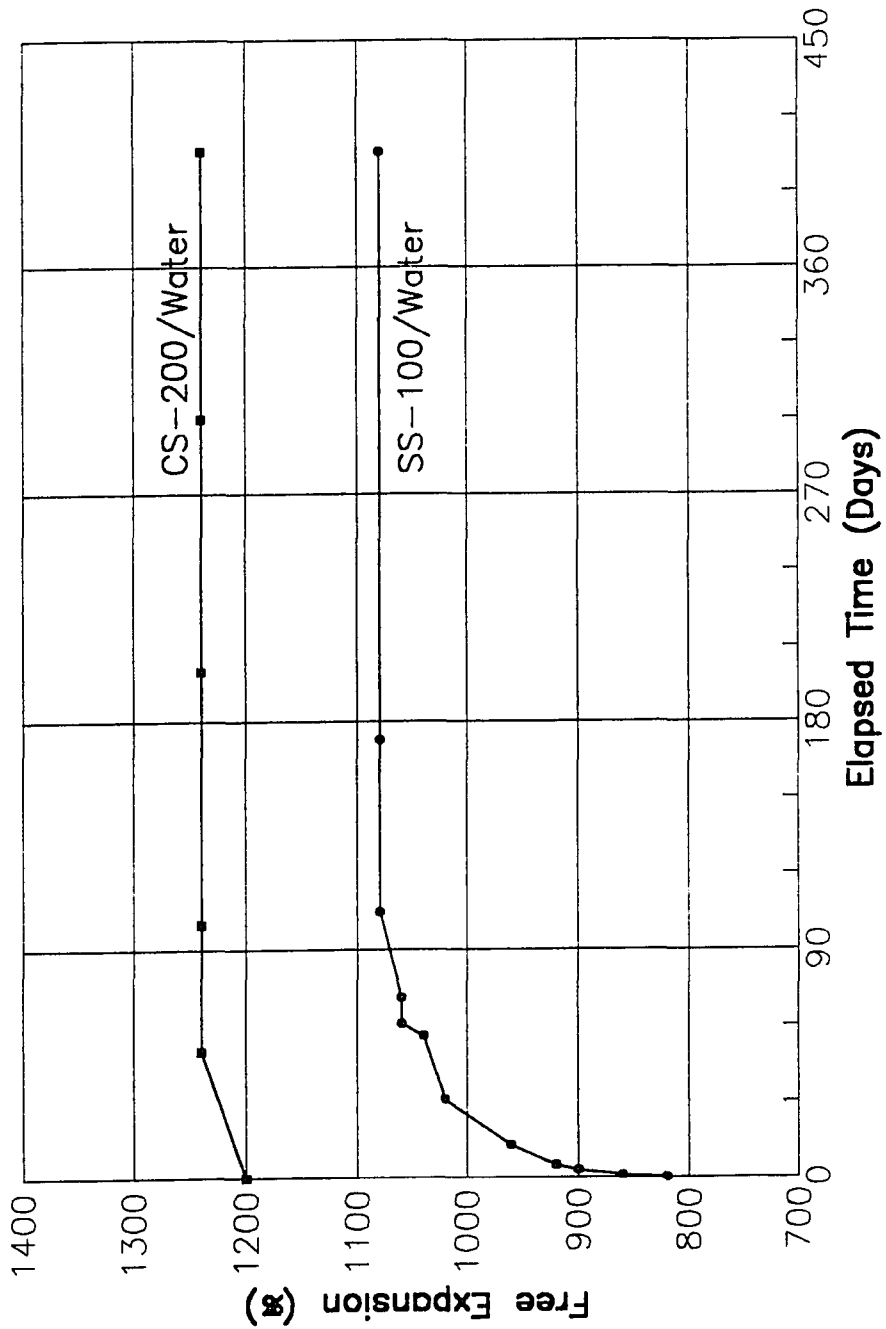


Figure 4-3A Comparison of Expansion Behavior for Bentonite SS-100 and CS-200 with Water.

treatment of SS-100 with a dispersing agent could decrease its expansion as compared to ordinary bentonite (Olston, 1988).

As shown in Figure 4-3B, F_e values of diluted phenols decreased with the increasing concentration; however, their differences from that of water are insignificant. Figure 4-3B also depicts that, at a concentration of 10,000 ppm, aniline showed an F_e of 950 percent which was about 25 percent less than that of water.

In Figure 4-3C, expansion with diluted toluene varied little from that with water. After 250 days, the expansion for all the three concentrations dropped. The reduction may have resulted from temperature variation. Since no further observation was made, this effect is inconclusive.

Note that the expansion of CS-200 in these organic liquids showed relatively higher values than that of SS-100. Such observation appears to be contradictory to the expected behavior as SS-100 is "contaminant resistant" and should show higher expansion than the ordinary bentonite.

Figure 4-3C also shows that the landfill leachate affected CS-200 considerably. The expansion of CS-200 in this liquid was about 63 percent less than in water. When compared with SS-100, it showed about 28 percent less expansion. Considering that in all other chemicals CS-200 expanded more than SS-100, it appears that SS-100 may present certain degradation resistances in a composite contaminant environment as claimed by the manufacturer. However, since

EXPANSION TEST

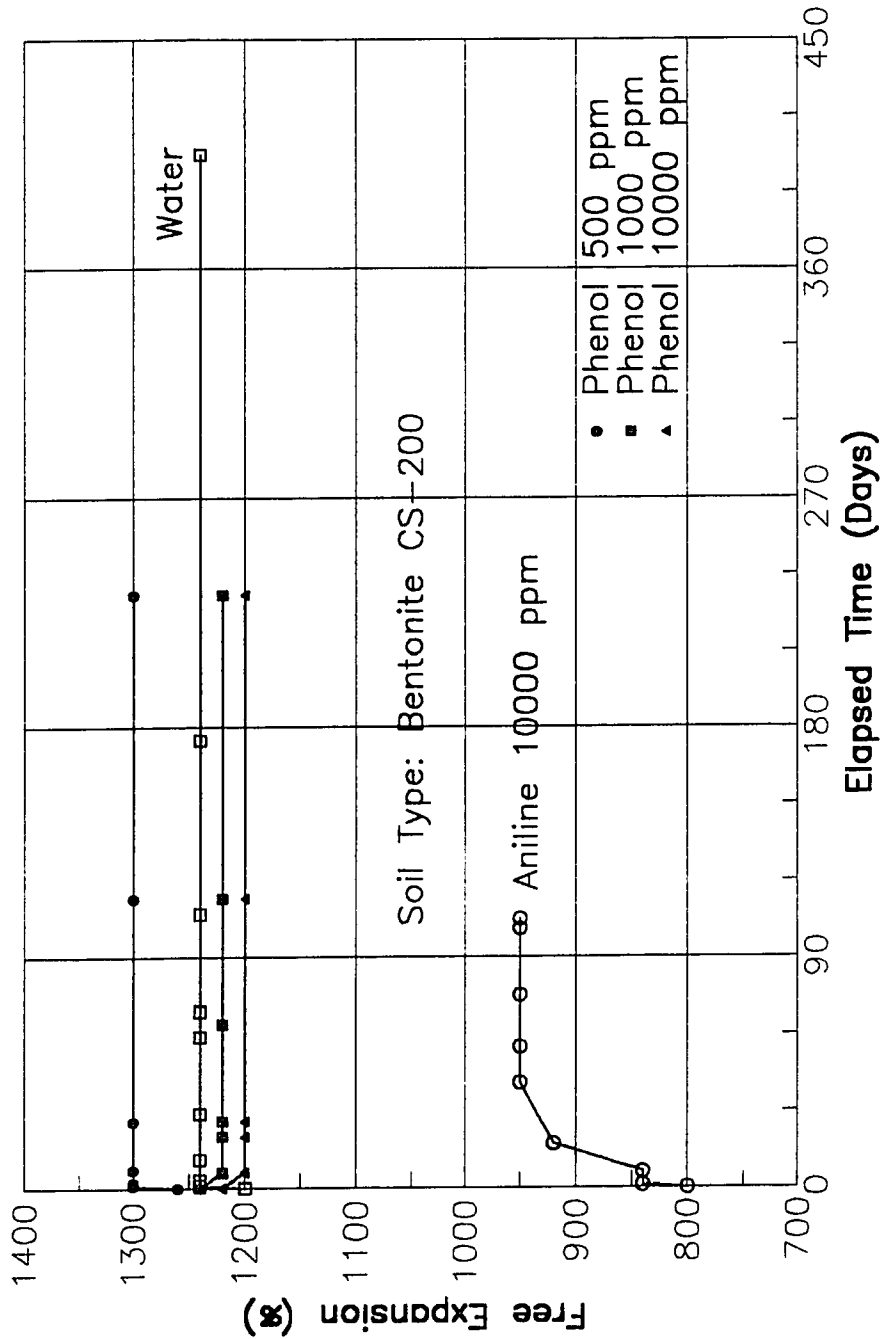


Figure 4-3B Free Expansion of Bentonite CS-200 with Phenol and Aniline.

EXPANSION TEST

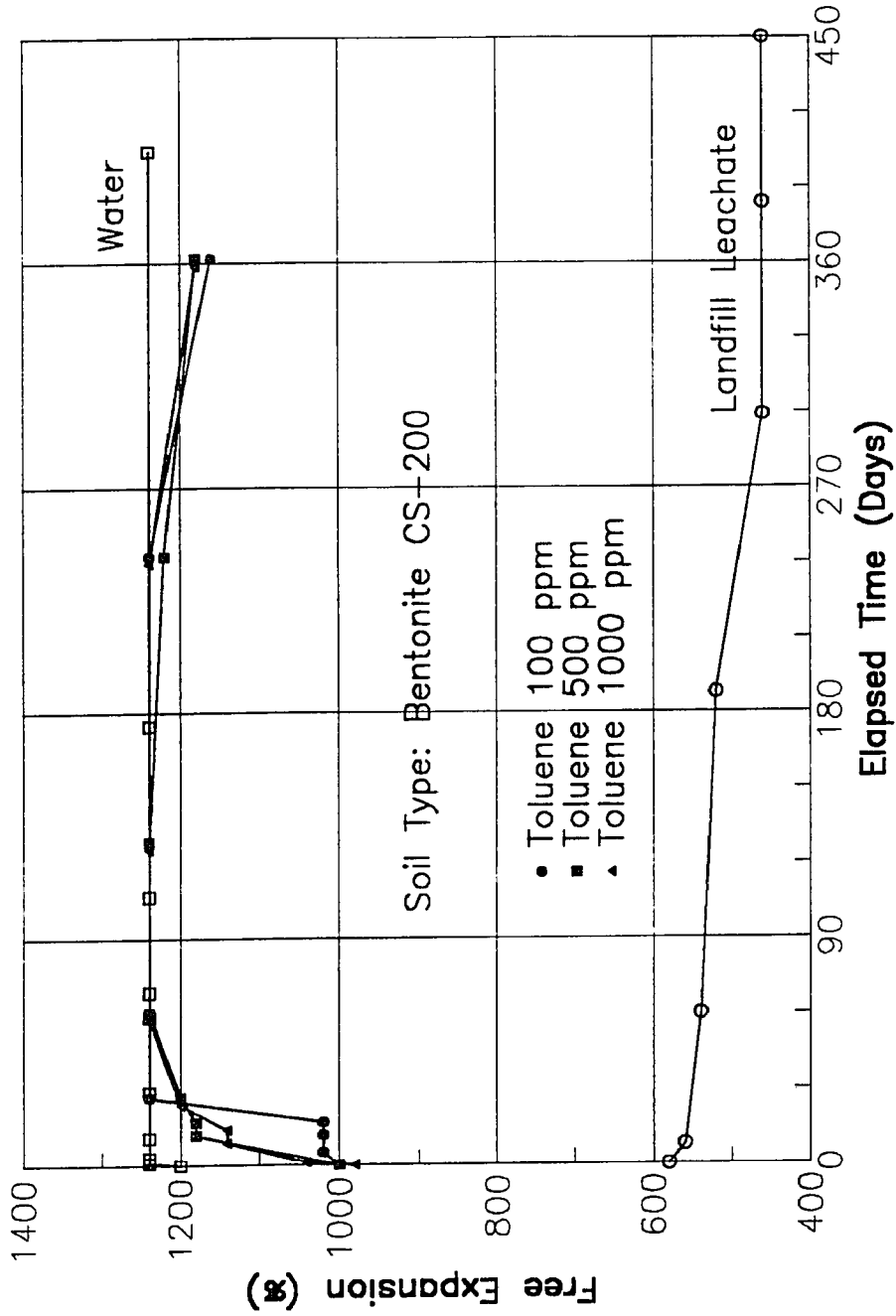


Figure 4-3C Free Expansion of Bentonite CS-200 with Toluene and Landfill Leachate.

this is true only for a single test, more experiments should be conducted to draw a conclusion.

At a concentration of 10 ppm, CS-200 with chromium expanded 5 percent more than with water (Figure 4-3D). As the concentration increased to 50 ppm, expansion dropped to a value about 11 percent less than with water.

The expansion behaviors of CS-200 with sodium hydroxide and hydrochloric acid are presented in Figures 4-3E and 3F, respectively. The variations of their expansions are close to those found for SS-100.

The test results for CS-200 generally exhibited a similar tendency to those with SS-100; therefore, the reduction of expansion for CS-200 could be attributed to the same effect as described for SS-100.

4.1.2.2 Specific Expansions Figure 4-4A shows a comparison of specific expansion between the two bentonites. The final S_e of SS-100 was about 90 percent of that of CS-200.

The specific expansion of SS-100 with all the liquids of highest concentrations is presented in Figure 4-4B. All the organic liquids and chromium were found to have a value greater than 0.8, except aniline 10,000 ppm and the leachate which showed a value of about 0.6. Sodium hydroxide and hydrochloric acid showed considerably lower values of S_e . At a concentration of 4 percent, S_e for the sodium hydroxide was close to zero.

Similar presentation for CS-200 are shown in Figure 4-4C. The specific expansion of CS-200 with aniline 10,000 ppm

EXPANSION TEST

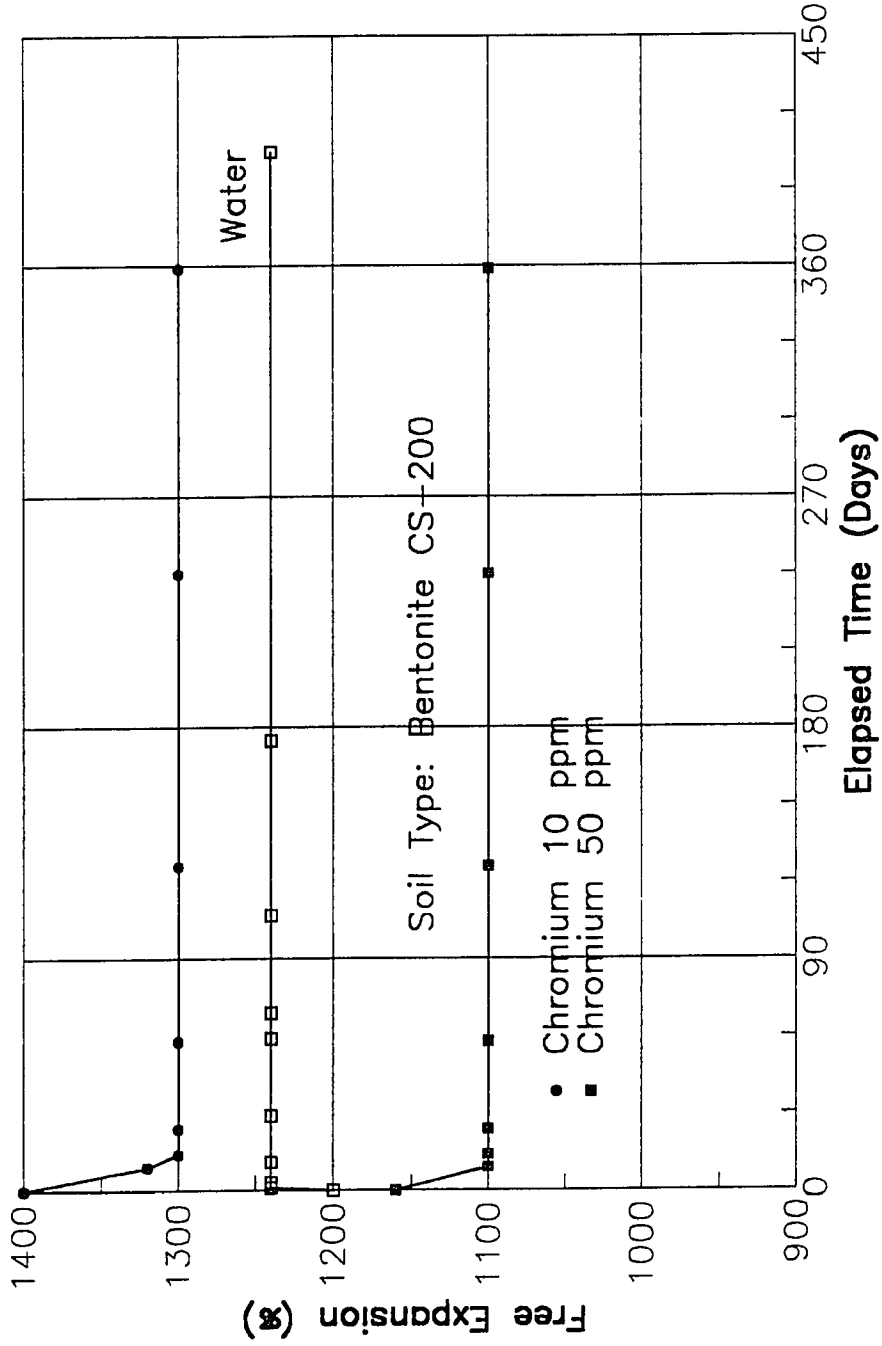


Figure 4-3D Free Expansion of Bentonite CS-200 with Chromium from Potassium Chromate.

EXPANSION TEST

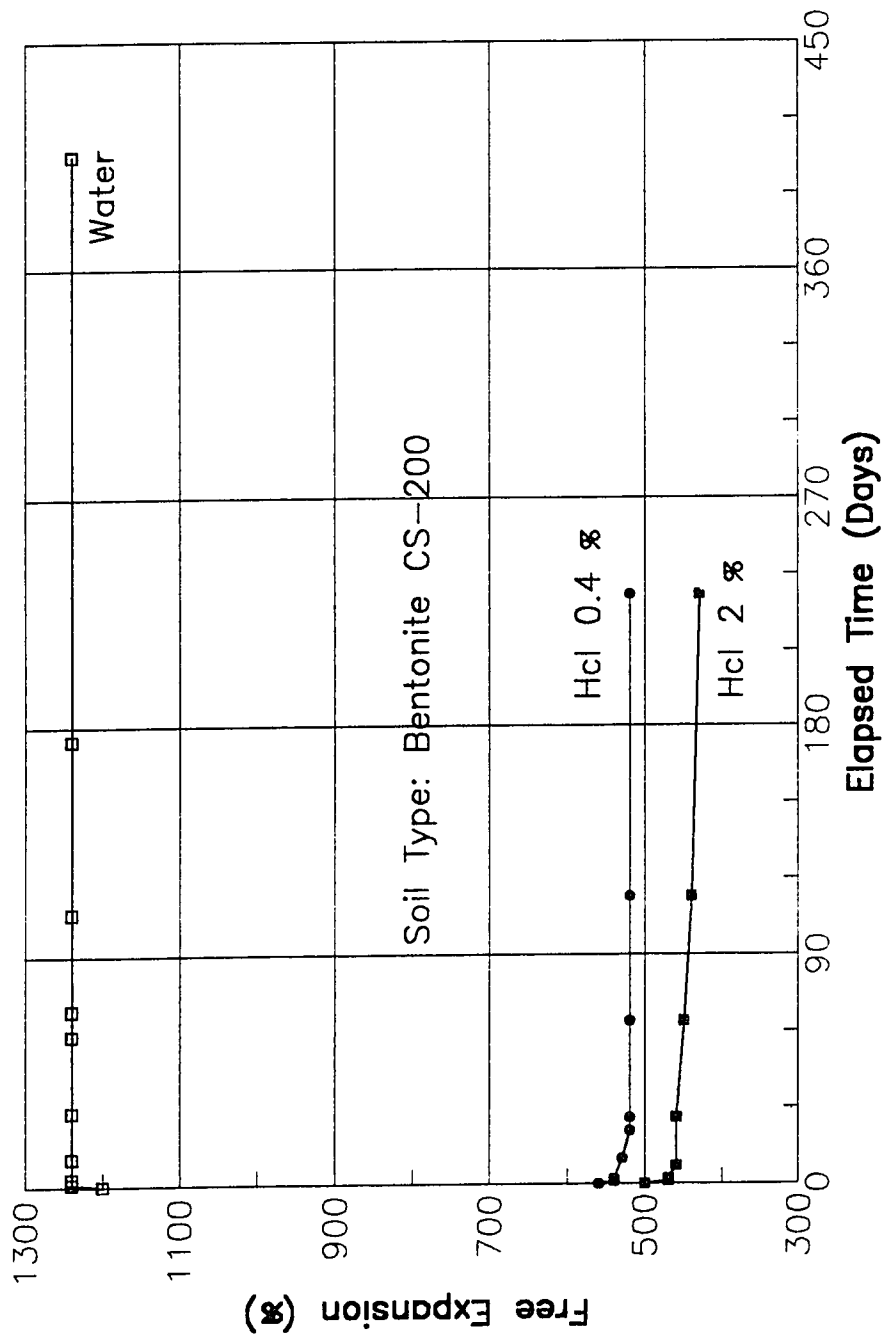


Figure 4-3E Free Expansion of Bentonite CS-200 with Hydrochloric Acid.

EXPANSION TEST

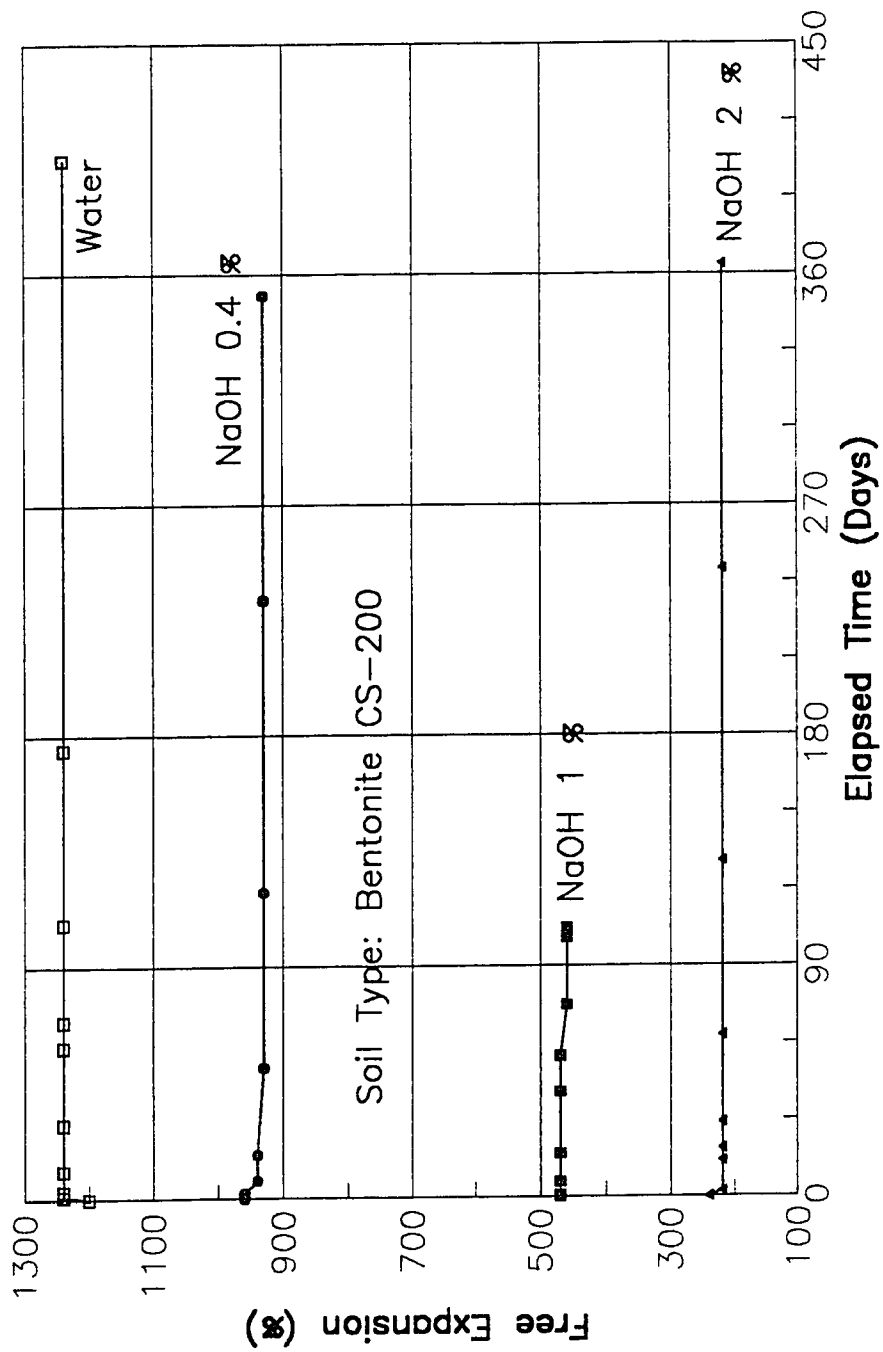


Figure 4-3F Free Expansion of Bentonite CS-200 with Sodium Hydroxide.

EXPANSION TEST

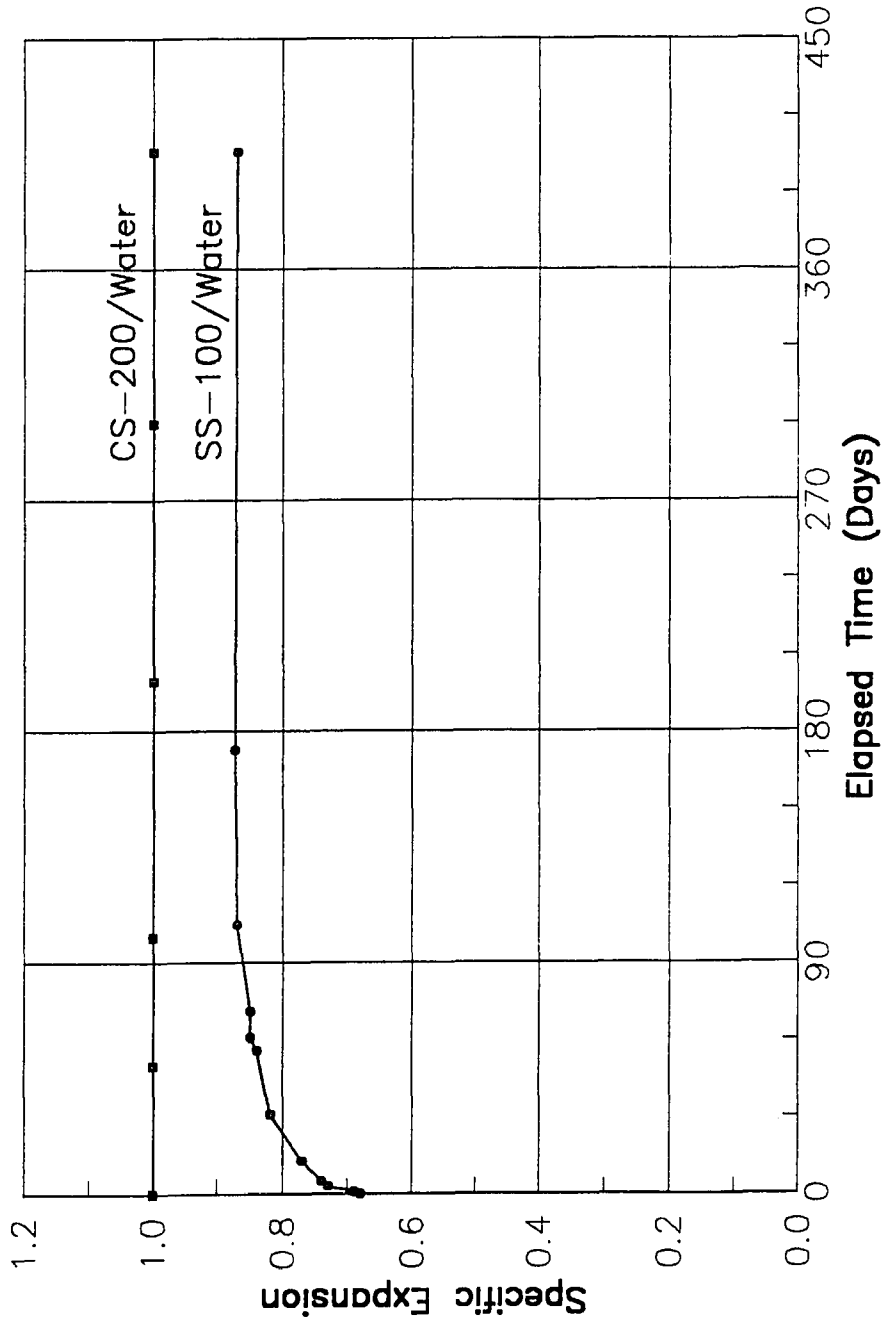


Figure 4-4A Relative Expansion Behavior of Bentonite SS-100 and CS-200 with Water.

EXPANSION TEST

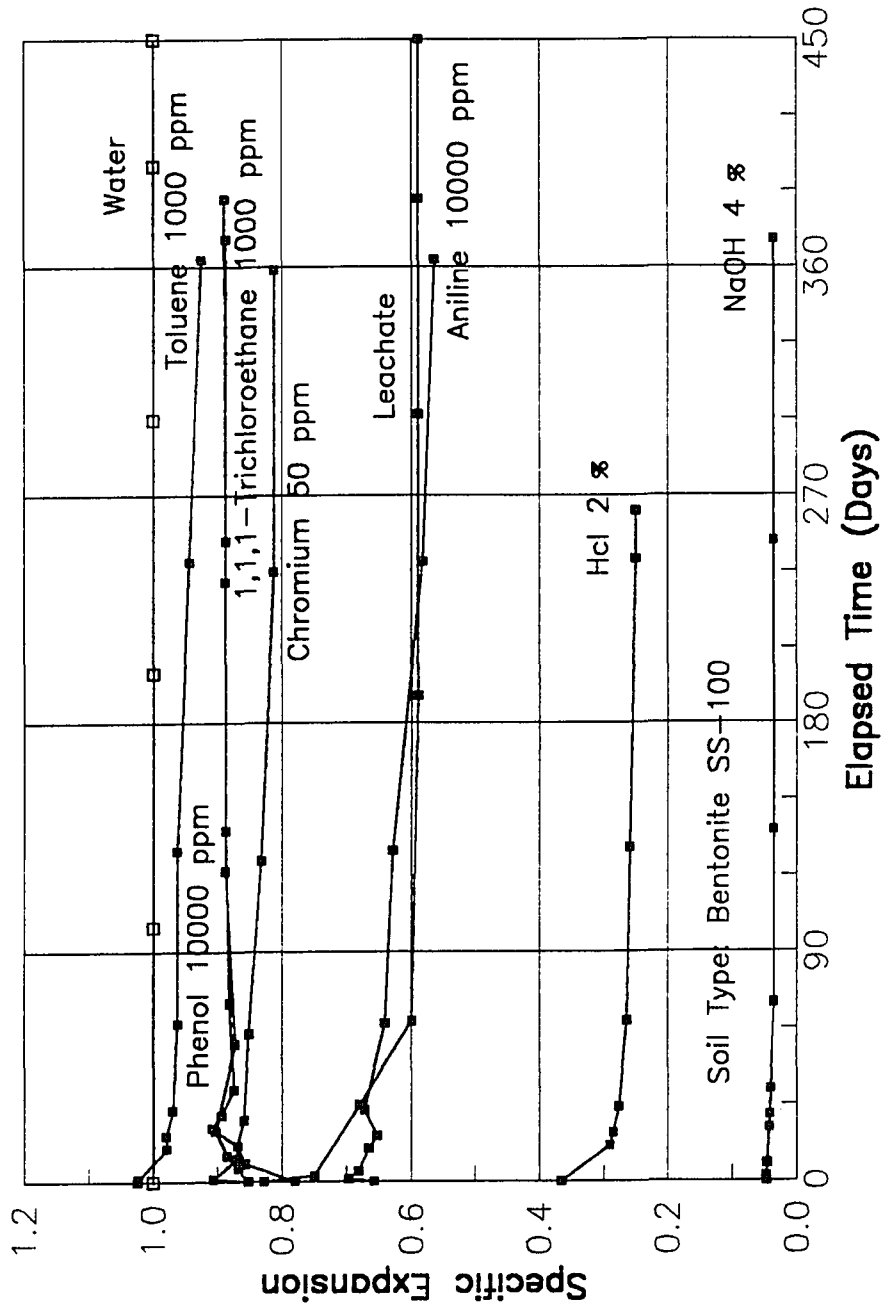


Figure 4-4B Specific Expansion of Bentonite SS-100 with Various Chemicals.

EXPANSION TEST

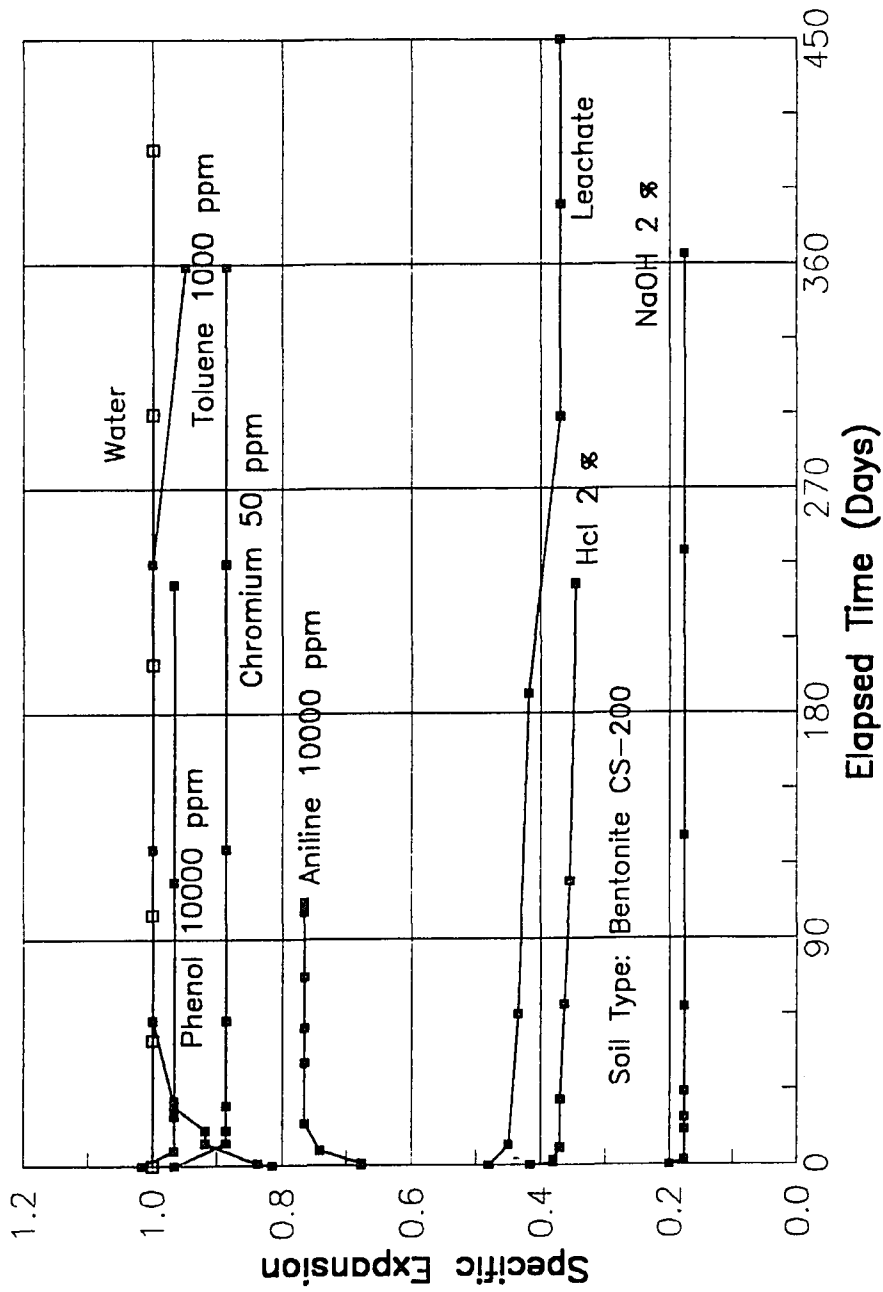


Figure 4-4C Specific Expansion of Bentonite CS-200 with Various Chemicals.

was close to 0.8. For all other organic liquids and chromium, S_e was about 0.9 or greater. The specific expansion for the landfill leachate was only about 0.4. With sodium hydroxide and hydrochloric acid, at higher concentrations, values of S_e for CS-200 were greater than those for SS-100. This indicates that SS-100 provided no special resistance against degradation for acid and base.

Specific expansion appears to be able to provide an indication about the expansion behavior of bentonite in a liquid in relation to that in water. Based on these values, one can easily identify the effect of a chemical on bentonite.

4.1.2.3 Exposure Time The expansion of both bentonites generally stabilized after a certain time. Table 4-2 presents the equilibrium time for all the liquids tested. In some cases, equilibrium time tends to have more than one value. In such cases, the second value is reported in parentheses. The change in the equilibrium time may be due to the effect of temperature variation or biodegradation. It appears that the equilibrium time tends to vary with the type of chemical and its concentration. Liquids that showed a more pronounced effect on the expansion of SS-100 generally reached equilibrium in a shorter time. For example, aniline solutions reached equilibrium at about 30 days whereas phenol solutions took about 135 days. It appears that for organic liquids, equilibrium time showed negligible difference among the concentrations tested.

Table 4-2 Equilibrium Time for Expansion

Type of liquid	Concentration (ppm)	¹ Equilibrium Time (Days)	
		SS-100	CS-200
Water	-	108	1
Aniline	500	31	-
	1000	28	-
	10000	28 (>360)	41
Phenol	500	137	1
	1000	137	1
	10000	136	1
Toluene	100	61 (>400)	5
	500	57	12
	1000	62 (>363)	1
1,1,1-Trichloroethane	100	122	-
	1000	122	-
Chromium	10	58	9
	50	58 (>240)	9
Sodium Hydroxide	0.4 %	54	51
	1.0 %	40	1 (75)
	2.0 %	1	1
	4.0 %	1	-
Hydrochloric Acid	0.4 %	1	21
	2.0 %	2	>232
Landfill Leachate	-	63	300

1) Equilibrium time is the time at which the expansion appears to be stable. In some liquids, equilibrium time tends to have more than one value. In such cases, the second value is reported in parentheses.

For sodium hydroxide, the volume change of 0.4 percent solution needed 54 days to stabilize. When the concentration increased to 1 percent, the equilibrium time reduced to 40 days. As the concentration further increased to 2 percent, the volume change reached to constant instantaneously. The higher the concentration, the shorter the time to

equilibrium.

For hydrochloric acid, the equilibrium occurred rapidly, and showed little difference between 0.4 percent and 2 percent solutions. As noted earlier, at lower concentration, hydrochloric acid showed more effect on the bentonite expansion than sodium hydroxide. When the concentration increased the tendency reversed. The observation of equilibrium time agrees fairly well with this statement.

Equilibrium time for CS-200 in organic liquids and chromium was generally much less than for SS-100. This tendency can be attributed to the effect of particle size. The smaller particle generally reacts faster with the liquid. Therefore, CS-200 showed a shorter equilibrium time.

When tested with hydrochloric acid and leachate, equilibrium time was found to be much longer than that for SS-100. Bentonite expansions in these liquids were considerably less than those in the organic liquids. It appears that the effect of particle size on the equilibrium time became insignificant.

As noted earlier, CS-200 in hydrochloric acid exhibited greater resistance than SS-100; therefore it may take longer to reach equilibrium. In the leachate, CS-200 showed less resistance than SS-100. Equilibrium time should be shorter than for SS-100. However, this is not the case. It appears to have certain time-dependent soil-chemical interactions occurred and caused the soil slowly degraded. Since no detailed chemical analyses were performed for the specimen

after the test, any explanation for the anomaly is inconclusive.

For sodium hydroxide that showed the strongest influence on the expansion, the time was about the same for both types of bentonites.

Equilibrium time generally correlates well with expansion behavior for the liquids tested. It could serve as a guide to select a proper testing period for obtaining more reliable values for permeability determinations.

4.1.3 Sedimentation Tests

Modified sedimentation tests were conducted on bentonite SS-100 with selected liquids. The test results for a hydrometer (sedimentation) analysis are normally presented as the percentage passing versus the grain size diameter. The diameter of soil particle is calculated according to Stokes' Law (ASTM D-422, 1988). The result is highly dependent upon temperature, viscosity, and the specific gravity of the suspending medium. ASTM procedure includes standard graphs and tables to facilitate calculations. However, these computational aids are for water. Their use for liquids other than water may lead to erroneous results. The calculations of the percentage of soil in suspension at any given time are rather simple and involve less uncertainty. Therefore, the plots of percent finer by weight as related to the elapsed time are used for presentation in Figures 4-5A through 4-5F. The linear representation of the data points was based on regression analysis.

SEDIMENTATION TEST

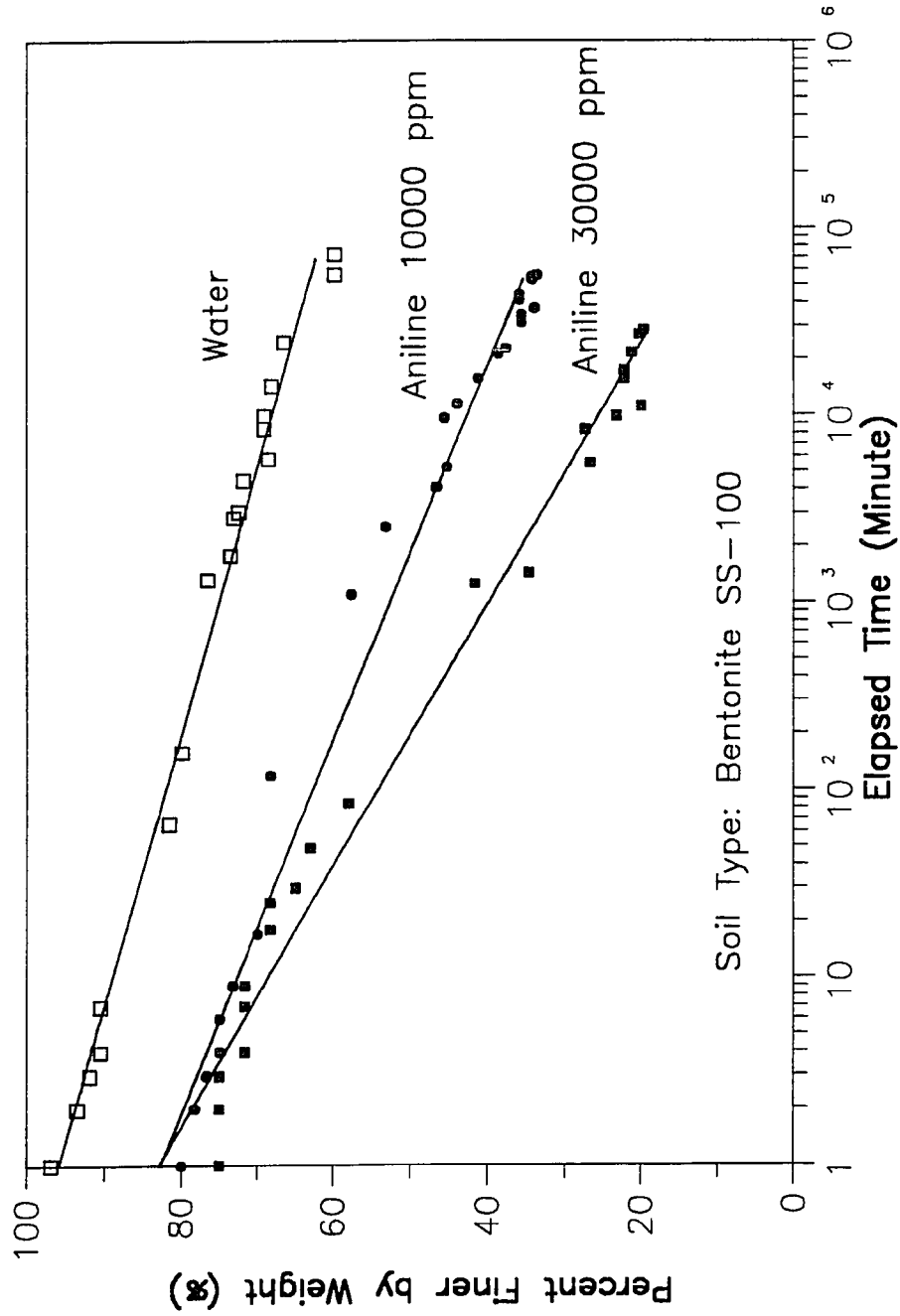
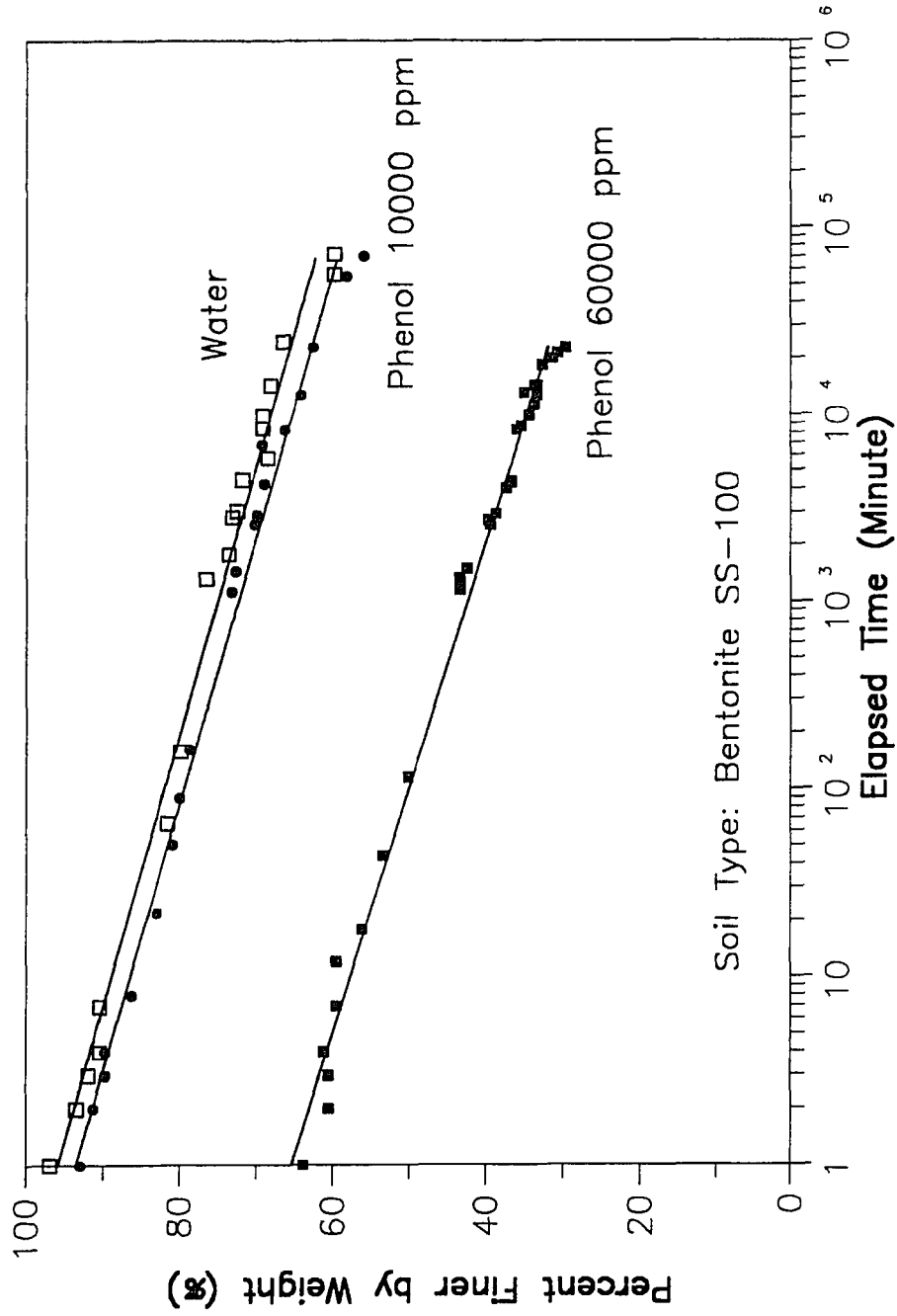


Figure 4-5A Sedimentation Behavior of Bentonite SS-100 with Aniline.

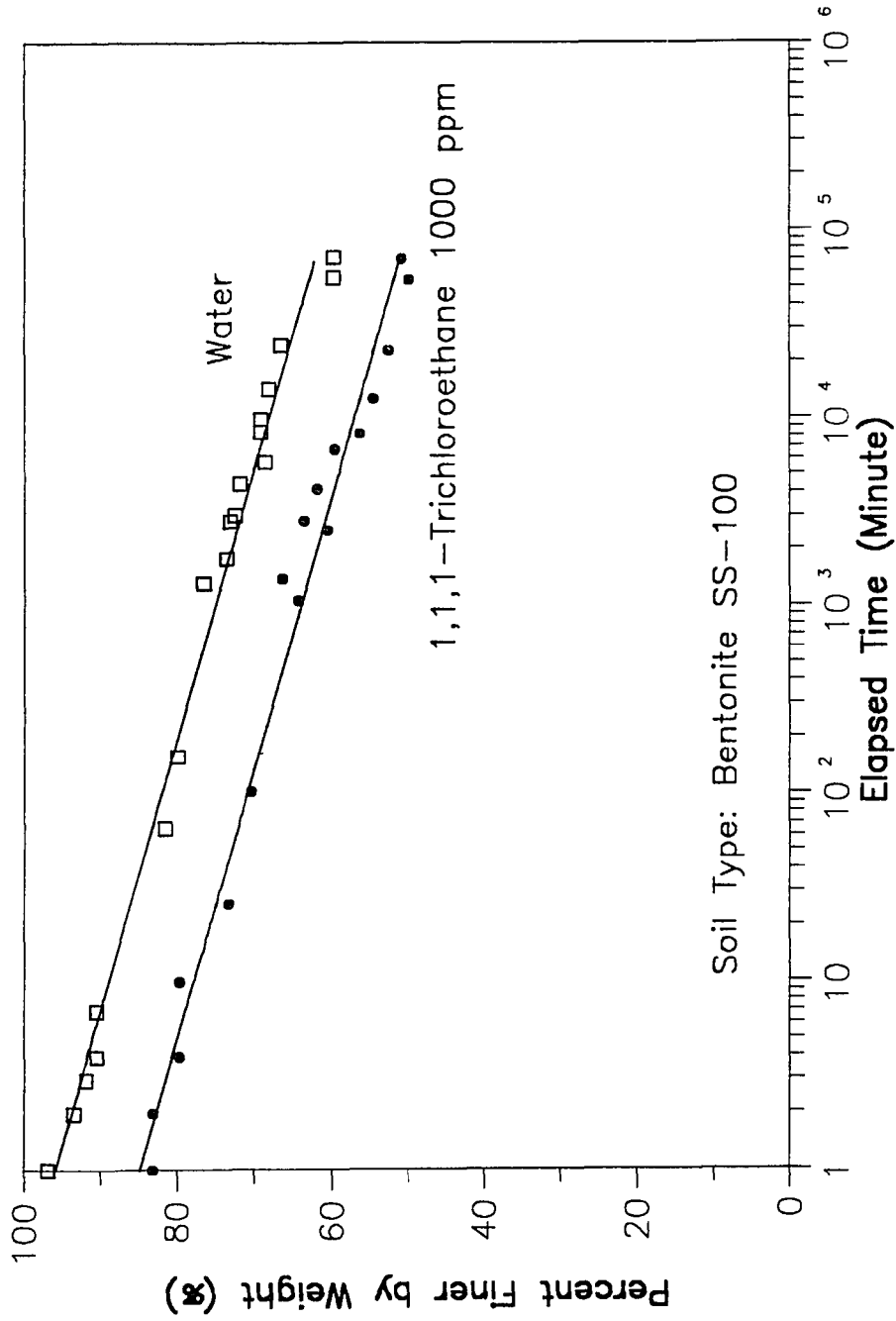
SEDIMENTATION TEST



Soil Type: Bentonite SS-100

Figure 4-5B Sedimentation Behavior of Bentonite SS-100 with Phenol.

SEDIMENTATION TEST



Soil Type: Bentonite SS-100

Figure 4-5C Sedimentation Behavior of Bentonite SS-100 with 1,1,1-Trichloroethane.

SEDIMENTATION TEST

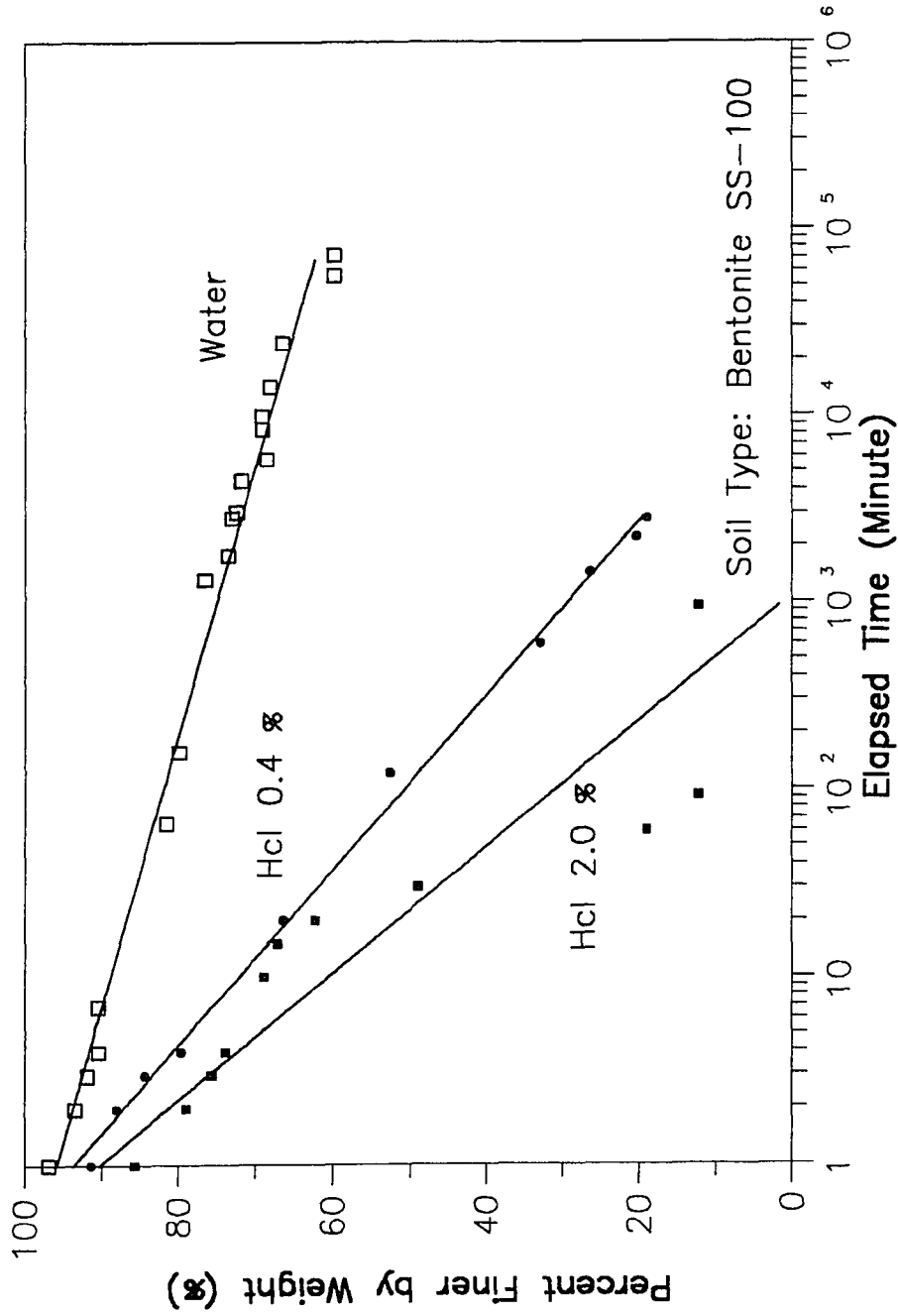


Figure 4-5D Sedimentation Behavior of Bentonite SS-100 with Hydrochloric Acid.

SEDIMENTATION TEST

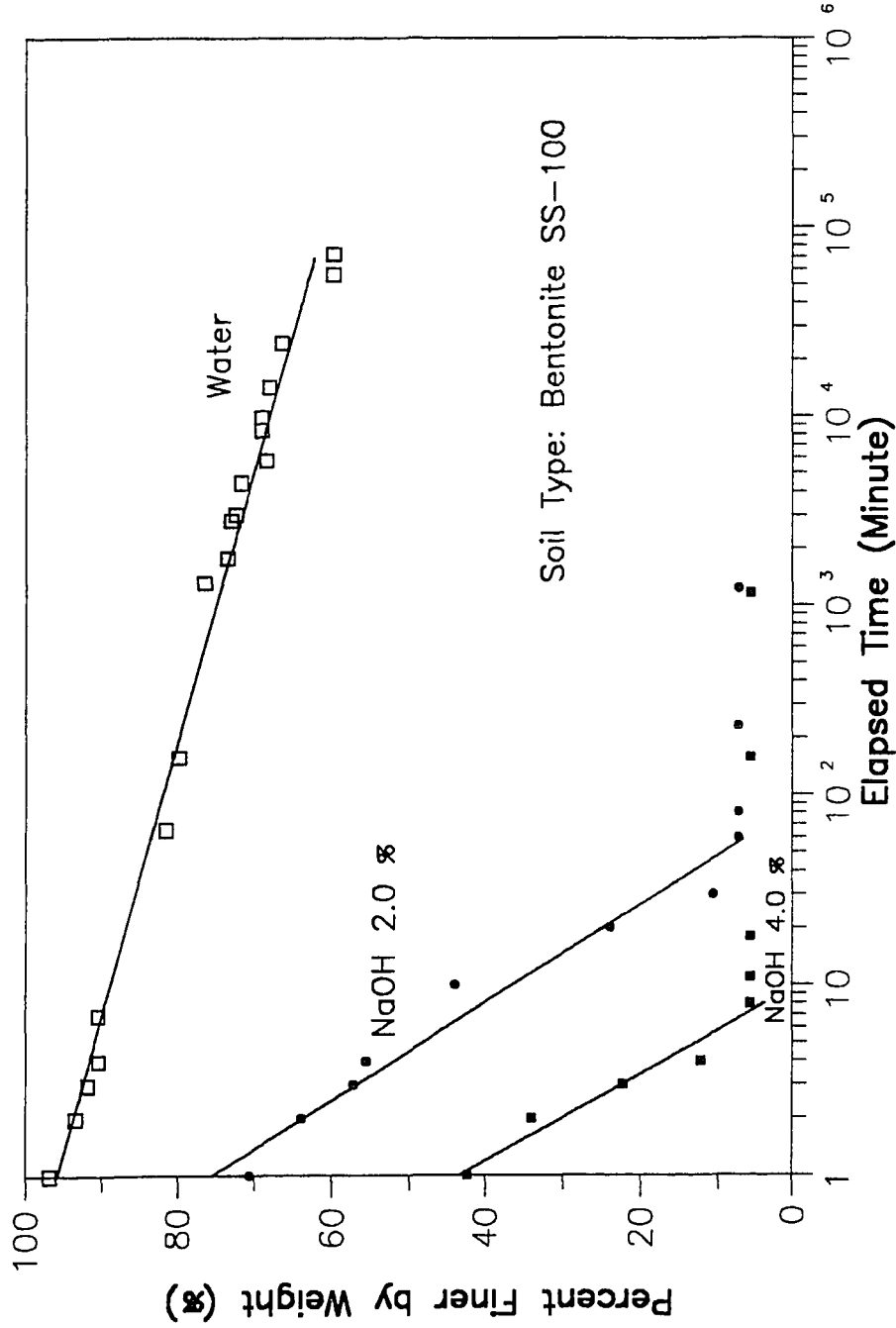


Figure 4-5E Sedimentation Behavior of Bentonite SS-100 with Sodium Hydroxide.

SEDIMENTATION TEST

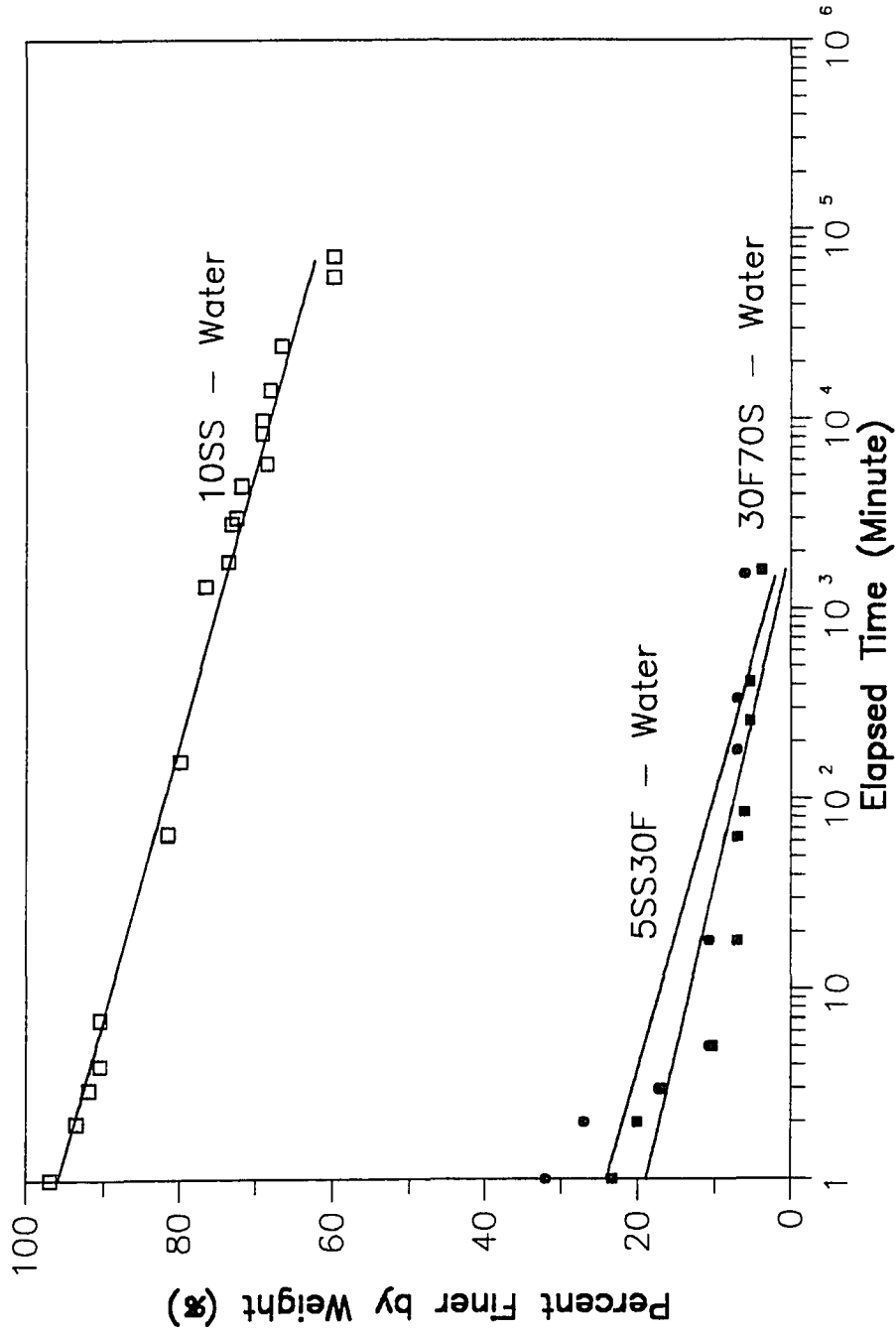


Figure 4-5F Sedimentation Behavior of Various Soil Mixtures with Water.

To describe the effect of chemicals on the bentonite quantitatively, two dimensionless terms are proposed. The first term is the slope of the regression line which is defined as the sedimentation index, S_i , and used to indicate the rate of sedimentation of the soil particles. The second term, initial sedimentation I_s , is defined as the intercept at an elapsed time of one minute along the ordinate. It can be used to express the immediate sedimentation behavior of a soil for a given liquid. Based on the sedimentation principles, lower I_s and higher S_i indicate a faster sedimentation condition. This in turn indicates that a shorter time may be required to complete the soil-chemical interaction. Table 4-3 presents the values of I_s and S_i for various tests.

It can be seen that I_s and S_i tend to be altered by chemicals. As the concentrations increase, the S_i values increase and the I_s values decrease.

For SS-100, lower S_i generally corresponds with higher I_s and vice versa. When tested with water, 5SS30F and 30F, containing a greater amount of coarse particles, have smaller I_s values. These values are less than one-fourth of that for SS-100. The less the bentonite content, the smaller the I_s . However, their S_i values are similar to SS-100. I_s appears to be a variable related to soil properties. In addition, a comparison of S_i for SS-100 with water and low concentrated liquids such as 1,1,1-trichloroethane 1000 ppm shows that their S_i values are close to each other. S_i appears to show

little variation provided that the properties of the liquids are similar. Based on this observation, I_s appears to be dependent upon the soil condition such as particle size, whereas S_i is strongly influenced by the characteristics of the liquid.

Table 4-3 Summary of Sedimentation Parameters

Type of Soil	Type of Liquid	Concentration (ppm)	I_s ¹	S_i ¹
SS-100	Water	-	95.9	7.0
	Aniline	10000	82.7	10.5
		30000	82.9	14.5
		60000	79.0	15.5
	Phenol	10000	93.6	7.5
		60000	65.4	8.0
	1,1,1-Trichloroethane	1000	84.8	7.0
	Sodium Hydroxide	2.0 %	75.5	40.0
		4.0 %	43.3	45.0
	Hydrochloric Acid	0.4 %	93.7	22.0
2.0 %		90.4	30.5	
5SS30F	Water	-	24.1	7.5
30F	Water	-	18.9	6.0

1) I_s - initial sedimentation and S_i - sedimentation index.

The sedimentation behavior of a soil in a liquid is strongly influenced by the physical properties of the liquid and the physico-chemical interaction of soil-pore fluid electrolyte. The physical properties of the liquid include viscosity and density. The physico-chemical interactions are related to dielectric constant, electrolyte concentration,

and pH.

Bowders (1985) conducted sedimentation tests with several types of clays and organic liquids. The results far exceeded the predicted values even after the density and viscosity were considered. He concluded that the physico-chemical interactions had a greater effect on sedimentation behavior.

According to Equation 2.4, the thickness of the diffuse double layer is directly related to the physico-chemical nature of the liquid. As the thickness of the double layer reduces, soil tends to become flocculated. A flocculated structure is known to have much higher settling velocities (Lambe, 1969).

With aniline the increase in concentration appears to have a greater influence on the value of S_i . The soil-chemical interaction occurred fairly rapidly. For phenol solutions, both I_s and S_i were altered as the concentration changed. Higher S_i and lower I_s values imply that the soil-chemical interaction requires shorter time to complete. In a comparison of the test result for these two chemicals at a concentration of 10,000 ppm, aniline appears to have more influence on bentonite.

The dielectric constant decreases and the conductivity increases with the increase of concentration. The higher the conductivity, the higher the electrolyte concentration. The lower the dielectric constant, or the higher the electrolyte concentration, or both, the smaller the double layer

thickness. It is, therefore, reasonable that a soil in an organic solution of lower dielectric constant and higher concentration would settle more rapidly.

For sodium hydroxide and hydrochloric acid solutions, the electrolyte concentration and the pH would be the two most important factors controlling the sedimentation behavior. According to the Gouy-Chapman theory, the thickness of the double layer decreases with decreasing pH and increasing electrolyte concentration. A comparison of both these solutions at a concentration of 2 percent shows that in sodium hydroxide the soil settled faster than in hydrochloric acid. This tendency was also found in the expansion test. As previously noted, the effect of ionic crowding in a sodium-rich environment could drastically alter the behavior of sodium bentonite. Therefore, it is reasonable to conclude that ionic crowding controlled the sedimentation behavior of SS-100 in a sodium hydroxide solution.

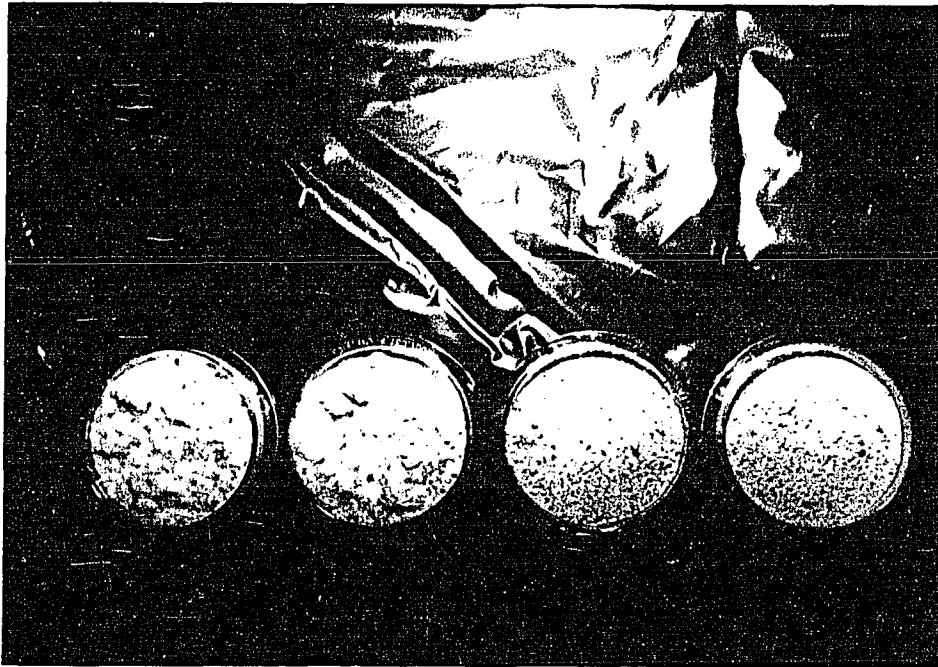
The performance of the sedimentation tests requires vigorous agitation. A significant amount of volatile material may be lost in the process. EPA (1979) reported that the experimental half-life for volatilization of one ppm 1,1,1-trichloroethane to be on the order of 20 minutes when stirred at 200 rpm in water at approximately 25°C in an open container. It is difficult to determine the amount of the residual for a volatile material during the sedimentation test. This is because the tests were performed to provide macroscopic observations of the effects of various liquids on

the soils. No effort was made to examine the microscopic variation of the physico-chemical nature of the soils. The test results with volatile materials, therefore, may be misleading if based on the initial concentration.

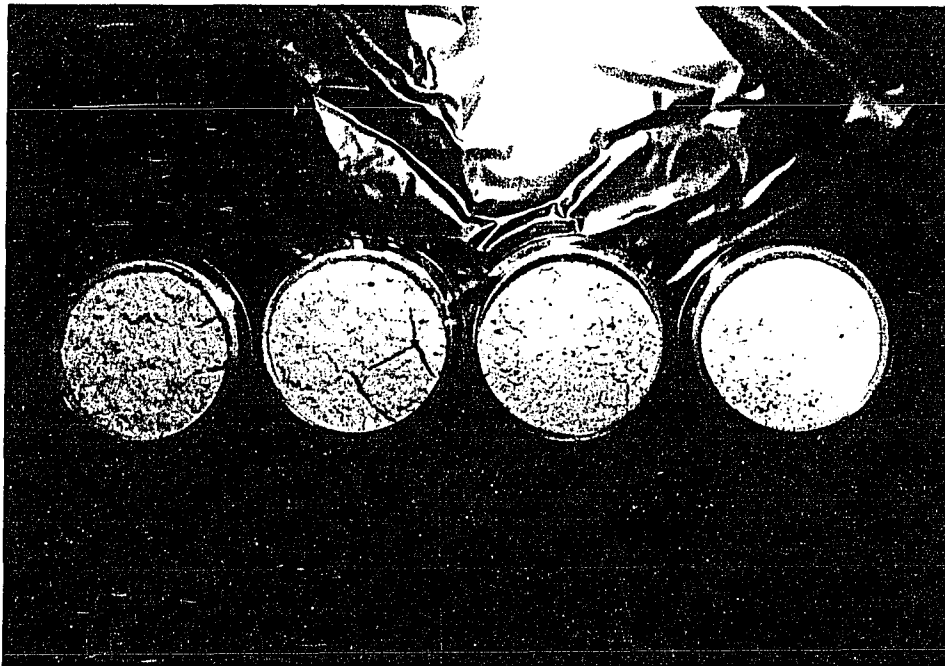
The sedimentation test results have shown good agreement with those found in the expansion tests. This technique provides a good indication regarding the influence of chemicals on the bentonite structure. However, its application tends to be limited by the characteristics of the leachate encountered in the field.

4.1.4 Cracking Observations

The cracking observations made for SS-100 with sodium hydroxide solutions are presented in Figures 4-6A and 4-6B. Initially, SS-100 with water and sodium hydroxide 0.4 percent solution expanded normally, and the slurry appeared to be integrated. When highly concentrated solutions were used, SS-100 was completely dissolved and appeared as a granular-like material. As the moisture gradually reduced shrinkage fissures and cracks were found in specimens for all the liquids tested. The cracking patterns tended to be more densely populated with the increasing electrolyte concentration. Fissures for 4 percent solution were hardly visible. The sizes of the fissures and cracks were considerably larger for water and 0.4 percent sodium hydroxide solution compared to those for 2 percent and 4 percent sodium hydroxide solutions. Alther et al. (1985) concluded that the more flocculated structure tends to draw

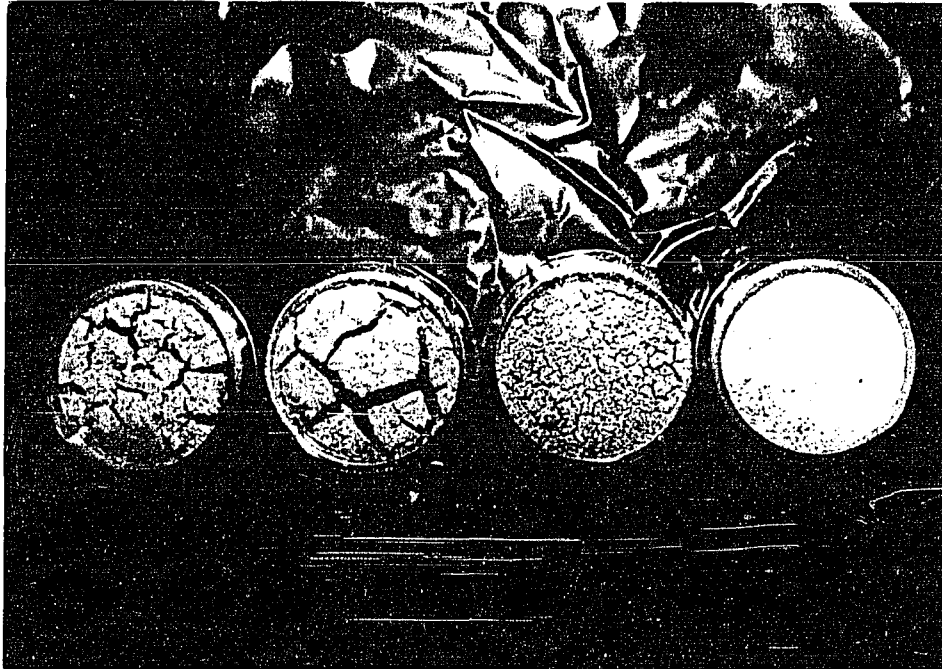


(a) Drying Period - Instant



(b) Drying Period - One Day

Figure 4-6A Cracking Patterns Observed in Response to Various Pore Fluid Conditions and Drying Periods. Fluids used (from left to right): Water, Sodium Hydroxide Solutions, 0.4%, 2%, and 4%.



(a) Drying Period - One Week



(b) Drying Period - Four Weeks

Figure 4-6B Cracking Patterns Observed in Response to Various Pore Fluid Conditions and Drying Periods. Fluids used (from left to right): Water, Sodium Hydroxide Solutions, 0.4%, 2%, and 4%.

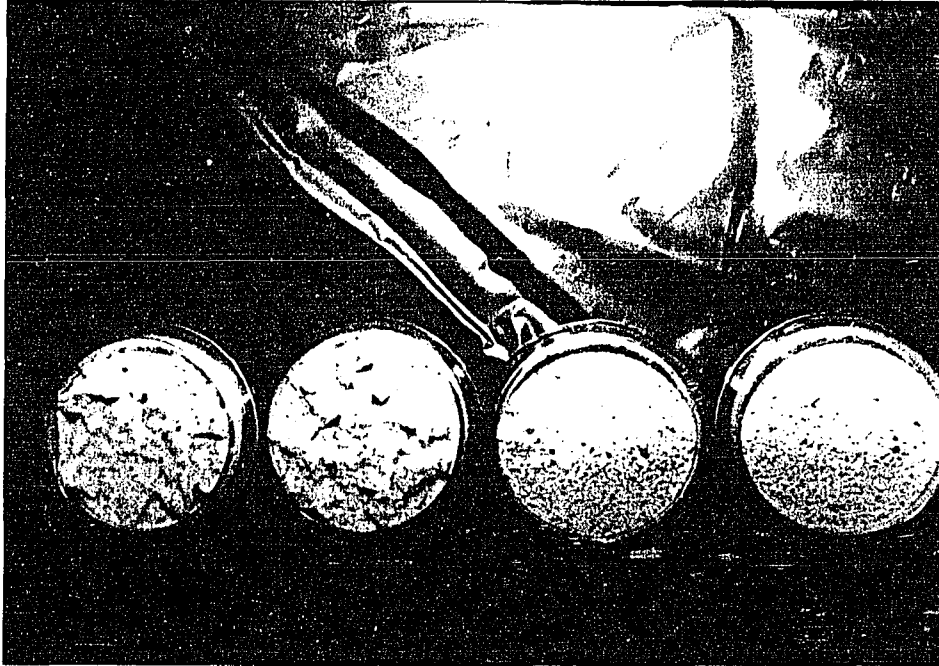
the particles closer together in all planes as drying progresses. Therefore, the cracking pattern of more flocculated specimens would result in more cracks and a more densely populated cracking pattern.

In one week, essentially all specimens were dry. Specimens for water and 0.4 percent solution showed the widest cracks. Specimens for higher concentrations showed a small increase in the openings of the cracks. However, they appeared to be relatively intact.

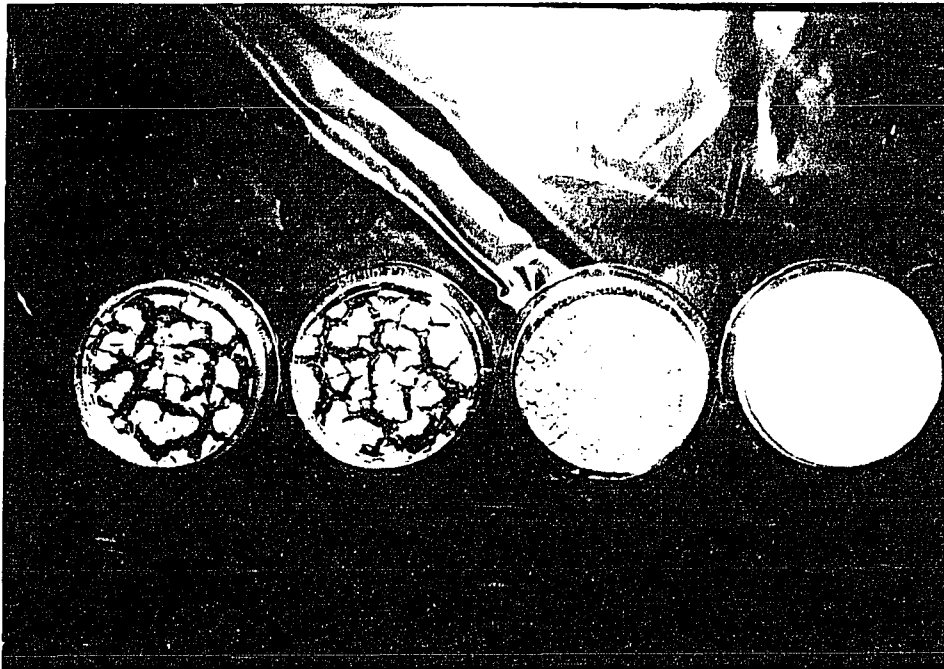
For the following three weeks, little change was observed for all the specimens. The second cycle of cracking observation was then conducted. The samples were treated as before. As shown in Figure 4-6C, cracks and fissures disappeared and the samples were essentially restored to a condition that existed before drying. A week later, the samples were completely dry. All the specimens responded in a manner similar to that after the first drying cycle. The observation of the cracking pattern on SS-100 agrees with others findings. It provides an indication of the effect of different liquids on a soil at their initial contact. However, it is not useful for long term and cyclic observations.

4.2 Permeability Tests

Three hundred and two permeability measurements were performed in this study. The tests were conducted with three types of permeameters. The consolidometer permeability tests were performed primarily with organic fluids. The flexible



(a) Drying Period - Instant



(b) Drying Period - One Week

Figure 4-6C Cracking Patterns Observed at 2nd Cycle in Response to Various Pore Fluid Conditions and Drying Periods. Fluids used (from left to right): Water, Sodium Hydroxide Solutions, 0.4%, 2%, and 4%.

wall permeameter was initially used with water. Then, for long term observations, it was used with leachate and phenol in high concentration. The triple ring permeability tests were initially conducted with water to verify the functioning of the apparatus, then it was used with sodium hydroxide solutions.

The measured permeability value, k , was normalized by multiplying it with a permeant ratio as defined in Equation 2.3. It is presented as a function of void ratio, e , to observe the chemical effect on the specimen from the volumetric viewpoint. The usual presentation with a plot of e on a natural scale versus k on a logarithmic scale will yield a linear relationship for most of the soils (Lambe and Whitman, 1969).

4.2.1 Consolidometer Permeability Tests

The consolidometer permeability measurements were conducted primarily with organic permeants at different consolidation pressures. Soil mixtures tested included 10SS, 10SS30F, 5SS30F and 30F. Permeability values were obtained by direct measurements and indirect computations. All the tests were performed in duplicate.

4.2.1.1 Tests with Organic Permeants As shown in Figures 4-7A through 4-7J, the permeability for 10SS specimens tested with water or organic permeants generally decreased with the void ratio. However, its variation became less when the void ratio was close to or less than 0.60 despite the differences in the nature of the permeants.

PERMEABILITY TEST

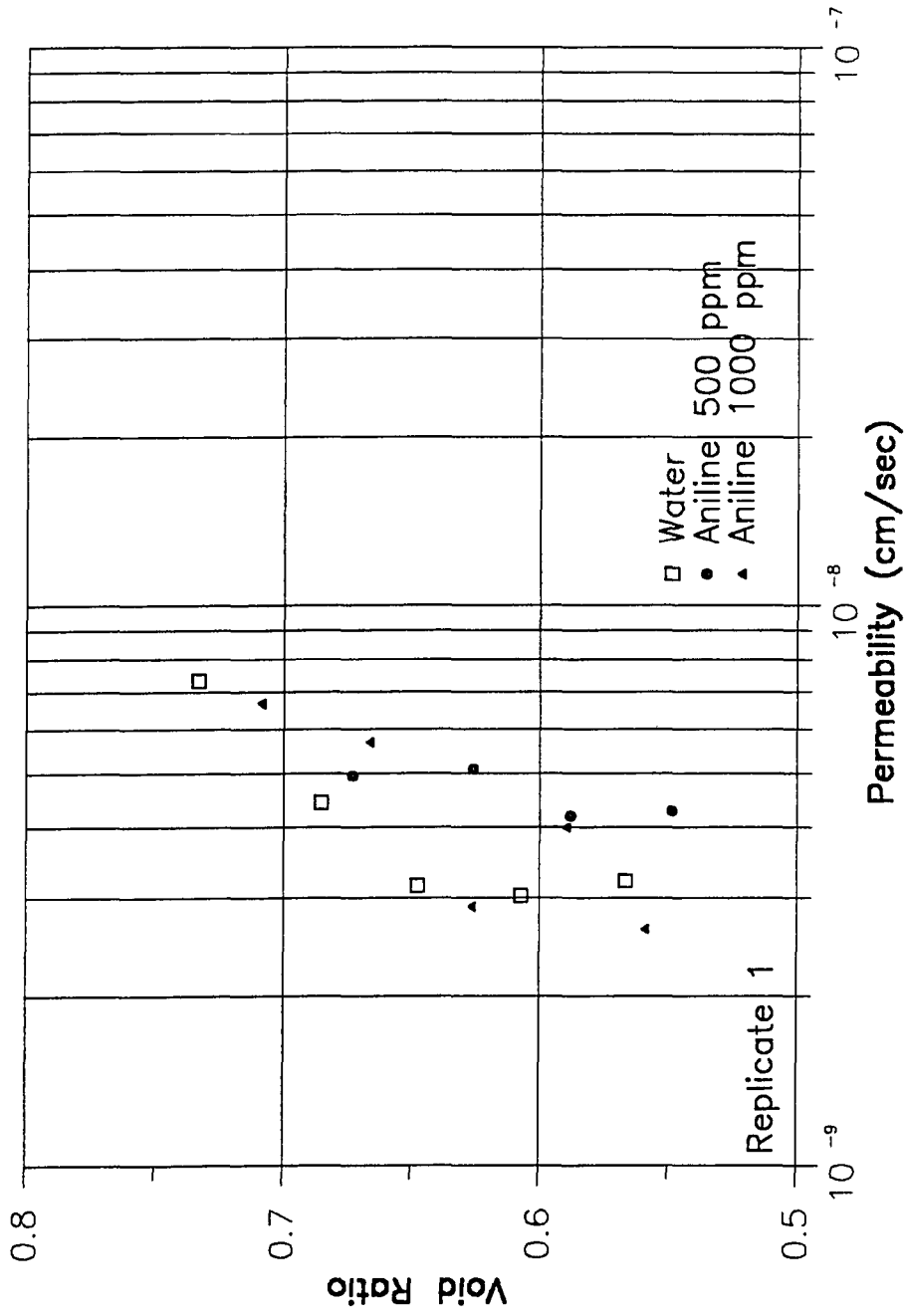


Figure 4-7A Permeability of 10SS Samples with Water and Aniline in a Consolidometer Permeameter (Replicate 1).

PERMEABILITY TEST

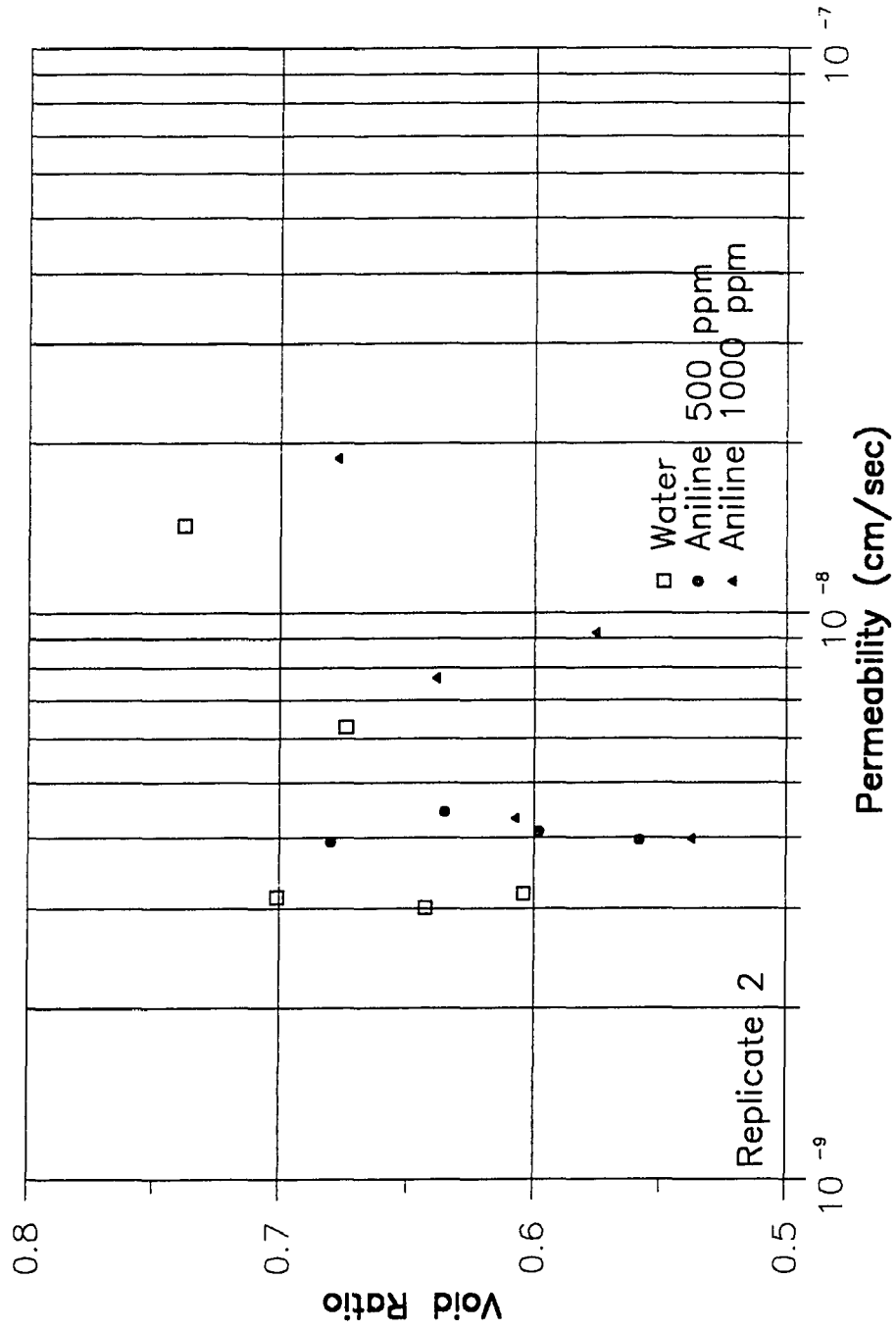


Figure 4-7B Permeability of 10SS Samples with Water and Aniline in a Consolidometer Permeameter (Replicate 2).

PERMEABILITY TEST

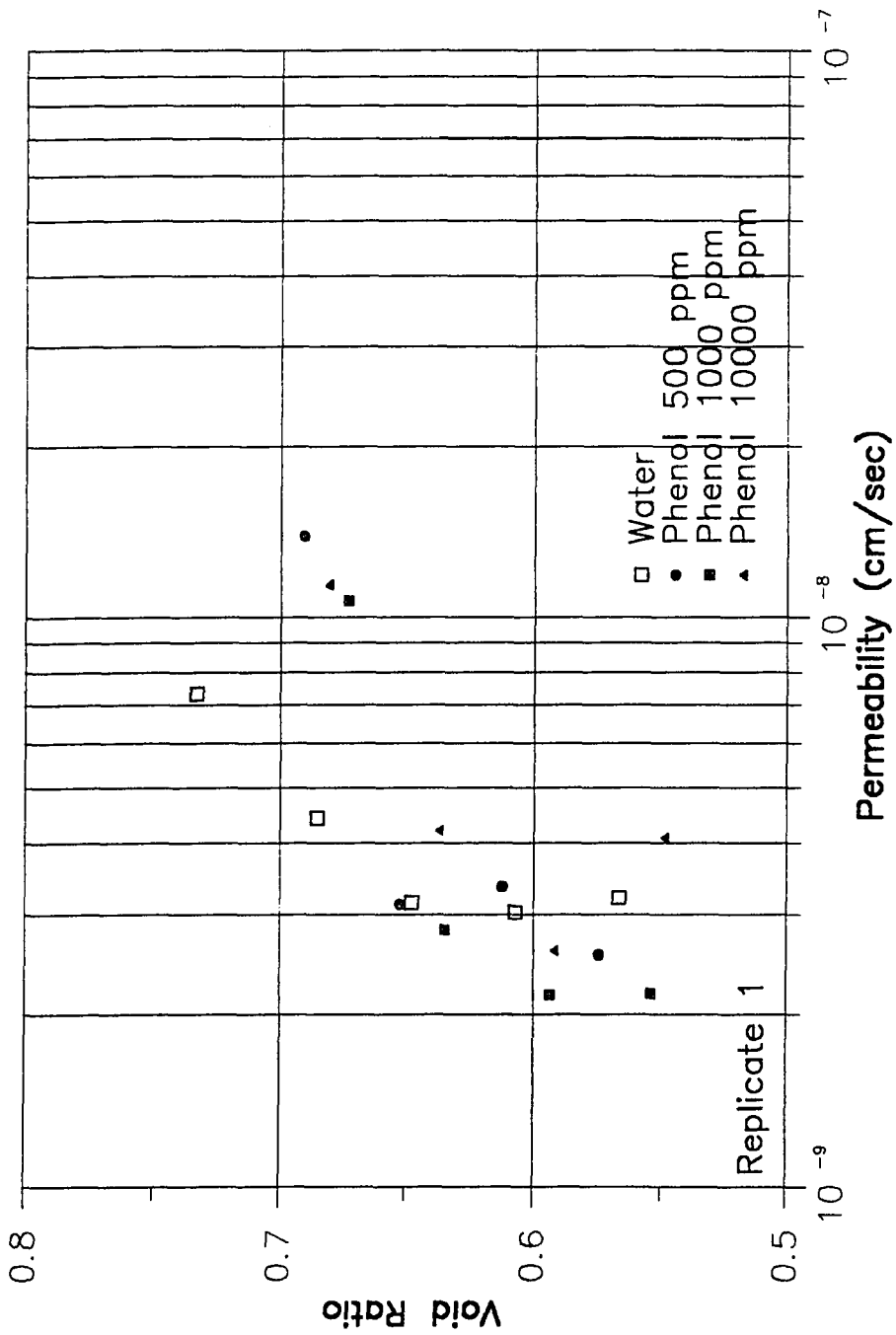


Figure 4-7C Permeability of 10SS Samples with Water and Phenol in a Consolidometer Permeameter (Replicate 1).

PERMEABILITY TEST

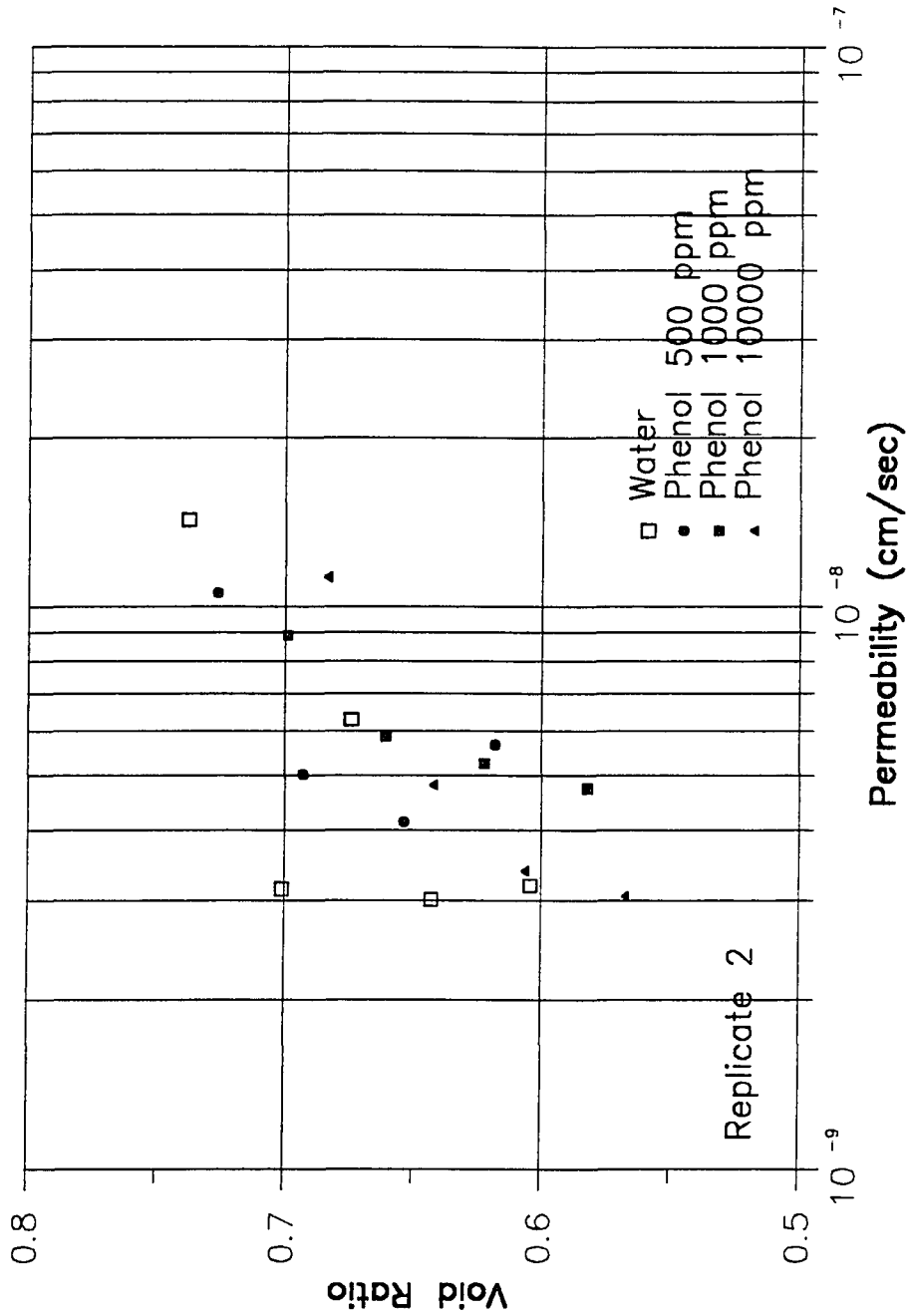


Figure 4-7D Permeability of 10SS Samples with Water and Phenol in a consolidometer Permeameter (Replicate 2).

PERMEABILITY TEST

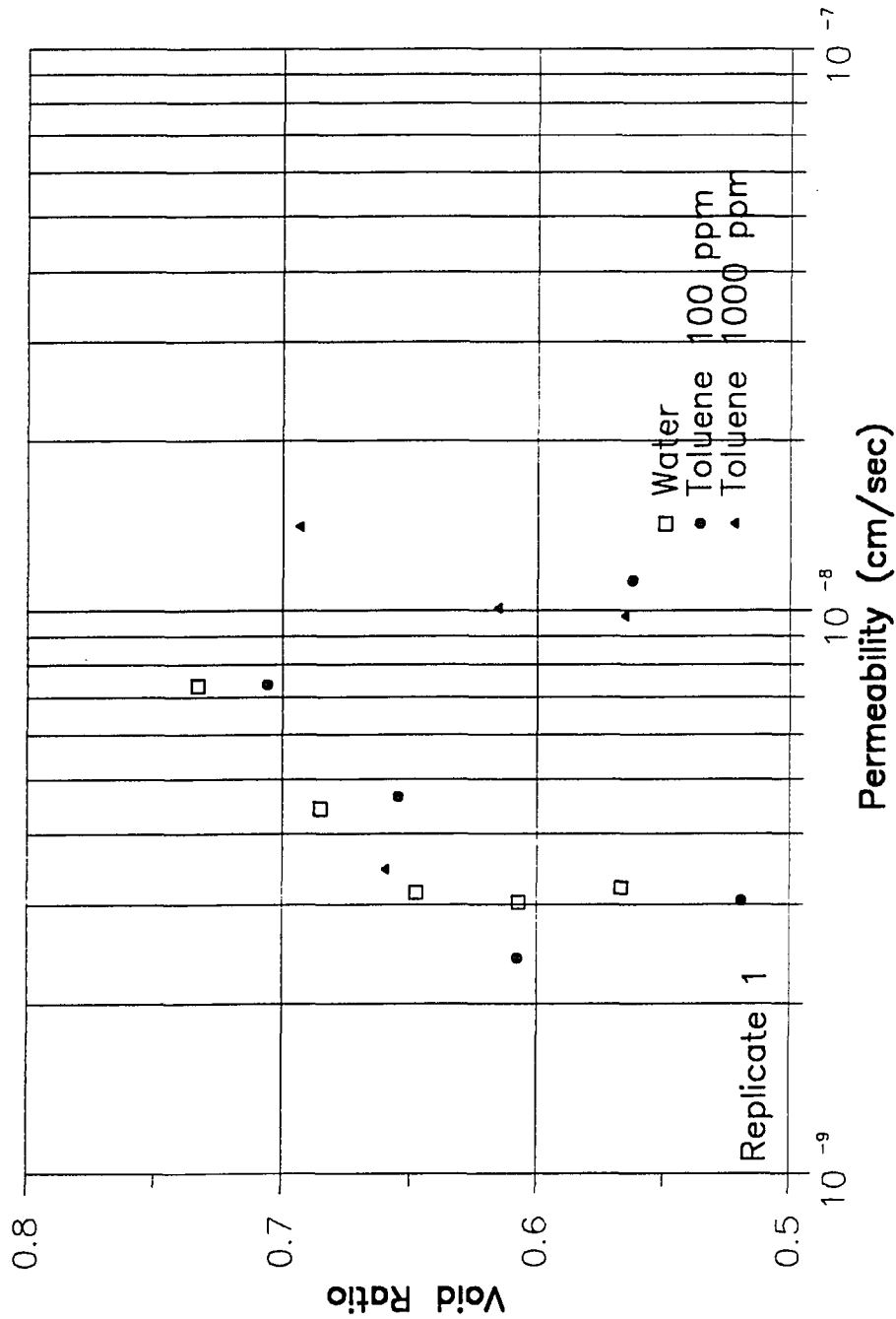


Figure 4-7E Permeability of 10SS Samples with Water and Toluene in a Consolidometer Permeameter (Replicate 1).

PERMEABILITY TEST

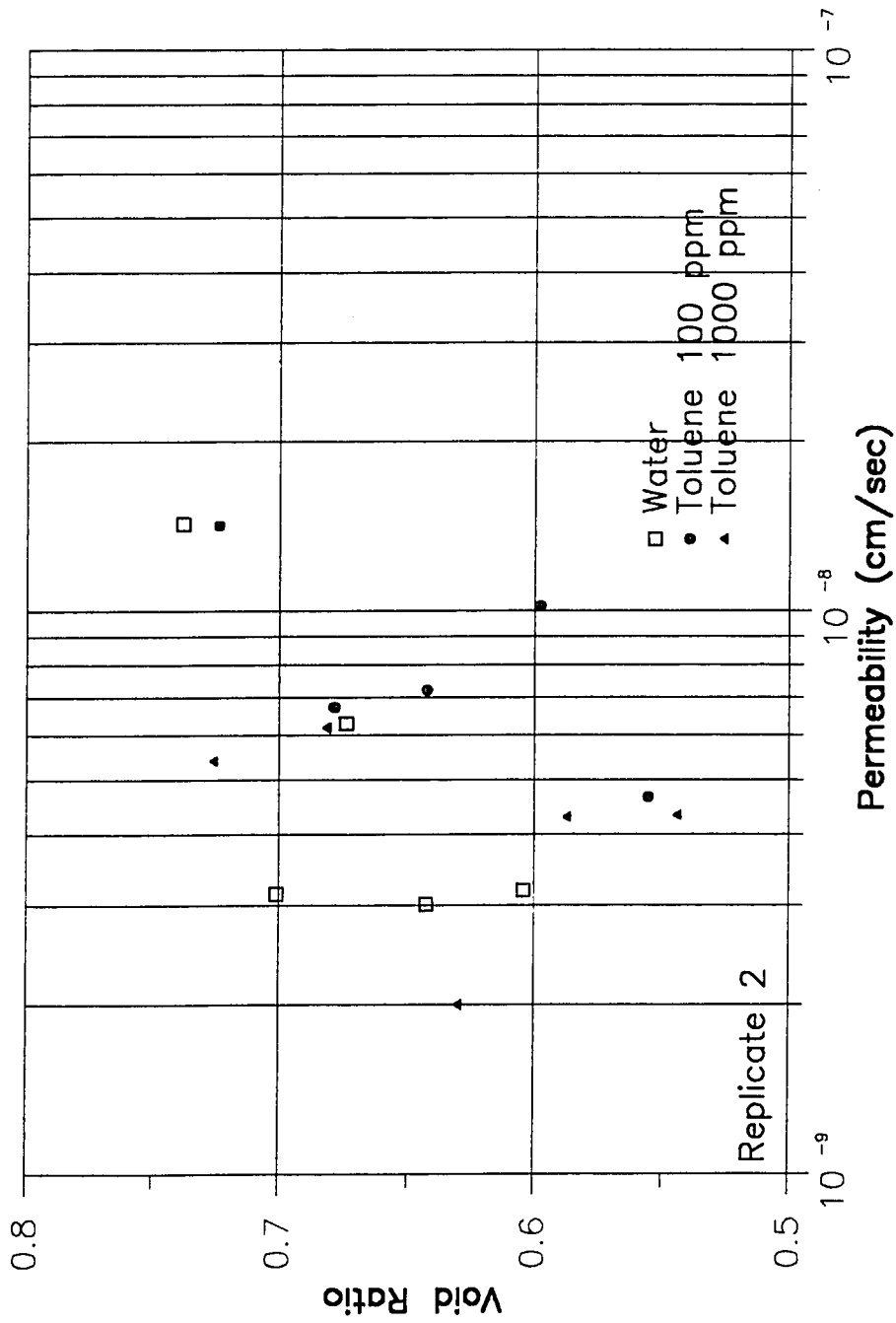


Figure 4-7F Permeability of 10SS Samples with Water and Toluene in a Consolidometer Permeameter (Replicate 2).

PERMEABILITY TEST

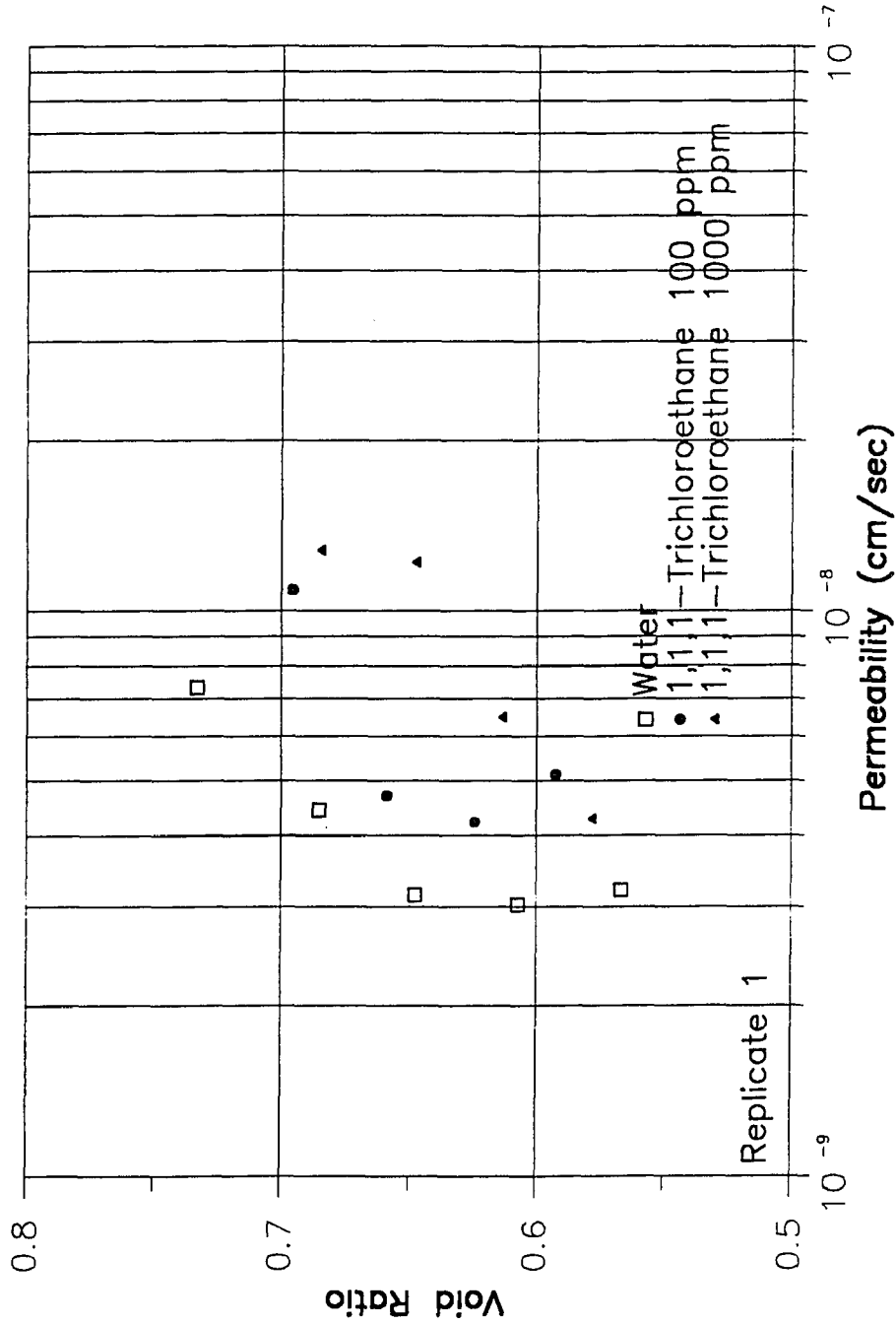


Figure 4-7G Permeability of 10SS Samples with Water and 1,1,1-Trichloroethane in a Consolidometer Permeameter (Replicate 1).

PERMEABILITY TEST

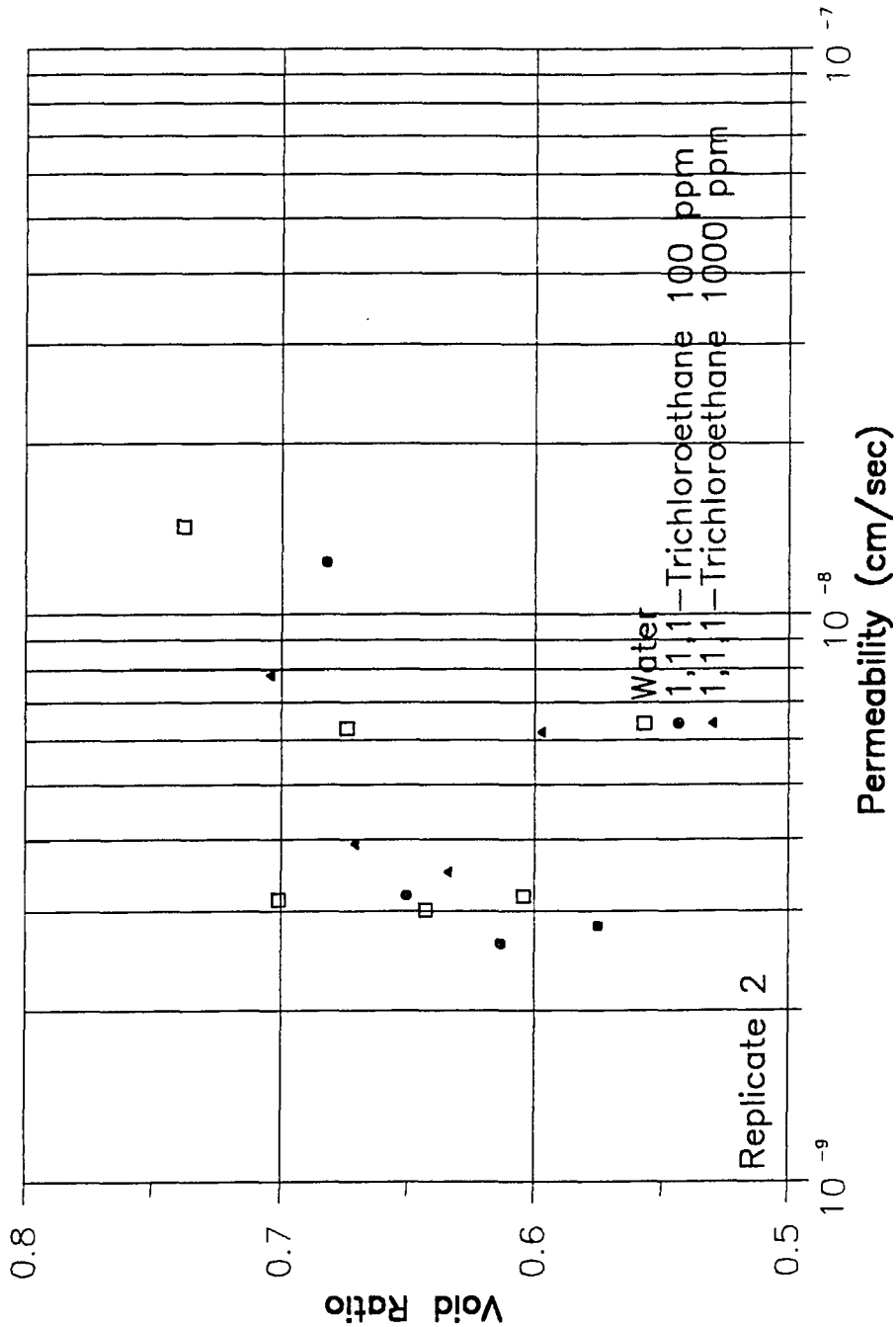


Figure 4-7H Permeability of 10SS Samples with Water and 1,1,1-Trichloroethane in a Consolidometer Permeameter (Replicate 2).

PERMEABILITY TEST

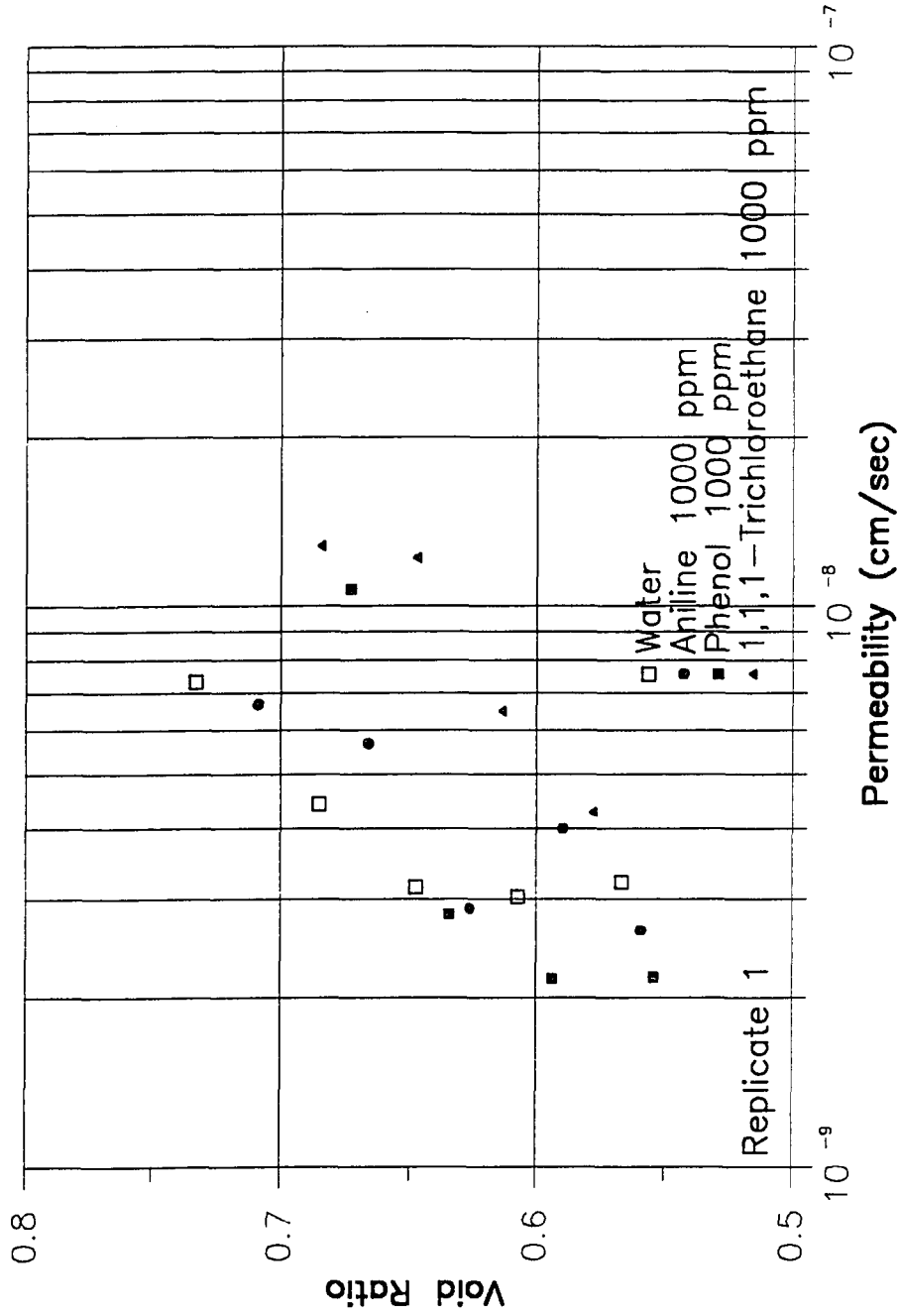


Figure 4-7I Comparison of Permeability values for 10SS Samples with Various Organic Permeants in a Consolidometer Permeameter (Replicate 1).

PERMEABILITY TEST

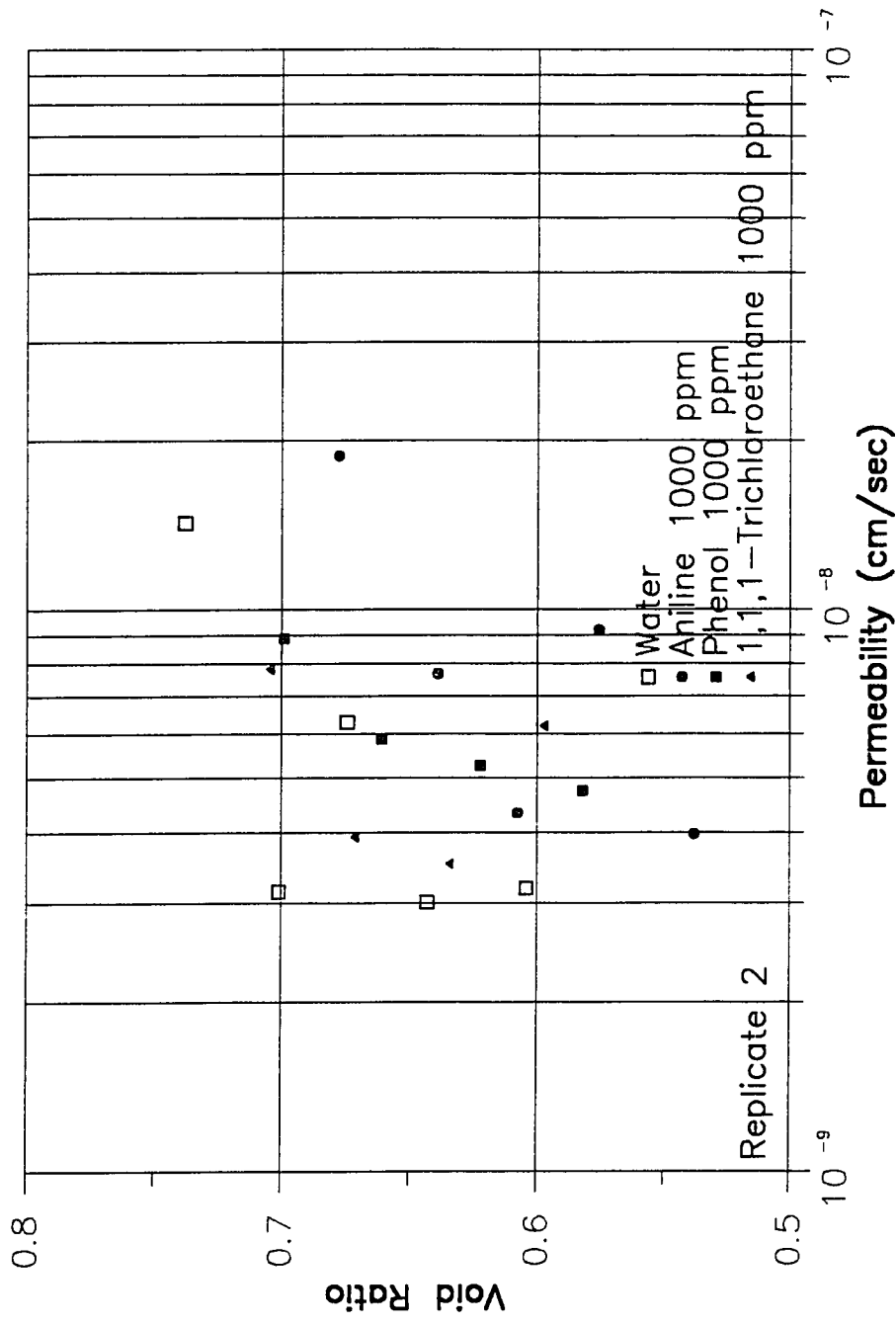


Figure 4-7J Comparison of Permeability Values for 10SS Samples with Various Organic Permeants in a Consolidometer Permeameter (Replicate 2).

To further define the relationship between void ratio and permeability, statistical correlation technique was applied. For 10SS specimens that have a void ratio at 0.60 or higher, Pearson's coefficient, $r(e,k)$, was found to be 0.4799. For the same samples with a void ratio less than 0.60, the $r(e,k)$ dropped to 0.0705.

A null hypothesis, H_0 , was developed to state that there is no correlation between e and k . Student t-tests were conducted to test whether H_0 is true or not. The results show that H_0 can be rejected with 95 percent confidence for specimen with a void ratio greater than 0.60. However, H_0 is true for specimen with a void ratio less than 0.60. This implies that e and k have correlation only when e is greater than 0.60. The changes in void ratio can not be relied upon to show permeability changes for bentonite mixtures.

Similar analysis was conducted for fly ash samples. The results show that $r(e,k)$ is 0.9778 and 0.9576 for 5SS30F and 10SS30F samples, respectively. The results of t-tests show that the void ratio has a very strong correlation to the permeability.

Test data were further analyzed with the Spearman rank test. The results of the analysis show a similar tendency to those found in the Pearson test.

A comparison of the data points of organic permeants shows that permeability varied little with the type and concentration of the liquids. It appears at a void ratio above 0.6, the permeability values of organic permeants were

generally higher than with water. The difference varied from negligible to about two-to-three-fold. The higher the void ratio, the greater the difference. When the void ratio decreased, most of the data points tended to converge to that of water. Organic permeants appeared to have an adverse effect on the permeability. However, when the consolidation pressure increased and the void ratio decreased, this effect tended to be minimized.

The test results contrast with the findings of others (Bowders, 1985; Uppot, 1984). They found that diluted organic chemicals have little effect on the permeability of the clays that were tested. A comparison of the test performance showed that their specimens were either compacted or had been consolidated to a pressure of 172 kPa or higher. Permeability has been reported to decrease with the increase of consolidation pressure (Boynton, 1983). In this study, 10SS specimens at a void ratio of 0.6 or less generally correspond to a consolidation pressure of 100 kPa or higher. The minimized chemical effect observed here and those reported by others thus can be attributed to the effect of higher consolidation pressure. To conclude that dilute chemicals are not detrimental to permeability without considering the effect of consolidation thus is not appropriate.

The permeability increase can be explained using the Gouy-Chapman theory. The double layer thickness reduces with the increase of electrolyte concentration and the decrease of the dielectric constant. Many researches have indicated that

the double layer thickness has a direct relationship to the permeability of clayey materials (Alther et al., 1985; Bowders, 1985; Evans et al., 1983; Uppot, 1984). In general, expansion of the double layer causes soil to swell, promotes a dispersed arrangement of soil particles, and tends to reduce permeability. Contraction of the double layer encourages soil to shrink, promotes a flocculated arrangement of soil particles, generates cracks or macropores, and tends to increase the permeability.

As noted in the expansion tests, the liquids for the permeability tests did show some inhibition of the bentonite expansion. A reduced expansion tendency indicates a possible increase in permeability. Test results generally agree with this hypothesis.

4.2.1.2 Tests with Chromium Figures 4-7K and 4-7L show permeability with 50 ppm chromium. At a higher void ratio, the permeability was similar to that of the organic permeants. However, as the void ratio decreased from 0.70 to 0.65, the permeability increased by one order of magnitude. For void ratios less than 0.65, the permeability slowly decreased again. This shows a completely opposite tendency to those found with other permeants. Tests were conducted with four repetitions and all rendered similar results.

The chromium solution was made from potassium chromate (K_2CrO_4). It showed a conductivity of 340 μ mho which was two orders of magnitude higher than the organic permeants. This is due to the potassium and the high valence of the chromium

PERMEABILITY TEST

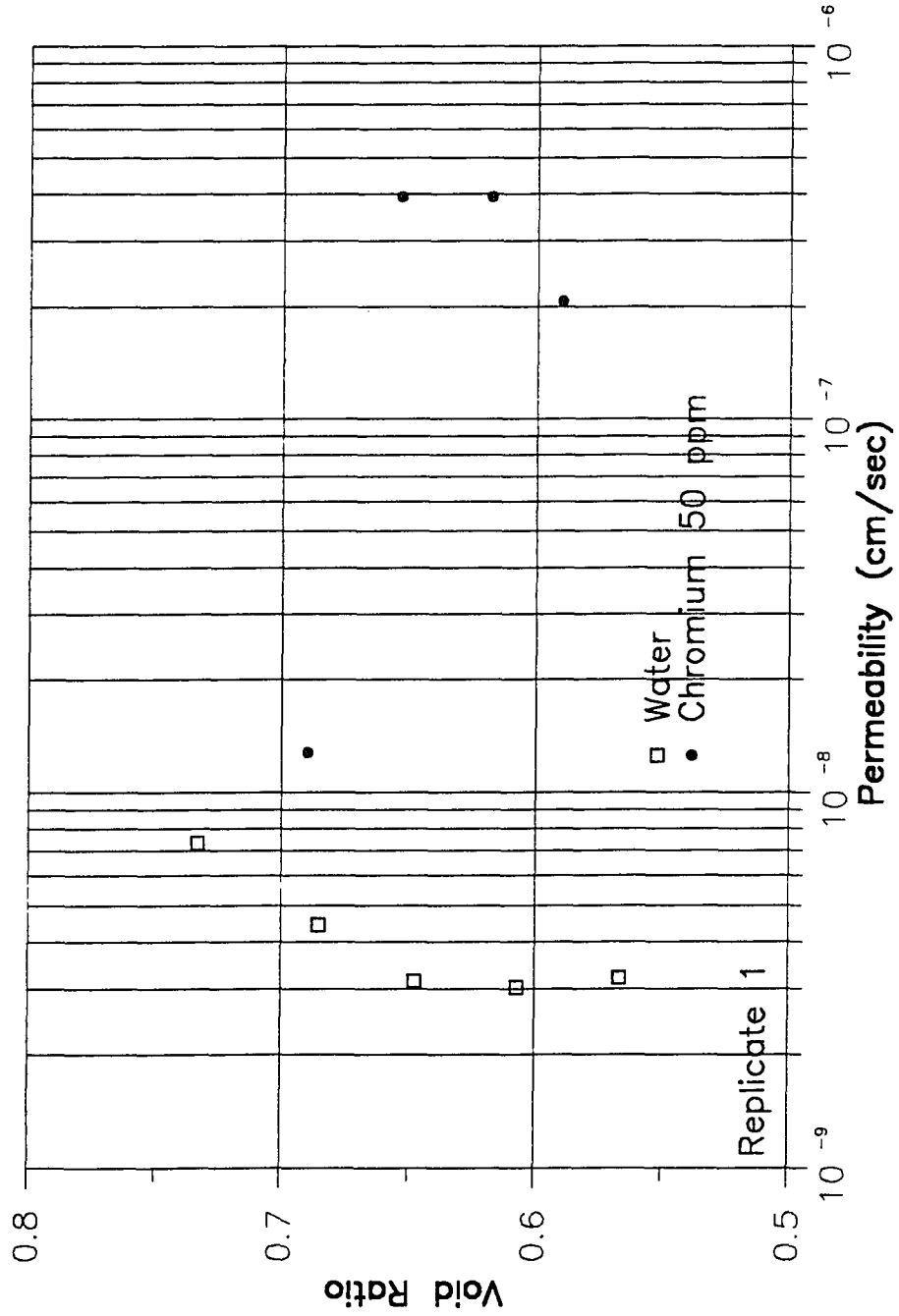


Figure 4-7K Permeability of 10SS Samples with Water and Chromium in a Consolidometer Permeameter (Replicate 1).

PERMEABILITY TEST

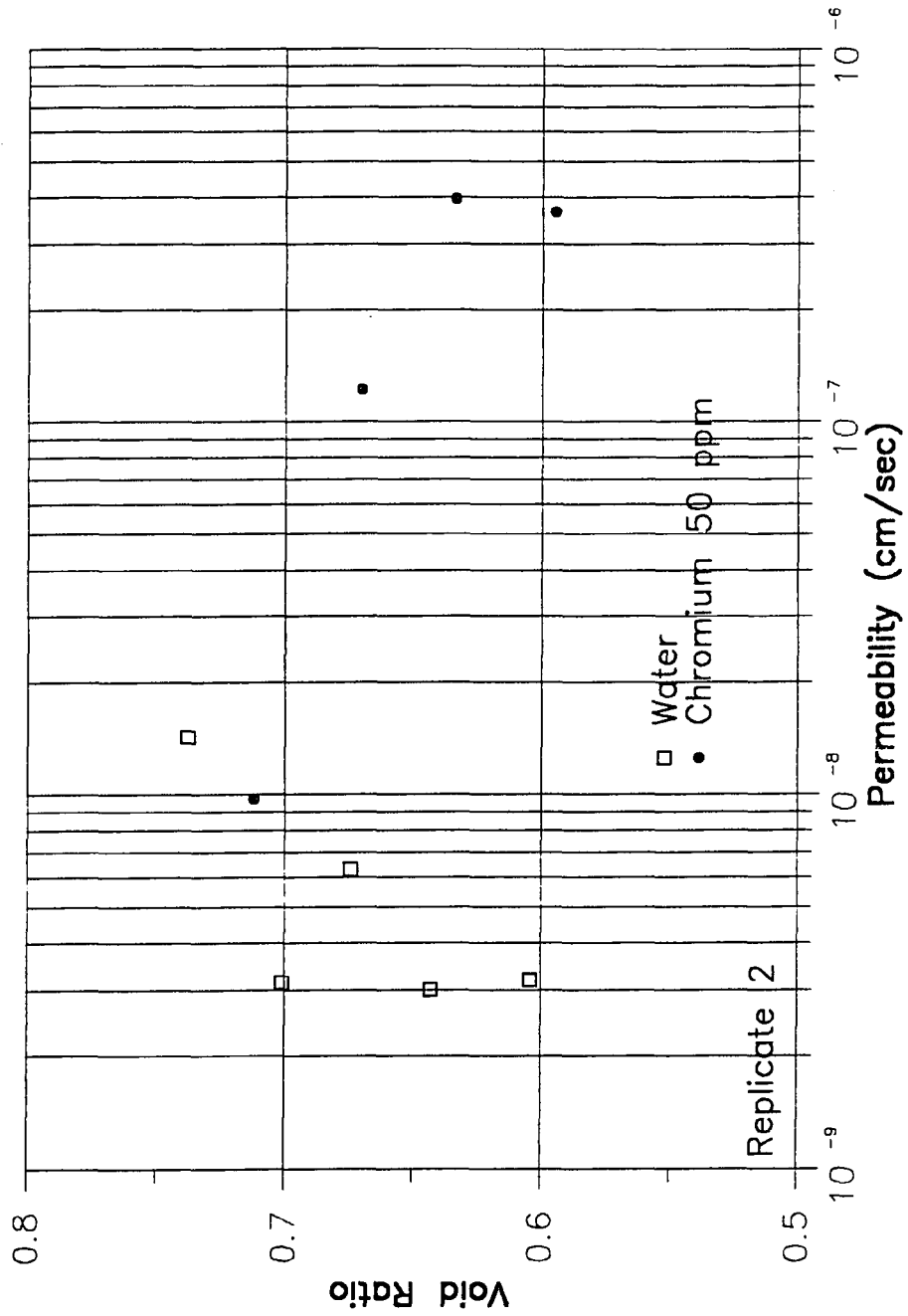


Figure 4-7L Permeability of 10SS Samples with Water and Chromium in a Consolidometer Permeameter (Replicate 2).

in the solution. The higher the conductivity, the higher the electrolyte concentration. The double layer thickness decreases as the electrolyte concentration and ion valance increase. The degradation of permeability could be caused by the coupling effects of the high electrolyte concentration and the hexavalent chromium ions. Alther et al. (1985) conducted permeability tests on bentonite slurry specimens with 16 inorganic aqueous solutions. They found that the solutions with potassium cations induced the largest permeability increases with increasing electrolyte concentration. They also observed that doubly charged cations have a greater initial effect on permeability than singly charged cations.

4.2.2 Flexible Wall Permeability Tests

Twenty-nine flexible wall permeability measurements were conducted on various soil mixtures. A total of 18 specimens were tested with water, nine with phenol, and two with landfill leachate. All specimens were consolidated to different stresses to observe the effect of consolidation on permeability.

4.2.2.1 Tests with Water As shown in Figure 4-8, the permeability of the 10SS sample confined with neoprene membrane is one order of magnitude greater than that with latex membrane. As noted previously, the neoprene is believed to be more compatible with corrosive chemicals than latex rubber. The test results indicate that it is not appropriate to use neoprene membranes at low confining stresses. Dunn

PERMEABILITY TEST

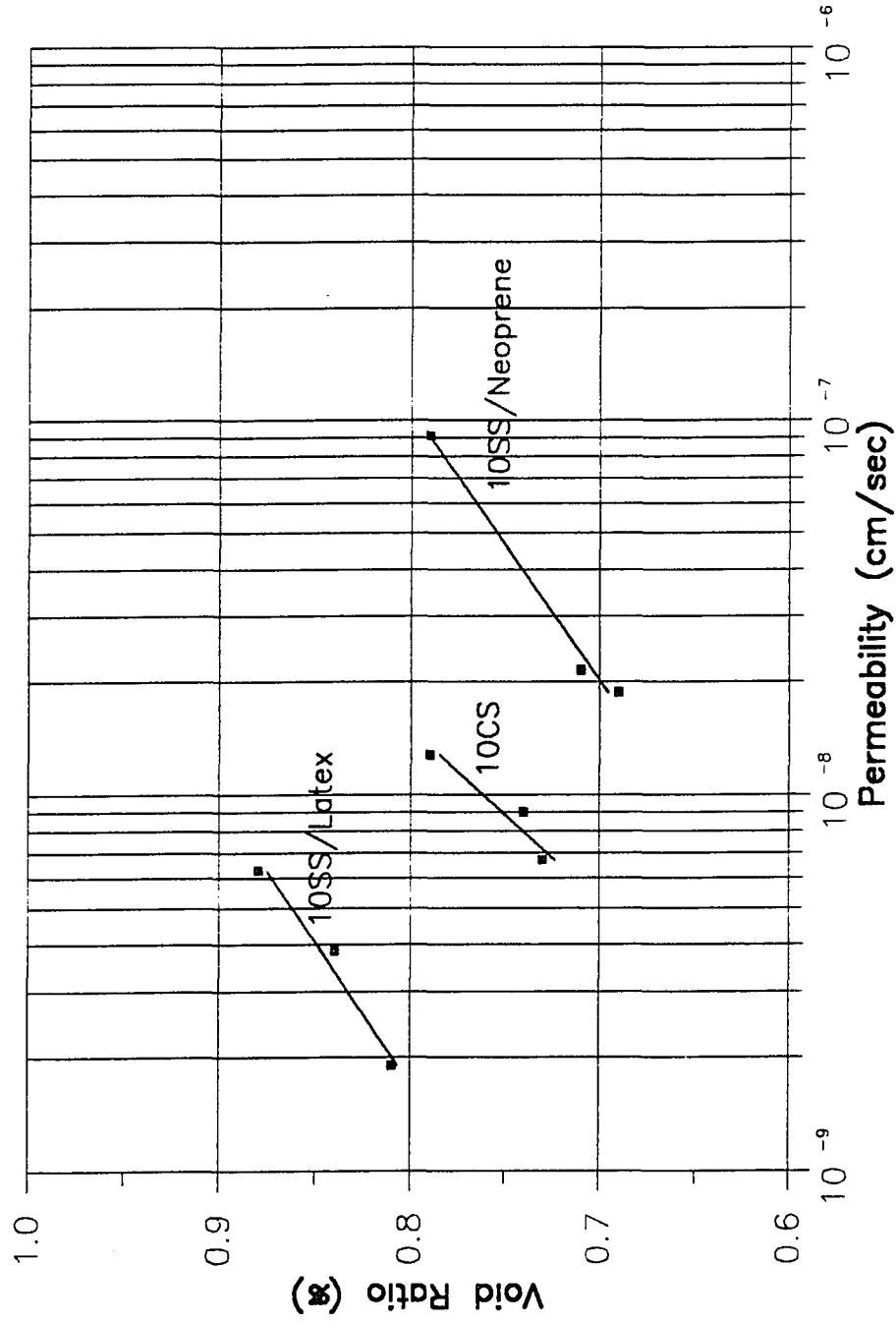


Figure 4-8 Permeability of Bentonite Mixes with Water in a Flexible Wall Permeameter.

(1983) has drawn similar conclusions.

Figure 4-8 also shows that 10CS was about seven-to-nine-fold higher in permeability than 10SS. As noted in the expansion test, SS-100 has been chemically treated and formulated with an additional dispersing agent. Though CS-200 expands more than SS-100 does, a more dispersed structure generally shows less permeability as compared with a relatively flocculated structure (Olston, 1988).

4.2.2.2 Tests with Phenol As shown in Figure 4-9A, phenol 60,000 ppm appeared to have no detrimental effect on permeability. However, its e-logk pattern showed great variation from that of water. Permeability varied sharply with void ratio if the sample contained an appreciable amounts of fines. Permeability changed much slower with void ratio as compared with water. As noted in the sedimentation test, phenol 60,000 ppm may cause some flocculation of the soil. A more flocculated structure tends to have higher compressibility and permeability (Lambe, 1969).

Figures 4-9B and 4-9C show the permeability values in relation to the pore volume displacements and the permeation time, respectively. Though there is some scatter in the data point, the permeability generally showed a descending tendency with increasing pore volume displacement.

Evans et al. (1985) conducted permeability tests for a soil bentonite backfill mix under a consolidation pressure of 207 kPa for diluted acetic acid, carbontetrachloride, and aniline. They found no appreciable changes in permeability.

PERMEABILITY TEST

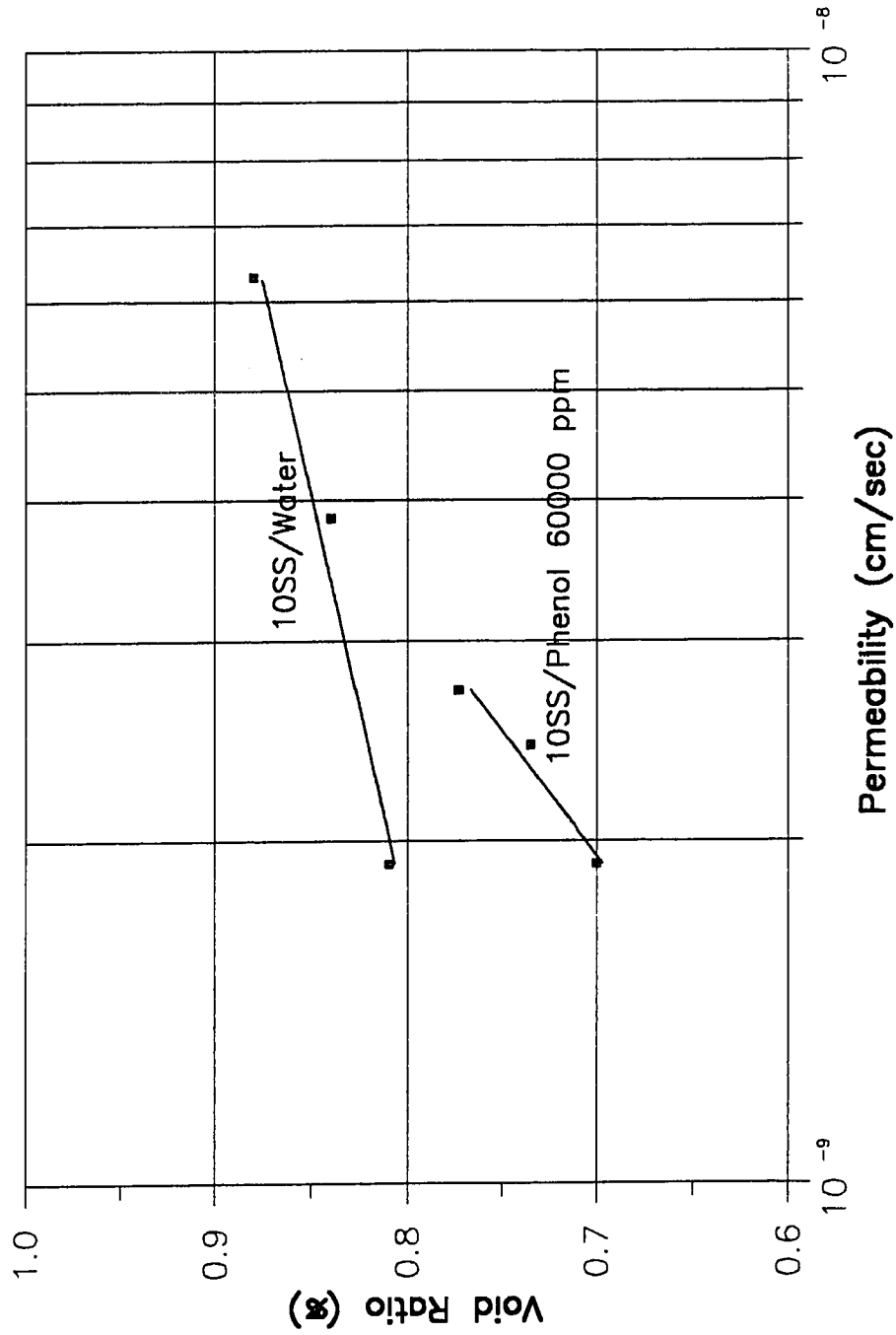


Figure 4-9A Permeability of 10SS Samples with Water and Phenol in a Flexible Wall Permeameter.

PERMEABILITY TEST

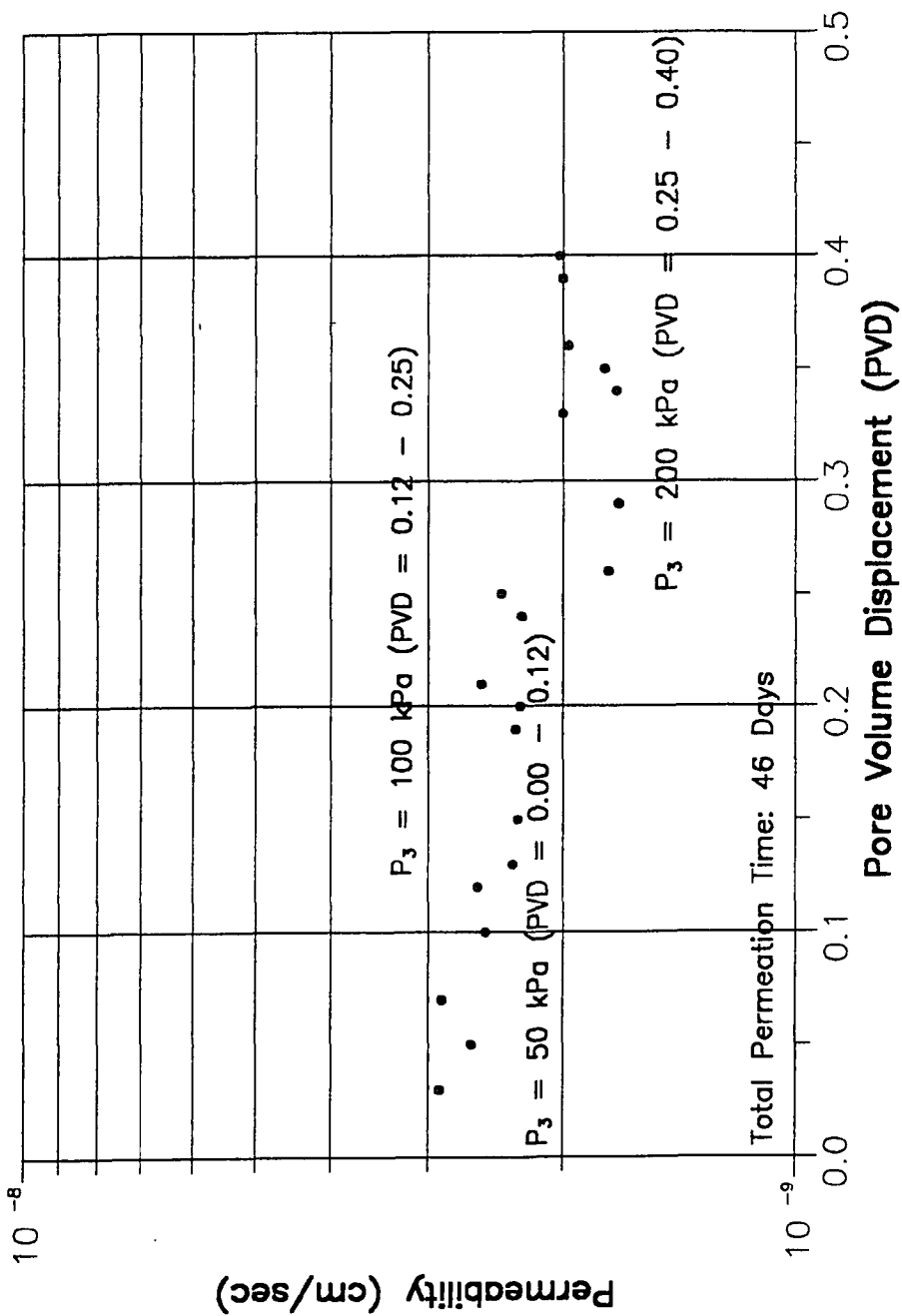


Figure 4-9B Permeability Versus Pore Volume Displacement for a 10SS Sample with Phenol 60,000 ppm in a Flexible Wall Permeameter.

PERMEABILITY TEST

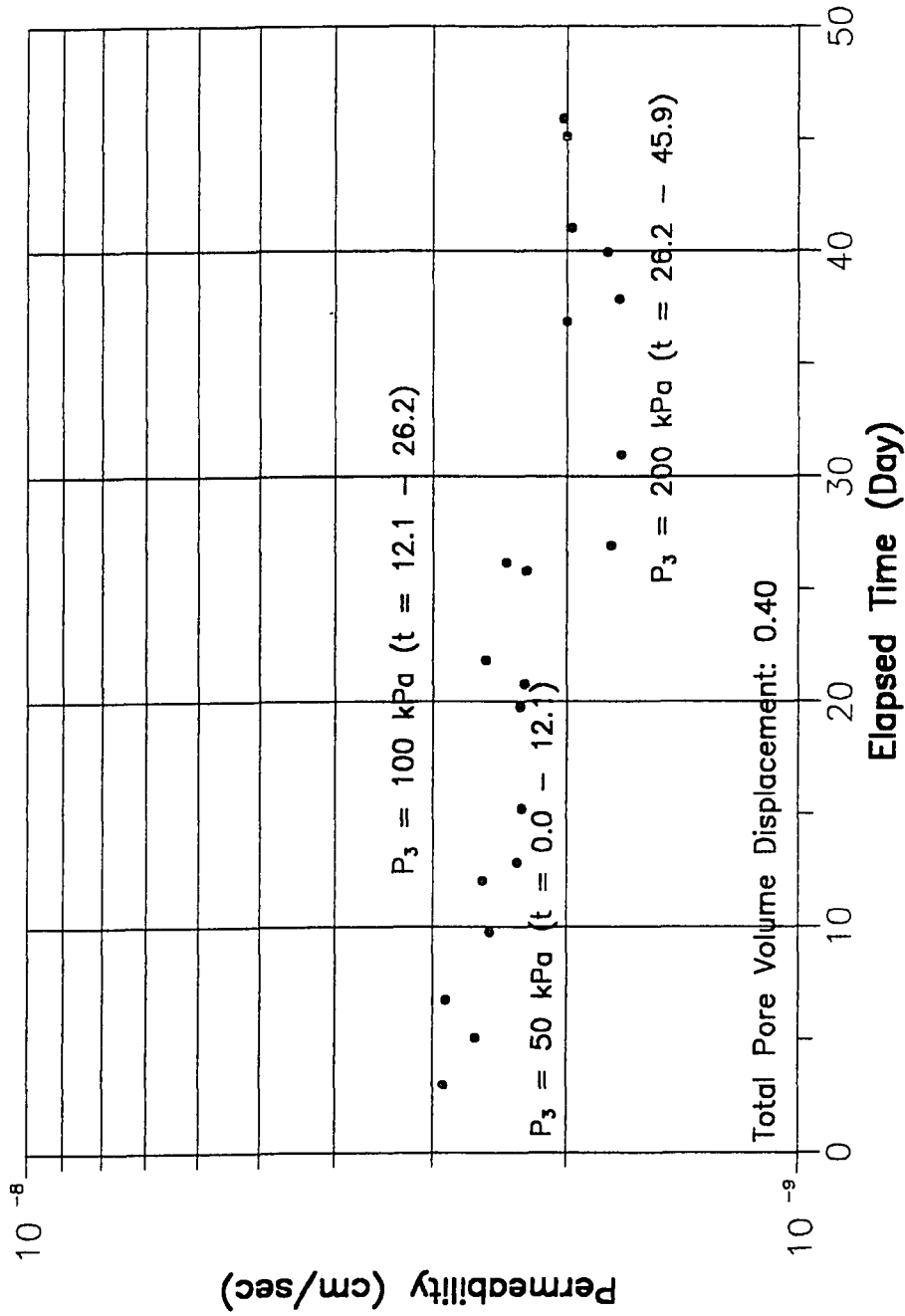


Figure 4-9C Permeability Versus Permeation Time for a 10SS Sample with Phenol 60,000 ppm in a Flexible Wall Permeameter.

They concluded that the aqueous organic solutions had a significantly less detrimental effect upon the permeability than the pure compounds. Tests with phenol 60,000 ppm agree with these results. However, as noted previously, the consolidation effect tends to decrease permeability. Tests with phenol and those performed by Evans et al. (1985) all had been consolidated under a pressure of 200 kPa or higher. The minor changes in the permeability thus can be attributed to the effect of a higher consolidation pressure. Since the consolidation effect may not be present all the time to conclude that phenol 60,000 ppm will not have an adverse effect on the permeability is not appropriate .

4.2.2.3 Tests with Landfill Leachate The effect of landfill leachate on bentonite, SS-100, was studied. The base material used for the backfill mix was a poorly graded fine sand. An additional 20 percent, by weight, of off-site clayey fines was introduced to reduce permeability. The results of trial batches showed that a total bentonite content of 10 percent, by weight, of the backfill mix should be used to have a permeability of less than 10^{-7} cm/sec. Long term flexible wall permeability tests were conducted for about four months. Specimens were permeated with 7 to 11 pore volume displacements under a hydraulic gradient of 75.

As shown in Figures 4-10A and Figure 4-10B, the permeability for the sample with 50 kPa consolidation pressure initially dropped sharply by about three fold of its original value. Then it increased fairly rapidly and became

PERMEABILITY TEST

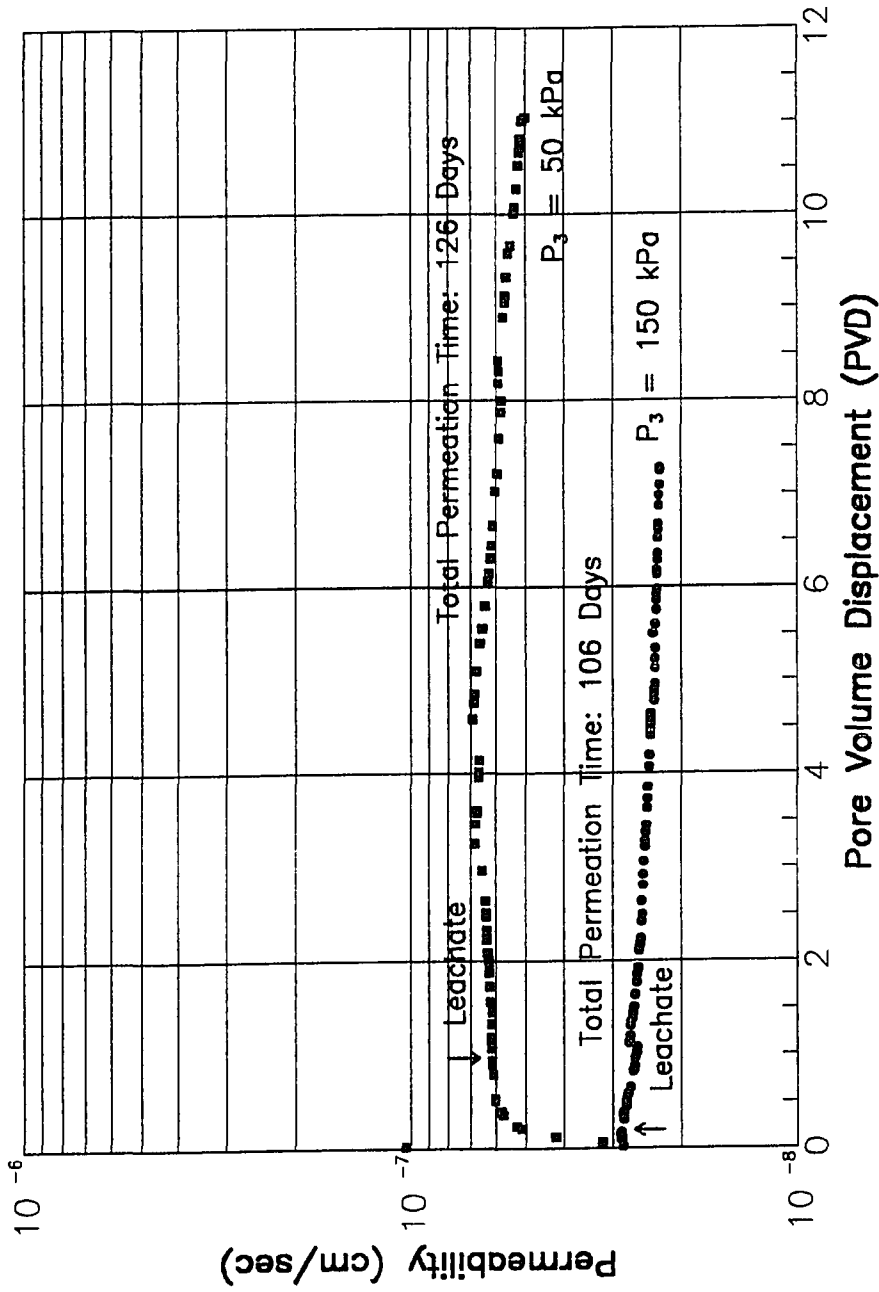


Figure 4-10A Permeability Versus Pore Volume Displacement for Bentonite Samples Consisted of Field Materials with landfill Leachate in a Flexible Wall Permeameter.

PERMEABILITY TEST

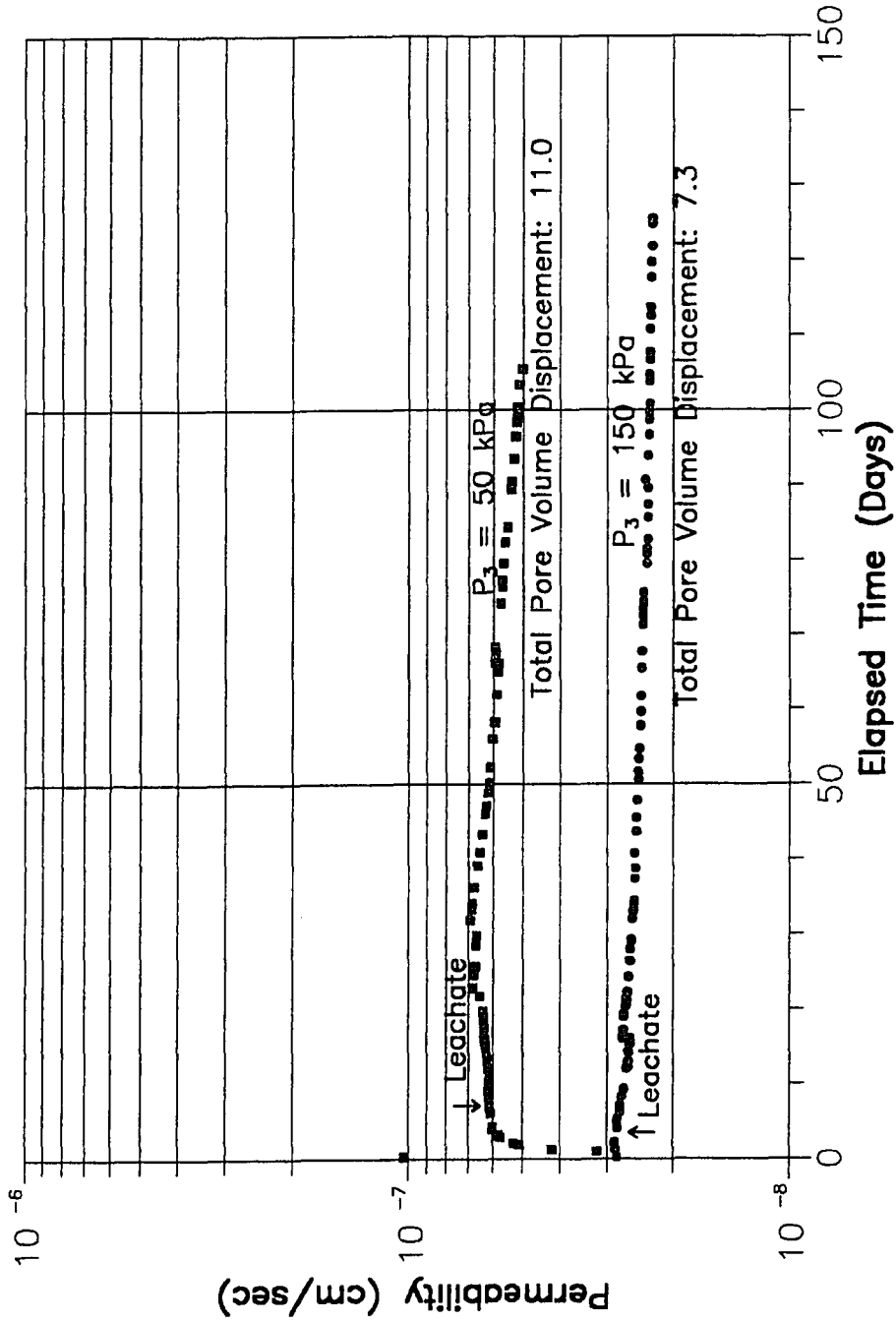


Figure 4-10B Permeability Versus Permeation Time for Bentonite Samples Consisted of Field Materials with landfill Leachate in a Flexible Wall Permeameter.

steady at about one pore volume displacement. Leachate was introduced at this time and a slight increase in the permeability was noted. However, upon continued permeation, after a six pore volume displacement, permeability dropped. The slight increase in permeability could be attributed to the soil-permeant interaction. As noted in the expansion test, bentonite expanded much less in leachate than in water. The decrease of expanded volume due to the soil-leachate interaction may result in additional voids, break the original soil structure, and promote another consolidation for the soil matrix. This consolidation tends to eliminate the additional voids and cause permeability decreases.

For specimen with 150 kPa consolidation pressure, the decreasing trend in permeability is observable. The landfill leachate appears to have no detrimental effect on permeability. This again can be attributed to the effect of higher consolidation pressure.

Daniel and Liljestrang (1984) conducted flexible wall permeability tests with landfill leachates on compacted clays. They used a consolidation pressure of 100 kPa and found similar results. Therefore, the evidence of a gradual decline in permeability with time could be attributed to the same influence as discussed.

4.2.3 Triple Ring Permeability Tests

4.2.3.1 System Verification Based on the performance of the consolidometer permeameter and the flexible wall permeameter, neither was completely satisfactory.

At low consolidation pressure, the consolidometer permeameter is not able to prevent the occurrence of sidewall leakage. It is also impossible to apply an upward gradient without taking the risk of boiling the soft specimen. The apparatus offers no provision for specimen saturation.

In a flexible wall permeameter, the specimen assembly requires considerable effort and skill. At higher concentrations, the membrane cannot be prevented from being damaged by chemicals (Acar et al., 1985). It is also impractical to test the specimen at lower hydraulic gradients (Dunn and Mitchell, 1984).

Based on these considerations, triple ring permeameter was developed to compensate for these limitations. Initial trials proved that it was functional as expected. The specimen can be assembled easily as that for a consolidometer permeameter and without using any membrane. Variable hydraulic pressures can be applied accurately to the specimen for saturation, consolidation, and permeation. The permeability also can be measured with smaller downward gradients.

A preliminary test on 10SS with water and without applying any consolidation pressure was performed. As shown in Table 4-4, the test results indicated the permeability of the material adjacent to the permeameter wall was slightly higher than that of the central portion. The minor increase could be caused by the irregular contact between the soil and the sidewall of the permeameter. Anderson et al. (1985)

observed a similar change for a double ring permeameter. When a consolidation pressure of 50 kPa was applied, the permeability values for each of the three portions of the specimen were essentially the same. The applied consolidation pressure apparently forced the soil against the rigid wall of the permeameter and thus minimized the gaps along the sidewalls and eliminated the possible leakage.

Table 4-4 Summary of Permeability Control Test Results for Triple Ring Permeameter

Type of Permeant	Void Ratio	Permeability @ 20°C (cm/sec) ¹		
		Outer	Intermediate	Center
Water ²	0.72	3.26×10^{-8}	1.89×10^{-8}	1.57×10^{-8}
Water ³	0.59	6.07×10^{-9}	5.92×10^{-9}	6.02×10^{-9}
NaOH 4.0 % ²	0.68	8.56×10^{-4}	7.63×10^{-4}	7.02×10^{-4}
NaOH 4.0 % ³	0.47	1.42×10^{-4}	1.07×10^{-4}	1.11×10^{-4}

1) All samples tested were 10SS.

2) Samples were tested without applying any consolidation pressure.

3) Samples were tested with a consolidation pressure of 50 kPa.

The performance of the permeameter was further verified with chemical permeants. A 10SS sample was permeated with a 4 percent sodium hydroxide solution. Permeability tests were conducted both with and without consolidation pressure. The permeability was found to be on the order of 10^{-4} cm/sec, indicating that the bentonite had completely degraded. However, the permeability relationship between the outer and

the inner portions was found to be similar to that with water. No significant sidewall leakage was observed whether any consolidation pressure was applied or not.

The preparation of the specimen allowed an active occurrence of soil-permeant interaction before the observation of permeability. At the time of permeability measurements, all soil-chemical interactions may have been completed. Further permeation of the liquid had no additional effect on the soil. In the experiments conducted by others where sidewall leakage was observed, soil and permeant interacted during the permeability measurement. Sidewall leakage, therefore, may occur if the permeant can cause the clay to shrink. D'Appolonia (1980) recommended that when other factors being equal, specimen that is already contaminated with the pollutant is preferable to uncontaminated specimen, because a smaller change will be induced during subsequent permeation with the pollutant.

The triple ring permeameter demonstrated satisfactory performance. Its design facilitates the assembly of slurry soil samples as compared with a flexible wall permeameter. It is more versatile than a consolidometer permeameter for pressure application. However, it needs two control panels to operate and requires more effort to manage the data. Since, in the control tests, no significant sidewall leakage occurred, further studies with the triple ring permeameter were conducted without monitoring the flow for the outer annulus. Since higher consolidation pressure may yield

a result not conservative, a pressure of only 10 kPa was applied to stabilize the sample.

4.2.3.2 Tests with Sodium Hydroxide Seventeen triple ring permeability measurements were conducted for 10SS samples with water and various concentrations of sodium hydroxide solutions. Detailed test results are presented in Appendix A.

When tested with 0.4 percent solution, permeability is not distinguishable from that of water. At a concentration of 2 percent, permeability increased by about three orders of magnitude as compared with that of water. As the concentration increased to 4 percent, the bentonite was completely degraded and showed a permeability of the order of 10^{-4} cm/sec. This is only about one order of magnitude lower than that of clean sand with water.

As noted in the index tests, the expansion of bentonite can be significantly inhibited in the sodium hydroxide solutions. The higher the concentration, the greater the effect. The less the expansion, the higher the permeability. The permeability results agree with this conclusion.

The permeability observed appear to be worse than that reported by D'Appolonia (1980). In his experiment, a bentonite mix consisted of 30 to 40 percent fines was permeated with a five percent solution of sodium hydroxide. He found the permeability increased by a factor of five to ten only. Since no detailed test performance described, the reason caused the difference may be attributed to the specimen composition or the consolidation effect, or both.

4.2.4 Effect of Fly Ash on Permeability

Figures 4-11A and 4-11B depict permeability values from a consolidometer permeameter for four soil mixes which consisted of various proportions of fly ash. Sample 10SS showed the lowest permeability whereas 10SS30F which contained an additional 30 percent fly ash showed about two orders of magnitude higher permeability. To determine the effect of bentonite content on permeability, sample 5SS30F which had only 5 percent bentonite was tested. Its permeability was about three-to-four-fold greater than that of 10SS30F. For sample 30F, where only fly ash was added to the sand, the permeability was considerably higher and was of the order of 10^{-5} cm/sec. Similar results were found when tested with flexible wall permeameter for these mixes (Figure 4-11C).

Based on these results, fly ash appears to have a detrimental effect on permeability. This is anomalous as with increasing fines the permeability should decrease. To further investigate this behavior, sedimentation tests were conducted on soil mixes with 10 percent SS-100 but different proportions of sand and fly ash. Samples of equal total dry weight were suspended within the same total volume of water (Figure 4-12). All samples settled within a short time. However, the bentonite in the 10SS sample remained in suspension for a much longer period than in the other samples. In addition, the more fly ash in a sample, the less time for bentonite to remain in suspension. The bentonite

PERMEABILITY TEST

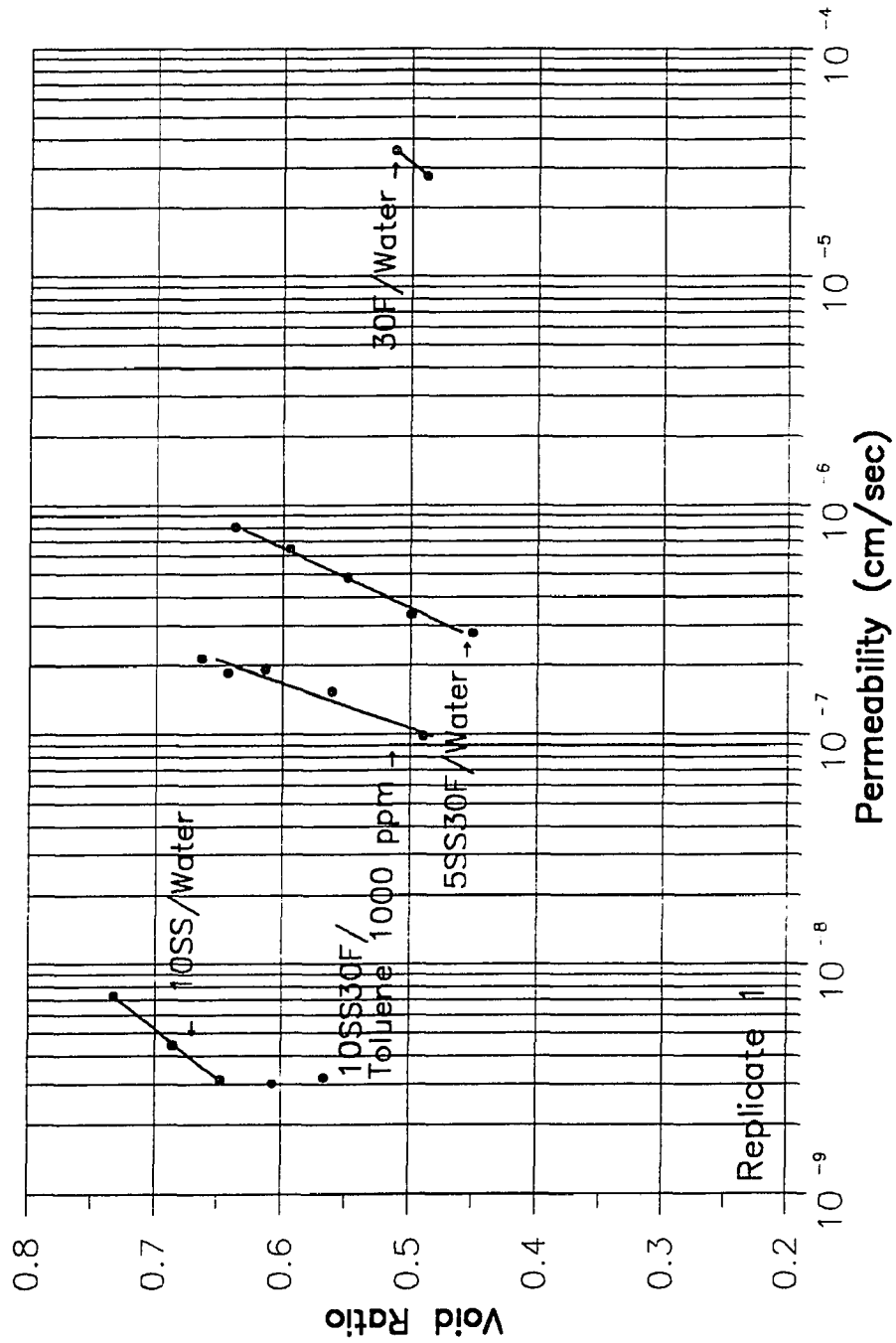


Figure 4-11A Effect of Fly Ash on the Permeability Tested in a Consolidometer Permeameter (Replicate 1).

PERMEABILITY TEST

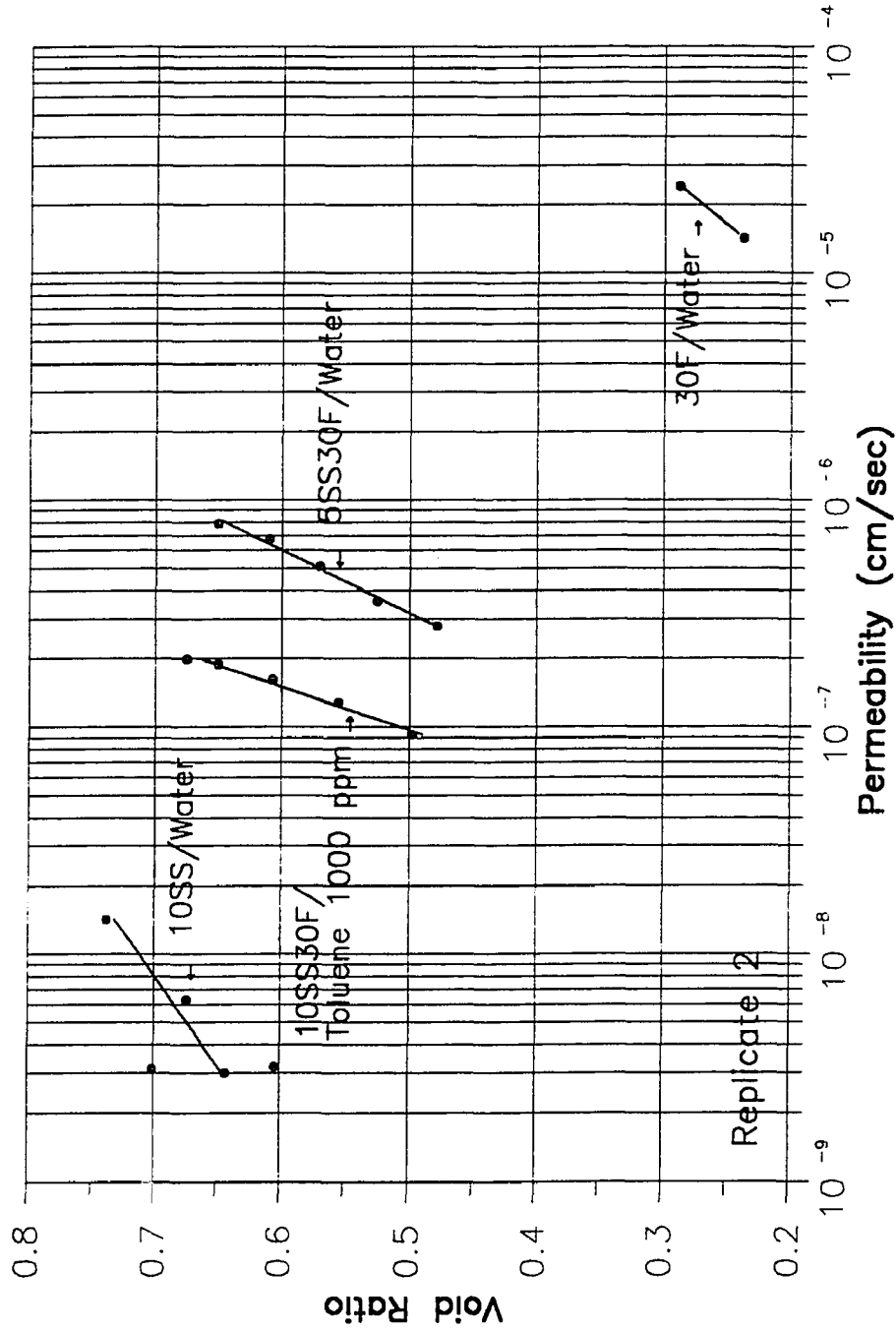


Figure 4-11B Effect of Fly Ash Content on the Permeability Tested in a Consolidometer Permeameter (Replicate 2).

PERMEABILITY TEST

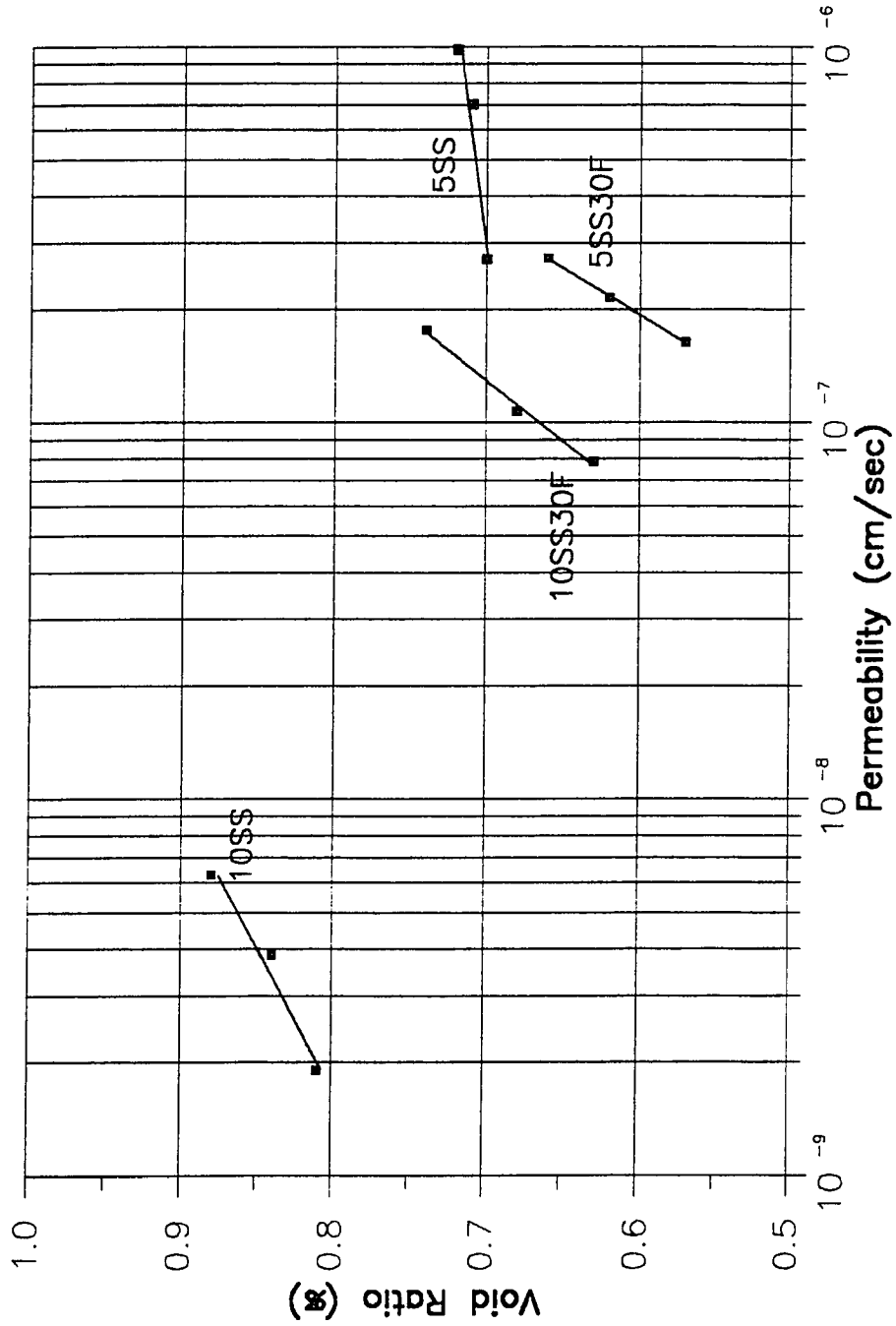


Figure 4-11C Effect of Fly Ash and Bentonite Content on the Permeability Tested in a Flexible Wall Permeameter.

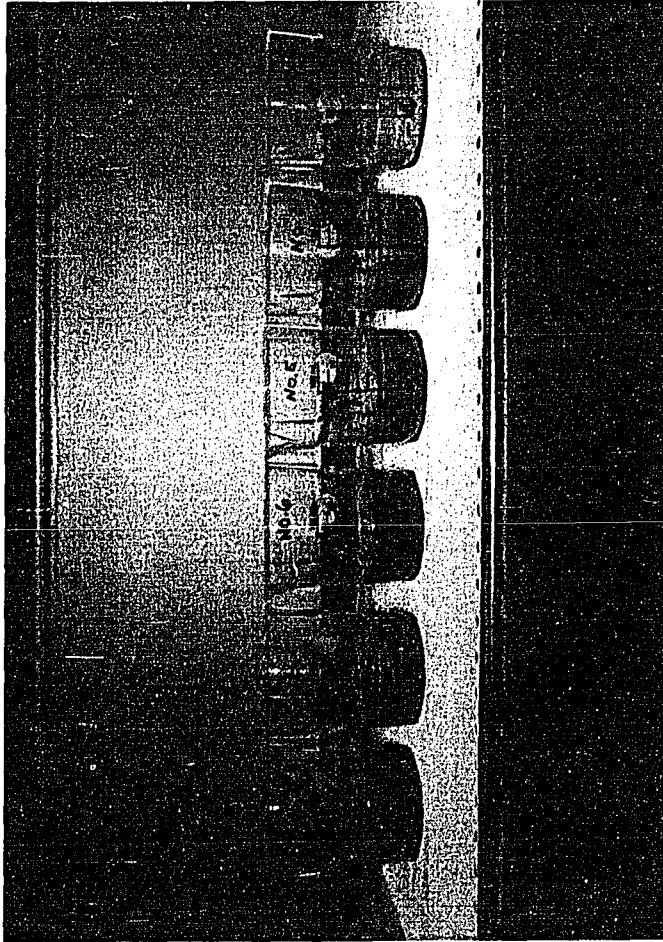


Figure 4-12 Sedimentation of Fly Ash Samples in Water. The Fly Ash Content for each of the sample is (from left to right) 90%, 60%, 40%, 30%, 20%, and 0%.

also appears less expansive in fly ash mix. It seems that these two materials are not compatible. As a result, the fly ash samples appear to be relatively loose in comparison with the 10SS sample. The more fly ash, the looser the mix volume.

The pH values for the suspensions range from 7.8 for 10SS to 8.7 for 10SSF, the sample which contained 10 percent bentonite and 90 percent fly ash. The differences are small and the slight alkaline solution should favor the expansion of the bentonite. Therefore, other factors must be present that degrade the bentonite.

Fly ash contains large quantities of silica (SiO_2), alumina (Al_2O_3), ferric oxide (Fe_2O_3), free lime (CaO), and minor amounts of various other oxides and alkalies. The most important chemical constituent of fly ash from an engineering viewpoint is probably free lime. It will influence the chemical reactivity of the ash.

Fly ashes with large amounts of free lime tend to be very reactive and can exhibit some degree of self-hardening. The principal ions contained in a fly ash leachate are calcium and sulfate with smaller quantities of magnesium, sodium, and potassium (DiGioia et al., 1979). Hulet et al. (1981) reported that very little material tends to leach from the fly ash itself. However, because of the high surface to volume ratio, there is much agglomerated material on the surface. This material consists of easily exchangeable or adsorbed molecules which, in the presence of a liquid, become dissolved.

To verify the presence of other ions, conductivity tests were performed on the supernatant of all the samples. The test results indicate a conductivity value of 375 μmho for the 10SS sample and 450 μmho for fly ash. For the bentonite and fly ash mixture, the initial value was found to be 625 μmho . However, this value increased with time. Finally, a value of greater than 1,000 μmho (beyond the instrument's limit of 999 μmho) was found for all the fly ash mixtures. The higher the conductivity, the higher the electrolyte concentration. Therefore, this could considerably reduce the double layer thickness of the bentonite. The reduction also could be caused by the replacement of the original sodium ions of the bentonite crystals.

Mitchell (1976) stated that the bentonite crystals that are saturated with calcium or other multivalent ions have a smaller thickness of the double layer. This is thought to occur because the larger multivalent ions are held more strongly to the clay, thus the effective net negative charge on each clay particle is lowered. The size of the double layer surrounding each clay particle is reduced.

EPA (1984) compared the physical properties of sodium and calcium-saturated bentonites. The results indicate that sodium bentonite exhibits swelling far more than calcium bentonite. A 5 percent suspension of hydrated sodium bentonite has a much higher viscosity than a 5 percent calcium bentonite. Permeability tests on mixtures consisting of the same proportions of quartz and clay, but different

types of bentonite, indicated the calcium bentonite samples to have two orders of magnitude greater permeability than the sodium bentonite samples.

The observed increase in the permeability of the fly ash samples could be the result of a combined effect of ion replacements and higher electrolyte concentration within the soil matrix. However, there can be a large variability in the fly ashes as a result of the nature of the coal burned and power plant operational characteristics (Bowders et al., 1987). It is not appropriate to conclude that all fly ash materials are detrimental to the bentonite backfill mix. Vesperman et al. (1985) found significantly different permeabilities for two self-hardening fly ashes. Edil et al. (1987) conducted permeability tests on compacted mixes of fly ash, sand, and bentonite. They reported that the addition of ten percent bentonite did not influence the permeability of their sample significantly.

4.2.5 Effect of Cracking on Permeability

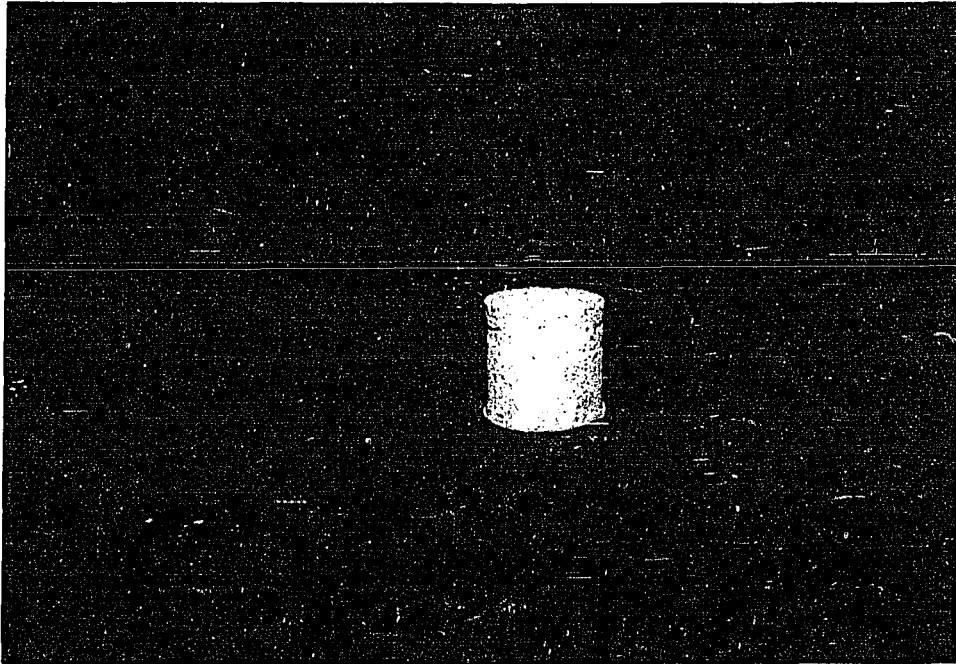
The desiccation effects on the slurry backfill mix were studied. Seasonal changes may result in considerable fluctuations in ground water level and the amount of available leachate. This will result in the desiccation of the backfill material where the ground water level fluctuates. Cracking may develop as a result of the shrinkage and increase permeability considerably. Uppot (1984) commented that secondary structures such as cracks and joints, if present and open, will yield the major part of the

flow. Daniel (1984) found field permeability to be 10 to 1000 times greater than the laboratory values for a clay liner. He concluded that the increase was caused by the desiccation cracks and fissures.

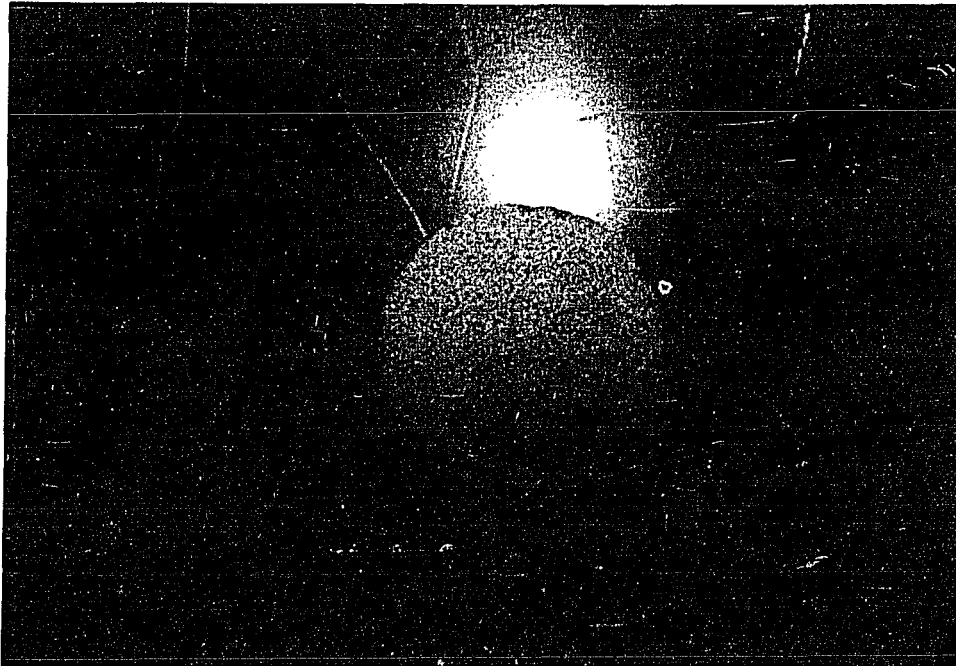
At the completion of a regular permeability measurement, the 10CS sample was carefully placed into an oven and dried at a temperature of 60°C. When the sample was essentially dry, it appeared to be monolithic, and only a few hairline fissures were observed (Figure 4-13). The dry sample was then subjected to permeability measurements. During saturation, the specimen absorbed an appreciable amount of water. Its volume expanded to almost the same as before drying.

As shown in Figure 4-14, the permeability of the dry sample was found to be two-to-three-fold larger than before drying. The increase could be attributed to fissures and other microscopic changes in the soil structure. The dry-wet effect also altered the pattern of e-logk relationship.

Because uniform drying and high temperature hardly occur in the field, a second dry-wet observation was made with a condition more close to that of the site. Two 10SS samples were prepared with phenol 60,000 ppm. One was immediately subjected to permeability measurements. The other remained in the sample ring and was allowed to dry at room temperature. When the surface of the specimen was dry, liquid was filled in again. This procedure was repeated for three cycles over four months. At the end of the treatment, the sample was extruded from the ring.



(a) Side View



(b) Top View

Figure 4-13 Photographs of a 10CS Sample Subjected to One Cycle Oven-dry and Wet Treatment with Water.

PERMEABILITY TEST

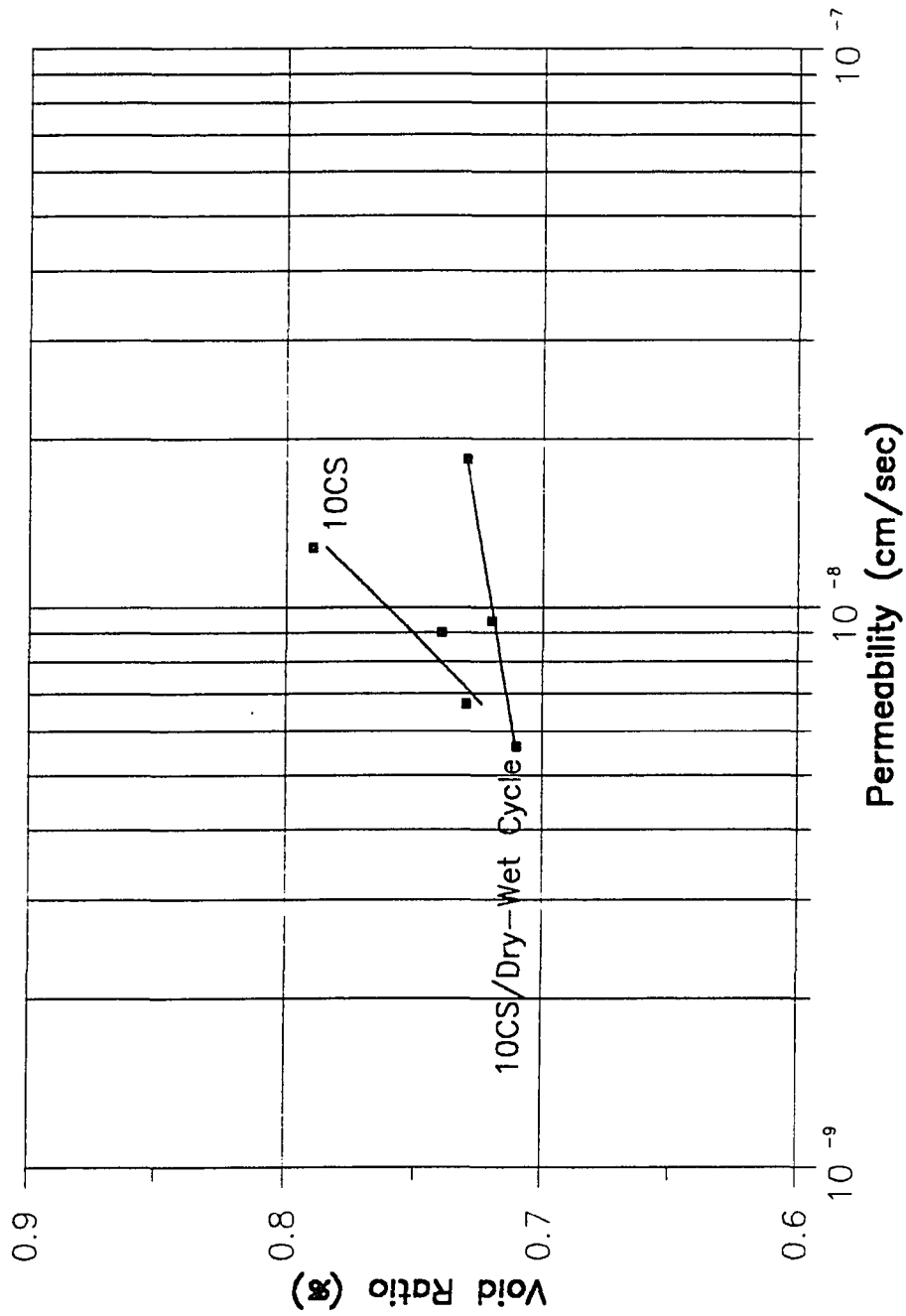
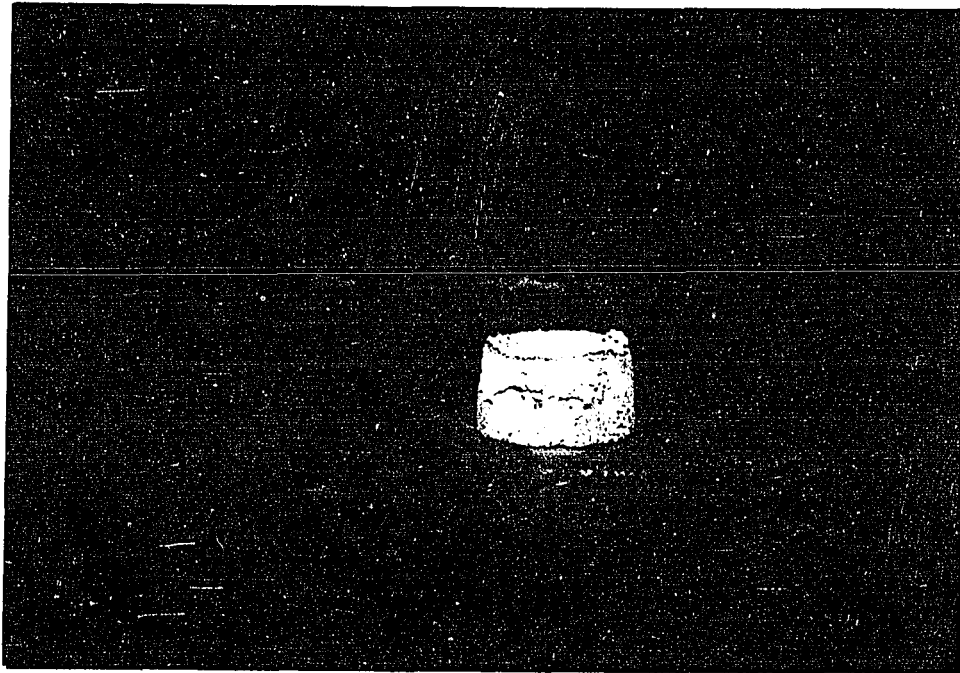


Figure 4-14 Effect of Cracking, from Oven-drying on the Permeability for a 10CS Sample with Water in a Flexible Wall Permeameter.

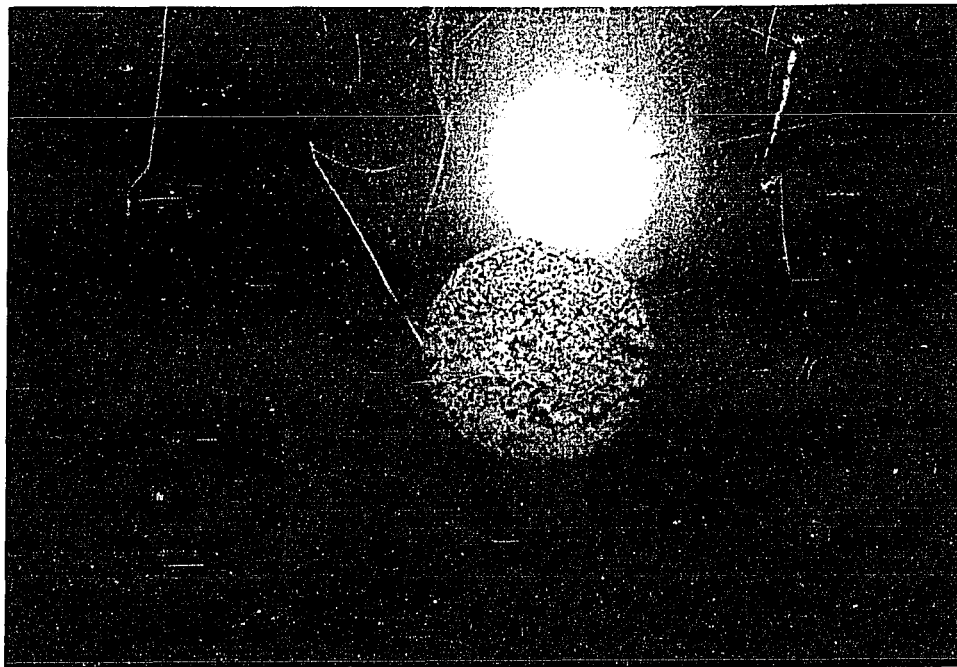
As shown in Figure 4-15, cracks appeared throughout the upper half of the sample with significant cracking at the interface of the dry and wet portion. The shape of the sample also became irregular. The quality of the sample under such dry-wet effect appeared to be greatly degraded.

Figures 4-16A through 4-16C show the permeability values of the cracked samples tested with phenol 60,000 ppm. The permeability values are almost identical to those obtained at normal condition. The dry-wet cycles showed no detrimental effect on permeability. This would be an anomaly as cracks and fissures should increase permeability. The sample was examined after the test. Most of the hairline fissures disappeared on the sample surface. The circumferential crack still existed but the opening was reduced somewhat. The self-healing of the bentonite apparently recovered some of the fissures. The circumferential crack showed no influence on permeability. This is because it was perpendicular to the flow direction and it also was tightly sealed by the membrane in the horizontal direction. In addition, the applied confining stresses had apparently closed the cracks in the specimen and significantly reduced its permeability.

Boynton (1983) studied the influence of effective stress on permeability. Samples with cracks and without cracks were tested. The test results indicated that the permeability of the desiccated samples decreased considerably when the effective stress was over 28 kPa. He concluded that high effective confining stress tends to close cracks that may be



(a) Side View



(b) Top View

Figure 4-15 Photographs of a 10SS Sample Subjected to Three Cycles Cool-dry and Wet Treatments with Phenol 60,000 ppm.

PERMEABILITY TEST

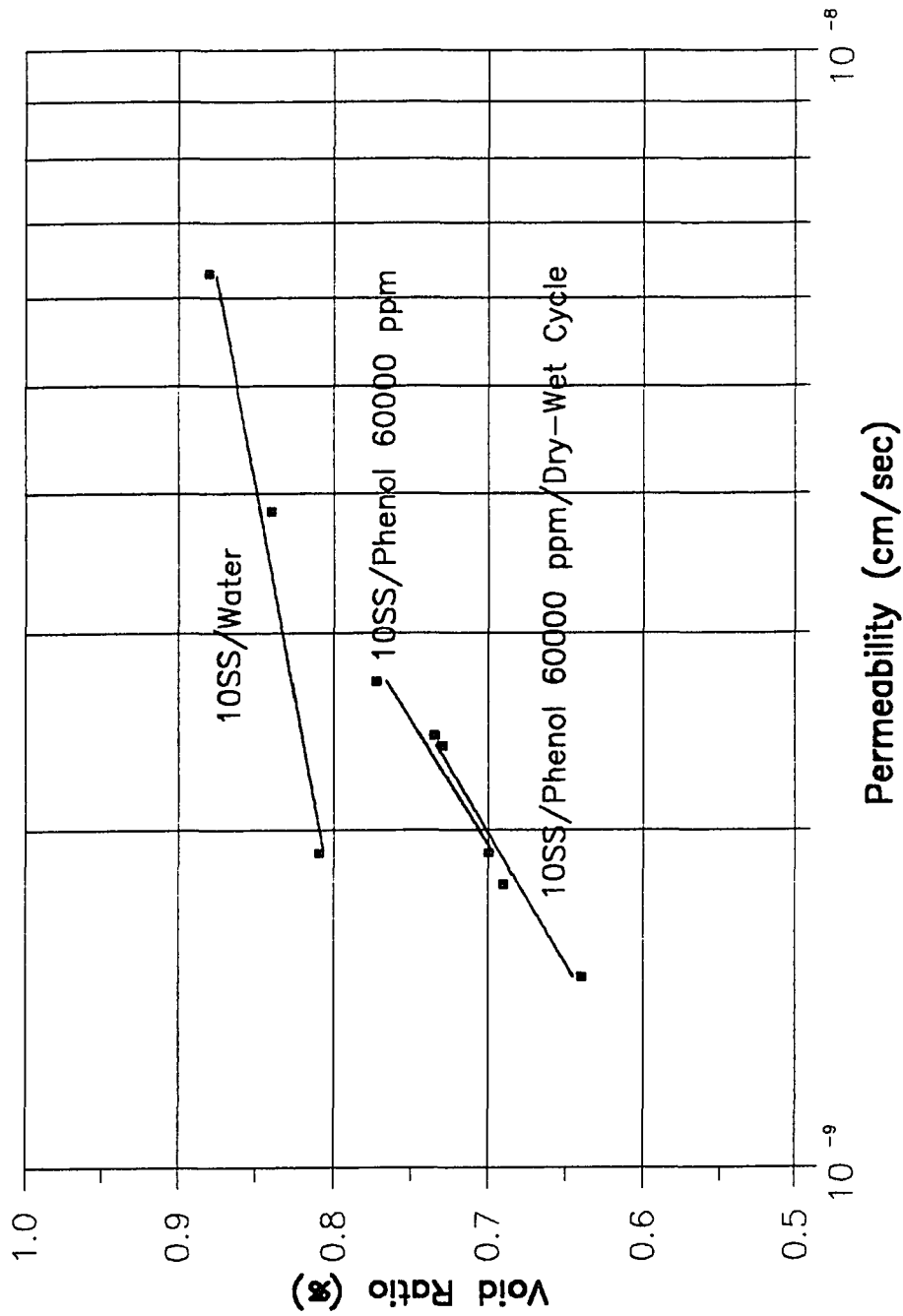


Figure 4-16A Effect of Cracking from Drying on the Permeability for a 10SS Sample with Phenol 60,000 ppm in a Flexible Wall Permeameter.

PERMEABILITY TEST

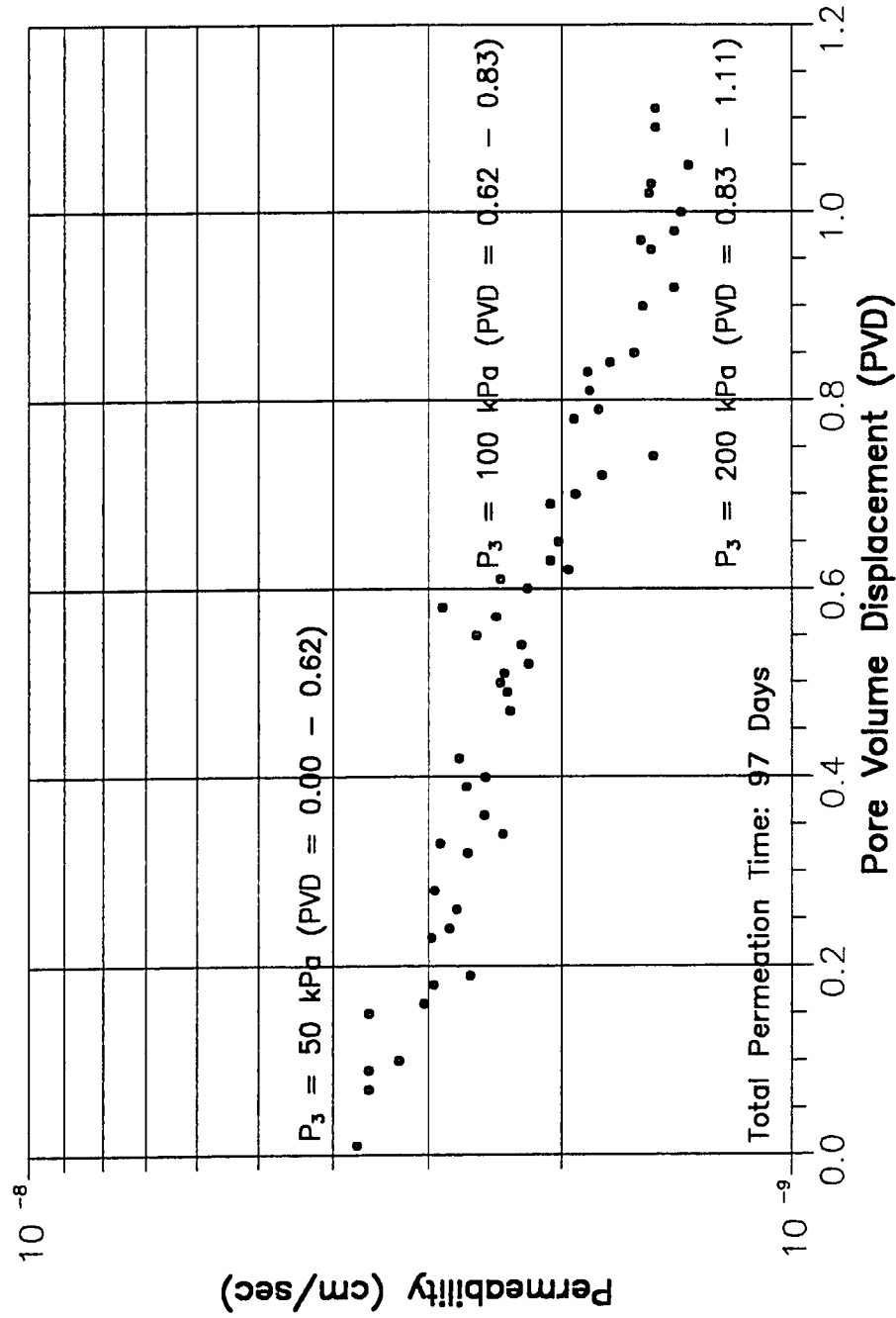


Figure 4-16B Permeability versus Pore Volume Displacement for a Cracked 10SS Sample with Phenol 60,000 ppm in a Flexible Wall Permeameter.

PERMEABILITY TEST

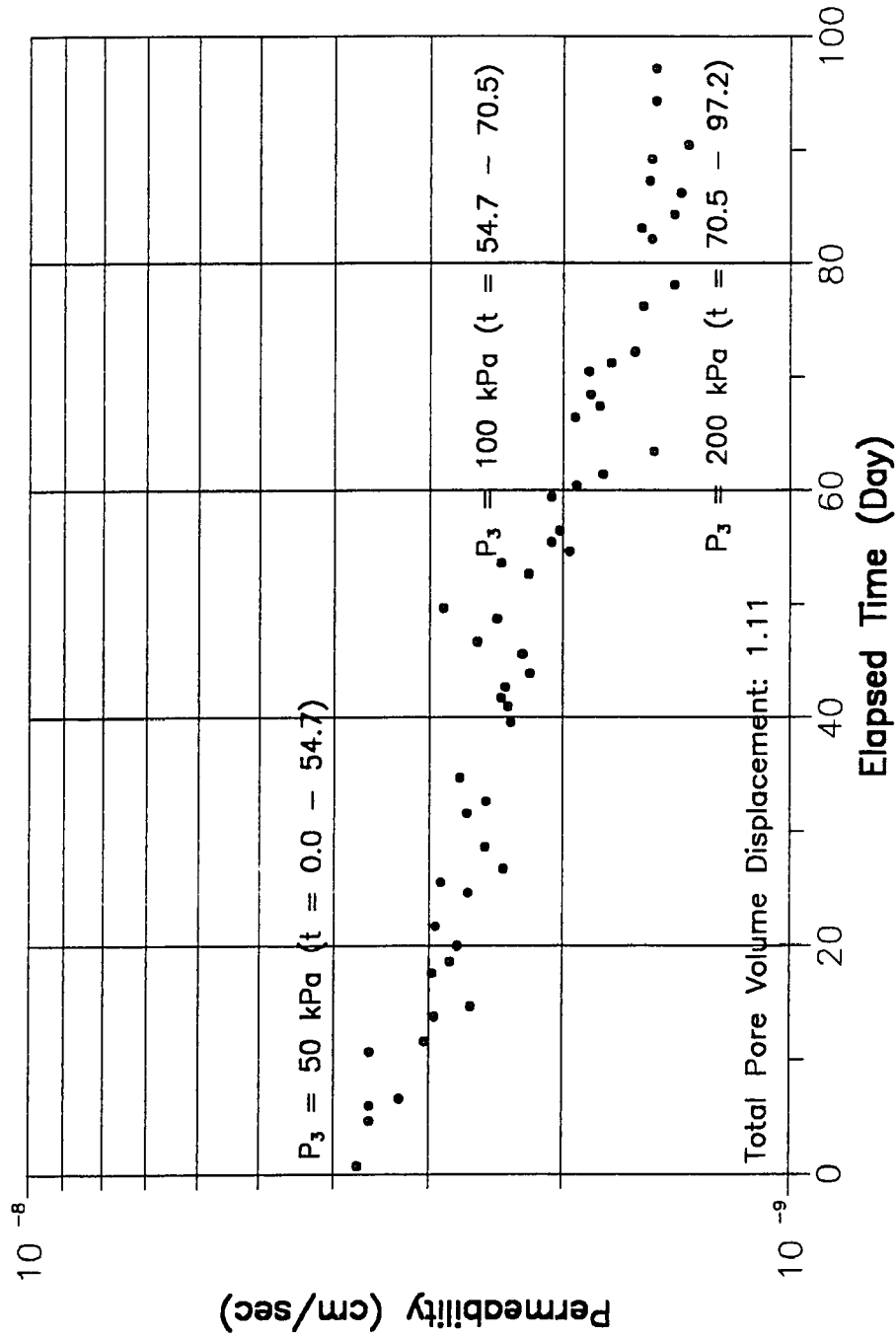


Figure 4-16C Permeability versus Permeation Time for a Cracked 10SS Sample with Phenol 60,000 ppm in a Flexible Wall Permeameter.

in the specimen. Carpenter (1982) also recommended that the hydraulic gradient used with FP should create an effective stress at the downstream end of the sample not greater than the preconsolidation pressure. The consolidation pressures used in this study were over 50 kPa. As evident from test results, the effect of soil cracking was completely eliminated by high stresses.

4.2.6 Comparisons of Test Results for the Consolidometer Permeameter and the Flexible Wall Permeameter

A comparison of the permeability values for the flexible wall permeameter (FP) and the consolidometer permeameter (CP) shows that the permeability values of FP are less than that of CP. The difference is about one-third for fly ash samples and about one order of magnitude for 10SS samples. This could be attributed to the effect of different methodology in applying the hydraulic gradient and the consolidation between these apparatus.

The hydraulic gradients for FP testing ranged from 75 to 100 while, for CP tests, the falling head technique generally generated a gradient of about 50. Carpenter (1982) found permeability to be essentially independent of the hydraulic gradient for samples confined in rigid rings whereas, for triaxial samples, the permeability decreased as the hydraulic gradient increased. He attributed these results to the increase in the effective stresses at the downstream end of the sample from the hydraulic gradient. As the effective stress increases, the void ratio decreases resulting in

smaller flow channels. Dunn and Mitchell (1984) also reported permeability decreases with increasing gradient. They concluded that clogging of pore spaces resulting from the migration of soil particles was the most probable reason.

In a consolidometer, specimen consolidates under a K_0 condition whereas, in a flexible wall permeameter, specimen consolidates almost isotropically. Therefore, specimen tends to have more consolidation effect in a flexible wall permeameter. The greater the consolidation effect, the smaller the permeability.

Tests with FP thus may estimate a lower permeability and are not conservative from the design point of view.

4.2.7 Comparisons of Measured and Computed Permeability

The permeability from the falling head method, k_m , and from the coefficient of consolidation, k_{cv} , were compared. Figure 4-17 shows a plot of k_m and k_{cv} for all the samples tested with the consolidometer permeameter. The distribution of the data indicates that for 10SS30F and 5SS30F samples, k_m and k_{cv} fairly correlated each other. For 10SS specimens, this tendency was found only for those having a void ratio at 0.60 or higher. k_m and k_{cv} measured at a void ratio less than 0.60 are considerably scattered and show no specific correlations.

k_m and k_{cv} were further analyzed with statistical correlation. The results indicate that Pearson's coefficient, $r(k_m, k_{cv})$, is 0.4425 and 0.8391 for 10SS30F and 5SS30F samples, respectively. The t-test results indicate that for

PERMEABILITY TEST

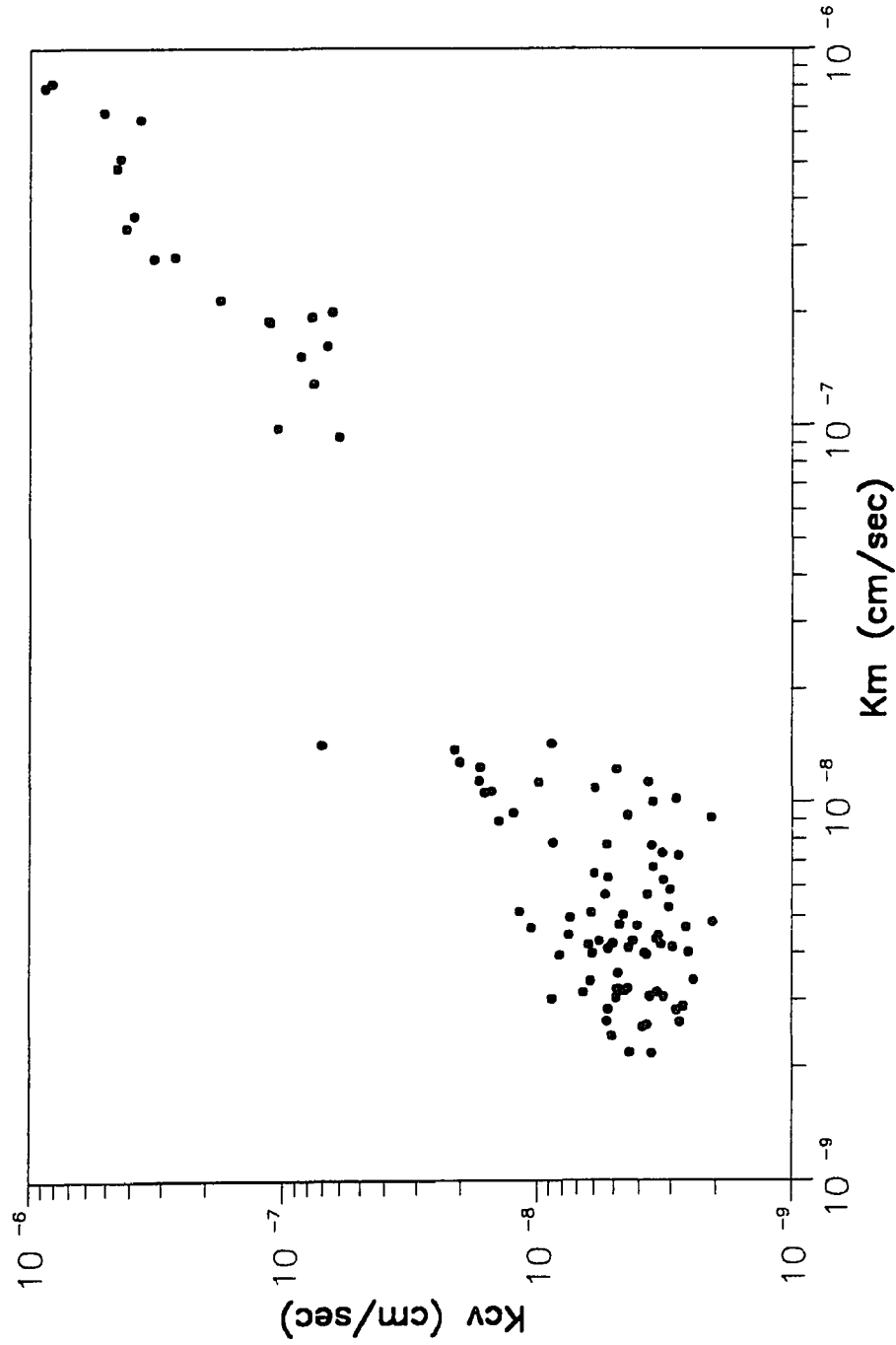


Figure 4-17 Comparison of Direct and Indirect Measurements of Permeability in a Consolidometer Permeameter.

10SS30F specimens, k_m and k_{cv} showed correlation in a 80 percent confidence interval. For 5SS30F specimens, k_m and k_{cv} correlated within a 99 percent confidence interval. k_m and k_{cv} , therefore, tend to be medium-to-strongly correlated for fly ash samples. The correlation decreases with the increasing bentonite content. For 10SS samples, $r(k_m, k_{cv})$ is equal to 0.236 for a void ratio at 0.60 or higher and -0.0426 for a void ratio less than 0.60. The t-test results indicate that at a void ratio of 0.60 or higher, k_m and k_{cv} for 10SS specimens showed a correlation in 90 percent confidence. When the void ratio is less than 0.60, k_m and k_{cv} have no correlation at all. The Spearman rank tests showed similar results.

Olson and Daniel (1981) reported that the results of comparisons for k_m and k_{cv} are controversial. This may be attributed to the fact that the coefficient of consolidation varies widely with the soil properties and its consolidation behavior. The larger differences observed for 10SS samples, therefore, may be due to the fact that these soils are predominantly coarse grained. For such soils, the coefficient of consolidation may not be determined reliably. The soil mixtures with fly ash contained appreciable amounts of fines and, therefore, allowed proper determination of the coefficient of consolidation values.

4.2.8 Effect of Consolidation on Permeability

Figure 4-18 is a plot of all the consolidometer permeability values with respect to the consolidation

PERMEABILITY TEST

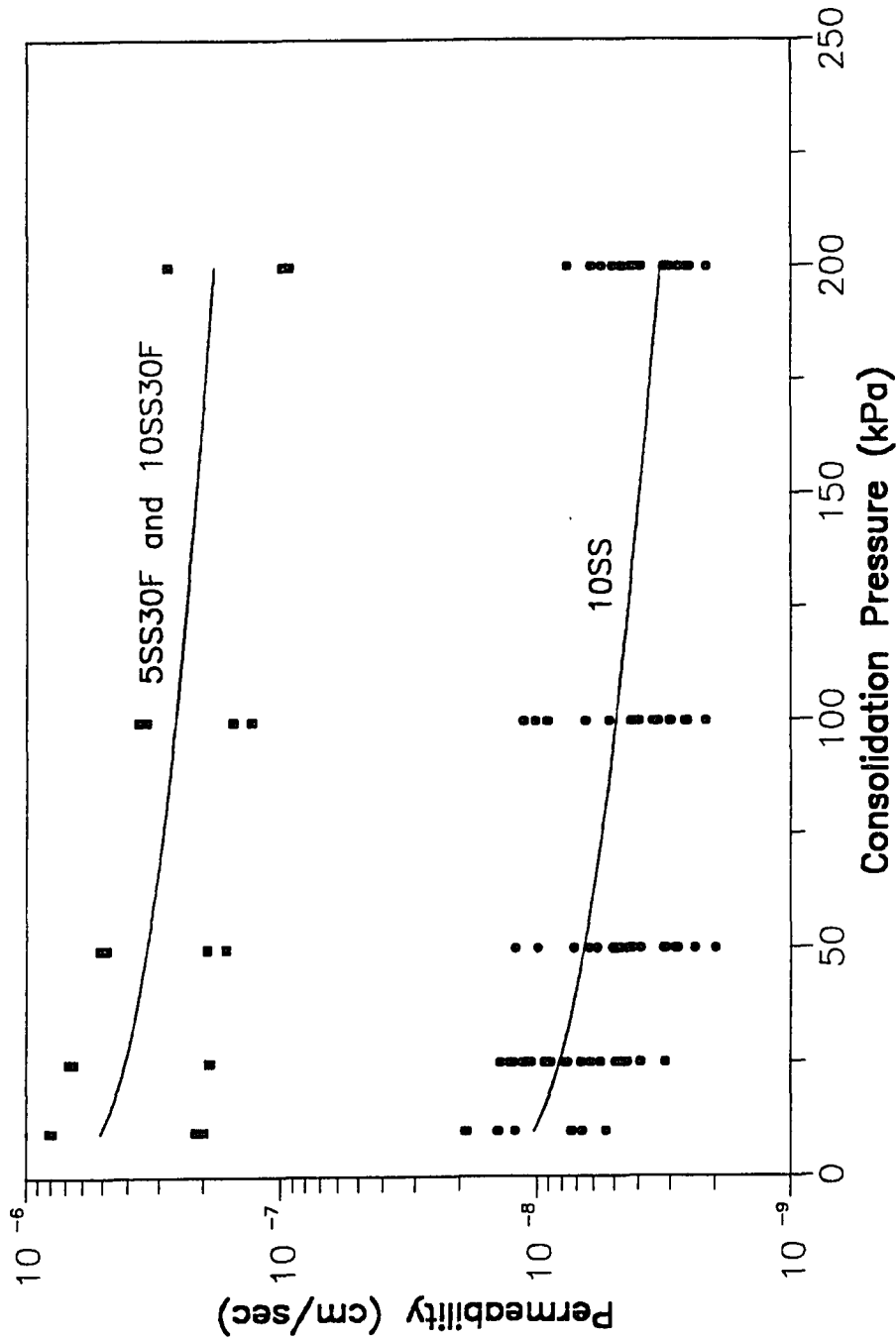


Figure 4-18 Effect of Consolidation Pressure on Permeability Tested in a Consolidometer Permeameter.

pressure, P . The correlation results indicate that, for 10SS specimens, the $r(P, k_m)$ is -0.3822 for a void ratio at 0.60 or higher and -0.3001 for a void ratio less than 0.60 . The t -test results indicate that the k_m is strongly correlated with P in a 99 percent confidence interval for specimens that have a void ratio greater than 0.60 . However, for specimens at a void ratio less than 0.60 , this correlation only exists at a 80 percent confidence interval.

For fly ash samples, $r(P, k_m)$ ranges from -0.8892 for 5SS30F samples to -0.9646 for 10SS30F samples. The t -test results indicate that k_m shows a strong correlation with P in a 99 percent confidence interval. The Spearman rank tests showed similar behavior.

As shown in Figure 4-18, permeability decreased with an increase in consolidation pressure. The decrease occurred relatively rapidly for stresses less than 25 kPa. This corresponds to an overburden pressure of about one to two meters (three to six feet) of soil.

The effect of the consolidation pressure on permeability has been noted earlier. Acar et al. (1985) reported similar results. The application of high consolidation stress may reduce permeability and, therefore, may be not conservative for the design. However, in the field, consolidation pressure may exist because of overburden. A typical example would be a landfill liner with 6 to 10 meters (20 to 30 feet) of overlying solid waste. Daniel and Liljestr nd (1984) studied the importance of consolidation pressure on permeability for

this condition. Their results indicated three orders of magnitude decrease in the measured permeability when the effective stress was changed by an equivalent weight of a few meters of waste. They concluded that overburden pressure does have a great effect on permeability. D'Appolonia (1980) noted a stress range of 50 to 200 kPa in many field loading situations.

A bentonite slurry wall would be used at a hazardous waste site to impede the movement of the leachate which could contaminate ground water. It is not commonly designed to sustain any loads. The vertical pressure most commonly applied to the wall is from the compacted clayey cap. This is equivalent to a pressure of about 25 kPa. The wall may also be consolidated under its own weight. However, this would take considerable time to complete due to its impervious nature. Therefore, the magnitude of consolidation pressure in the permeability measurement for the slurry wall should be small.

The compressibility of the bentonite-sand mix was presented by following the conventional e -log p relationship (Figure 4-19A through 4-19L). The void ratio generally varied linearly with the consolidation pressure. Regression was used to facilitate the calculation of the compression index, C_c . The results are summarized in Table 4-5.

For 10SS samples, C_c varied from 0.11 with water to 0.15 with 10,000ppm phenol. These values are far less than those observed for common clayey soils. Thus these mixtures can be

CONSOLIDATION TEST

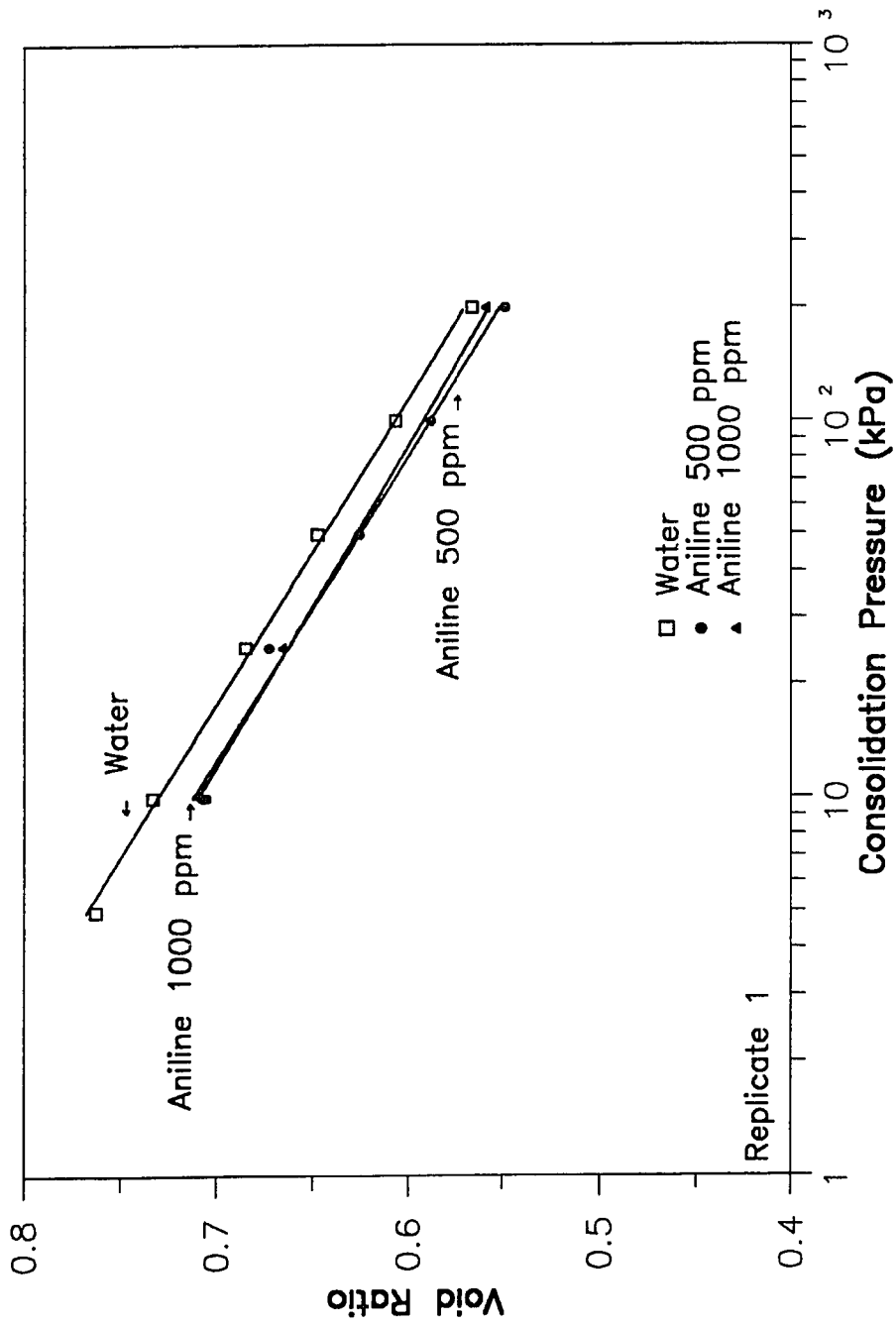


Figure 4-19A Consolidation Behavior of a 10SS Sample with Aniline (Replicate 1).

CONSOLIDATION TEST

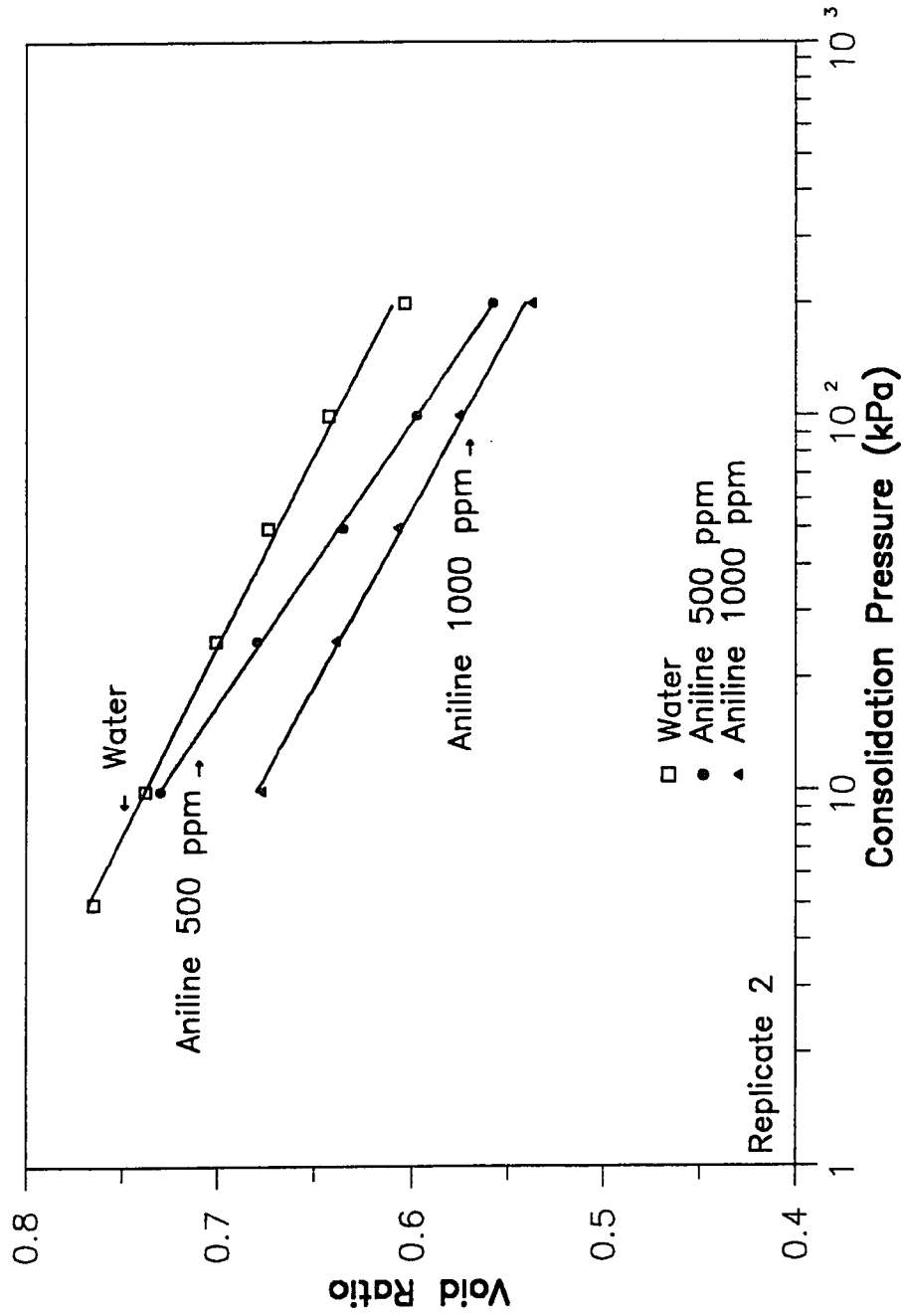


Figure 4-19B Consolidation Behavior of a 10SS Sample with Aniline (Replicate 2).

CONSOLIDATION TEST

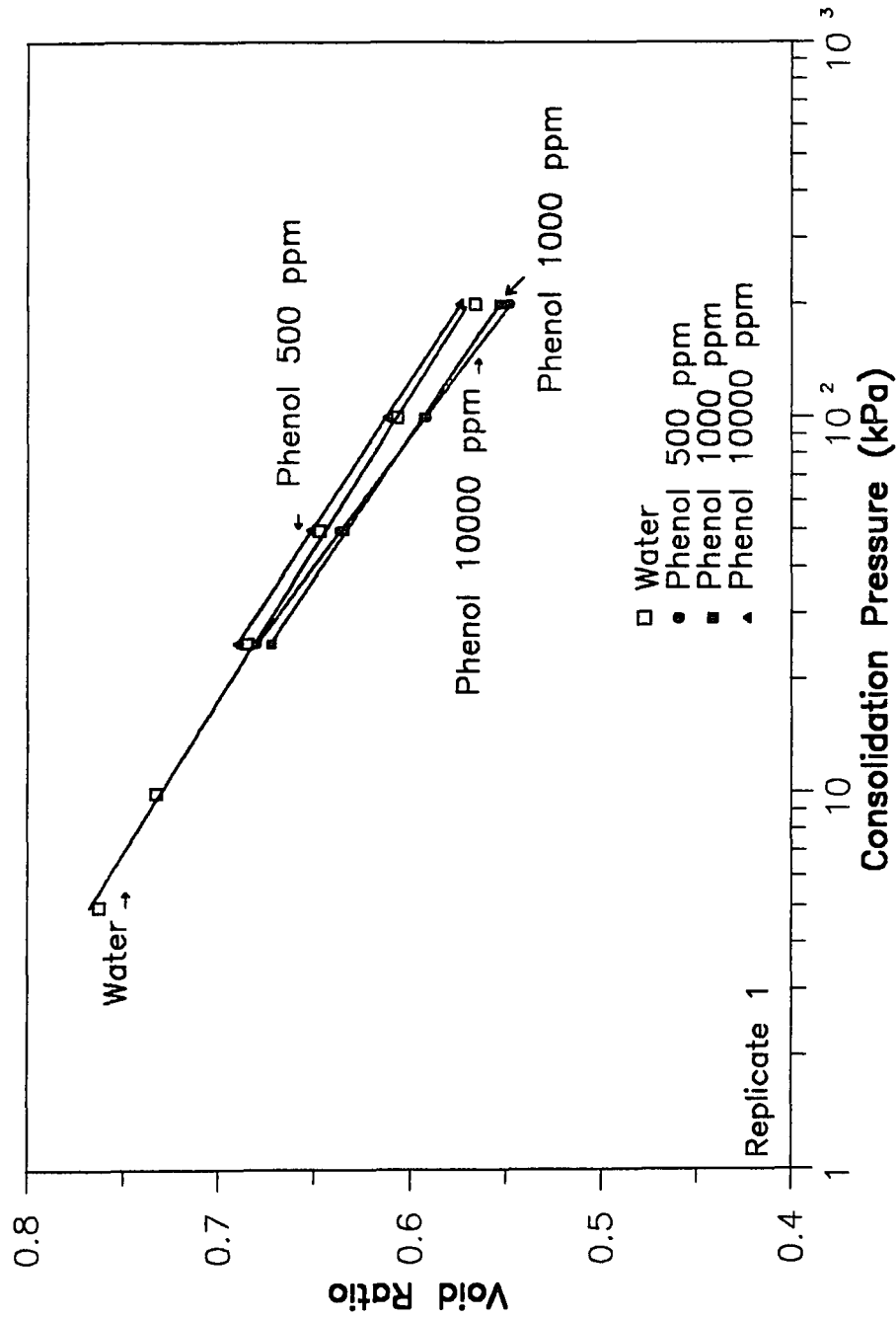


Figure 4-19C Consolidation Behavior of a 10SS Sample with Phenol (Replicate 1).

CONSOLIDATION TEST

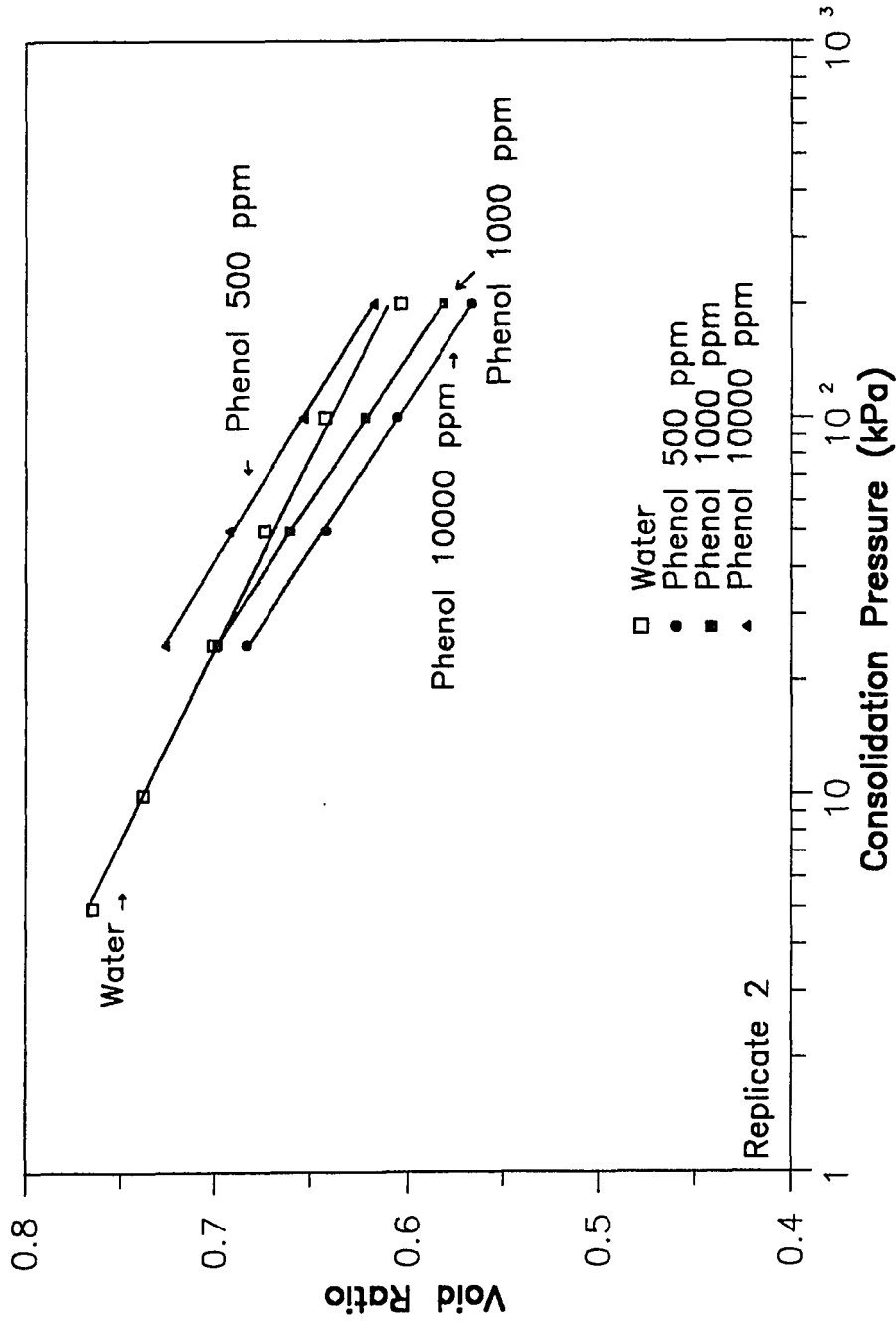


Figure 4-19D Consolidation Behavior of a 10SS Sample with Phenol (Replicate 2).

CONSOLIDATION TEST

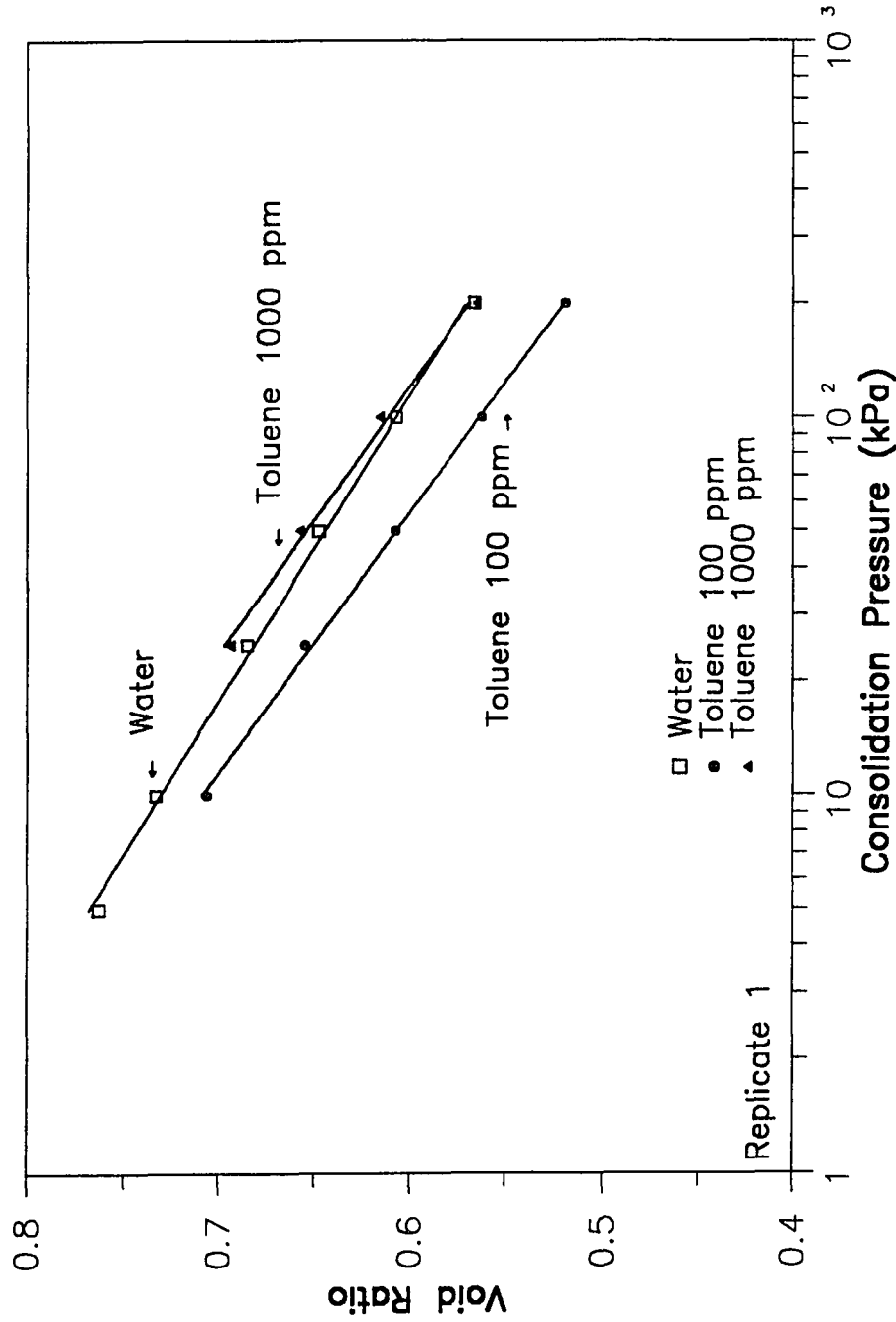


Figure 4-19E Consolidation Behavior of a 10SS Sample with Toluene (Replicate 1).

CONSOLIDATION TEST

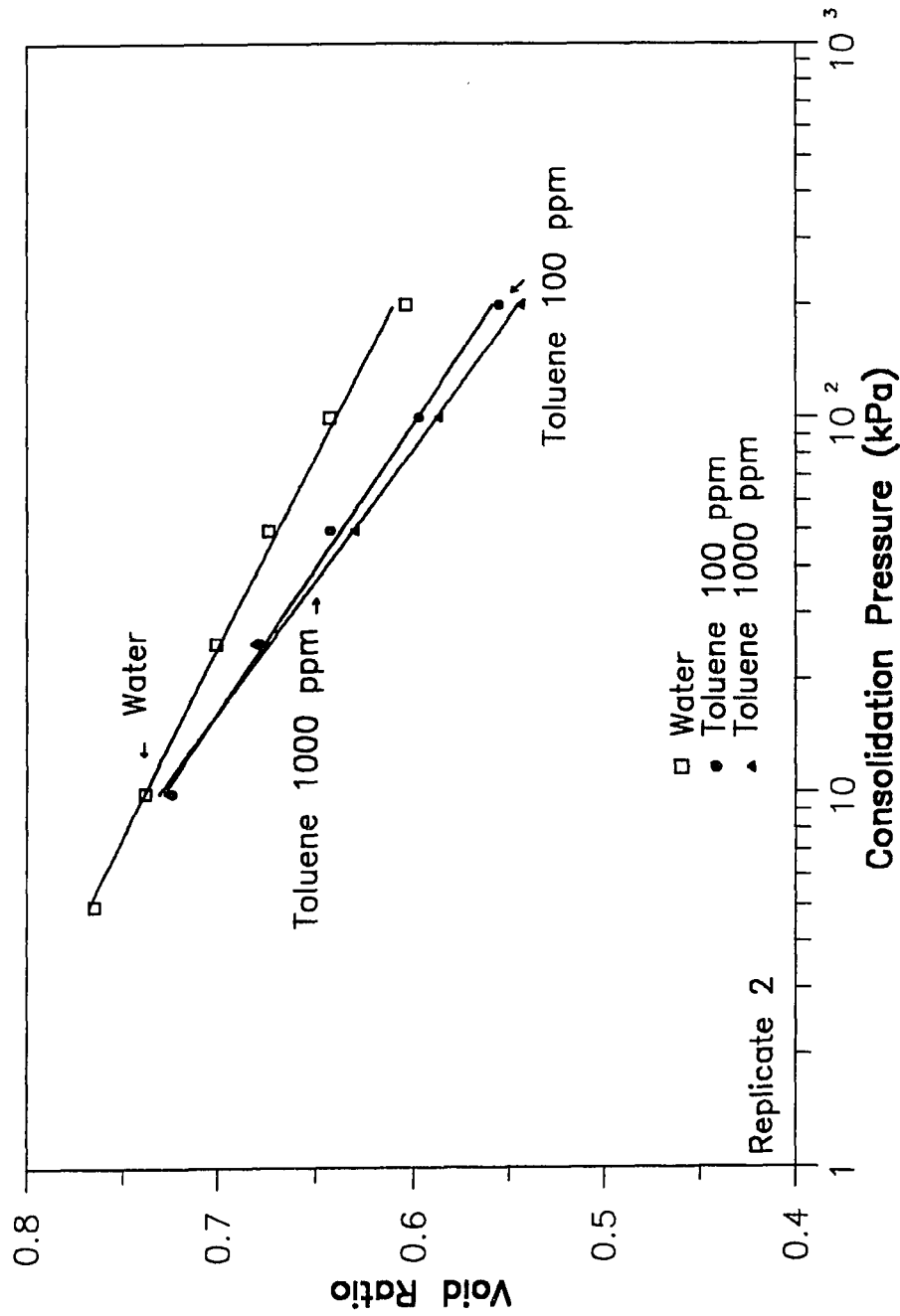


Figure 4-19F Consolidation Behavior of a 10SS Sample with Toluene (Replicate 2).

CONSOLIDATION TEST

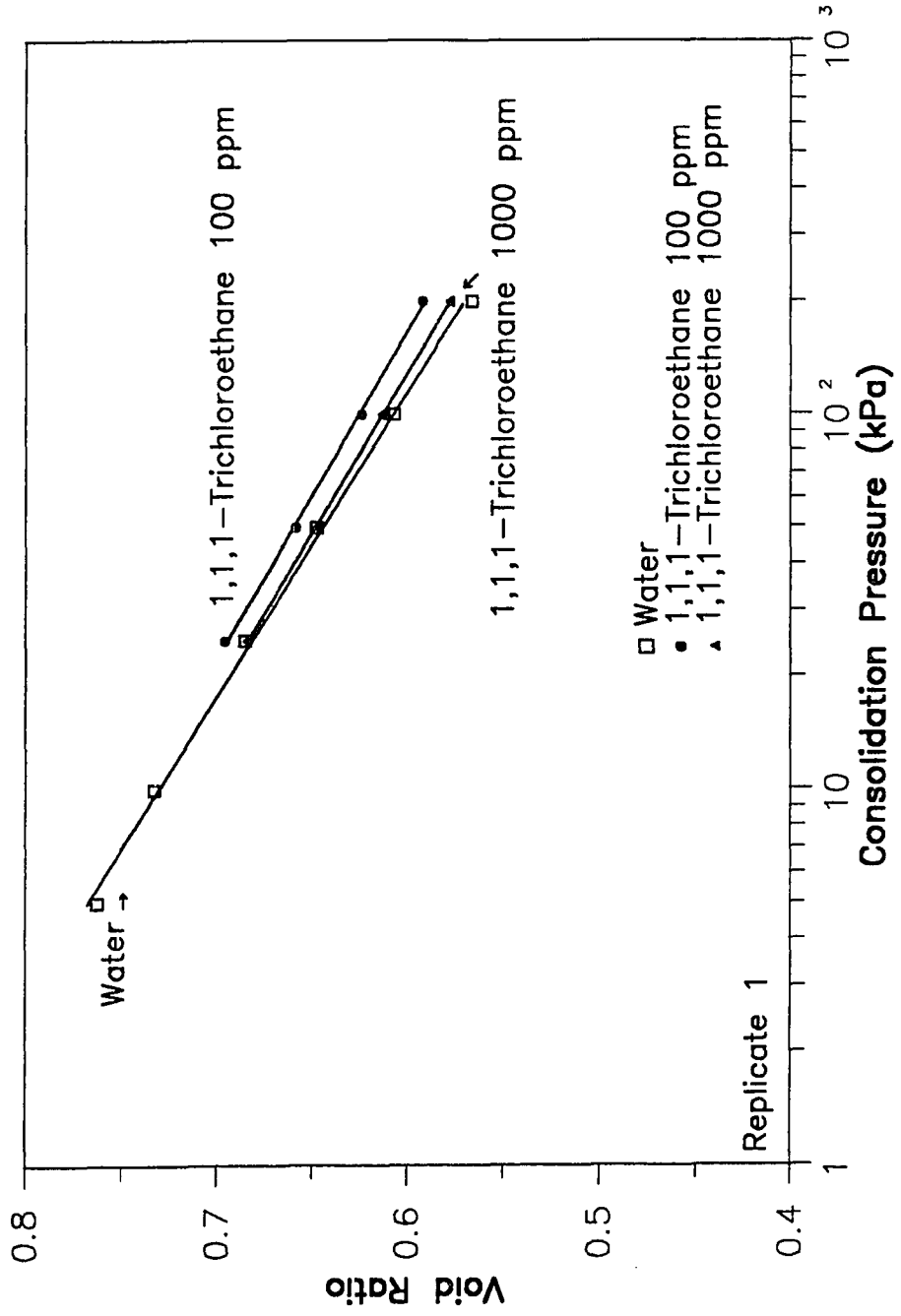


Figure 4-19G Consolidation Behavior of a 10SS Sample with 1,1,1-Trichloroethane (Replicate 1).

CONSOLIDATION TEST

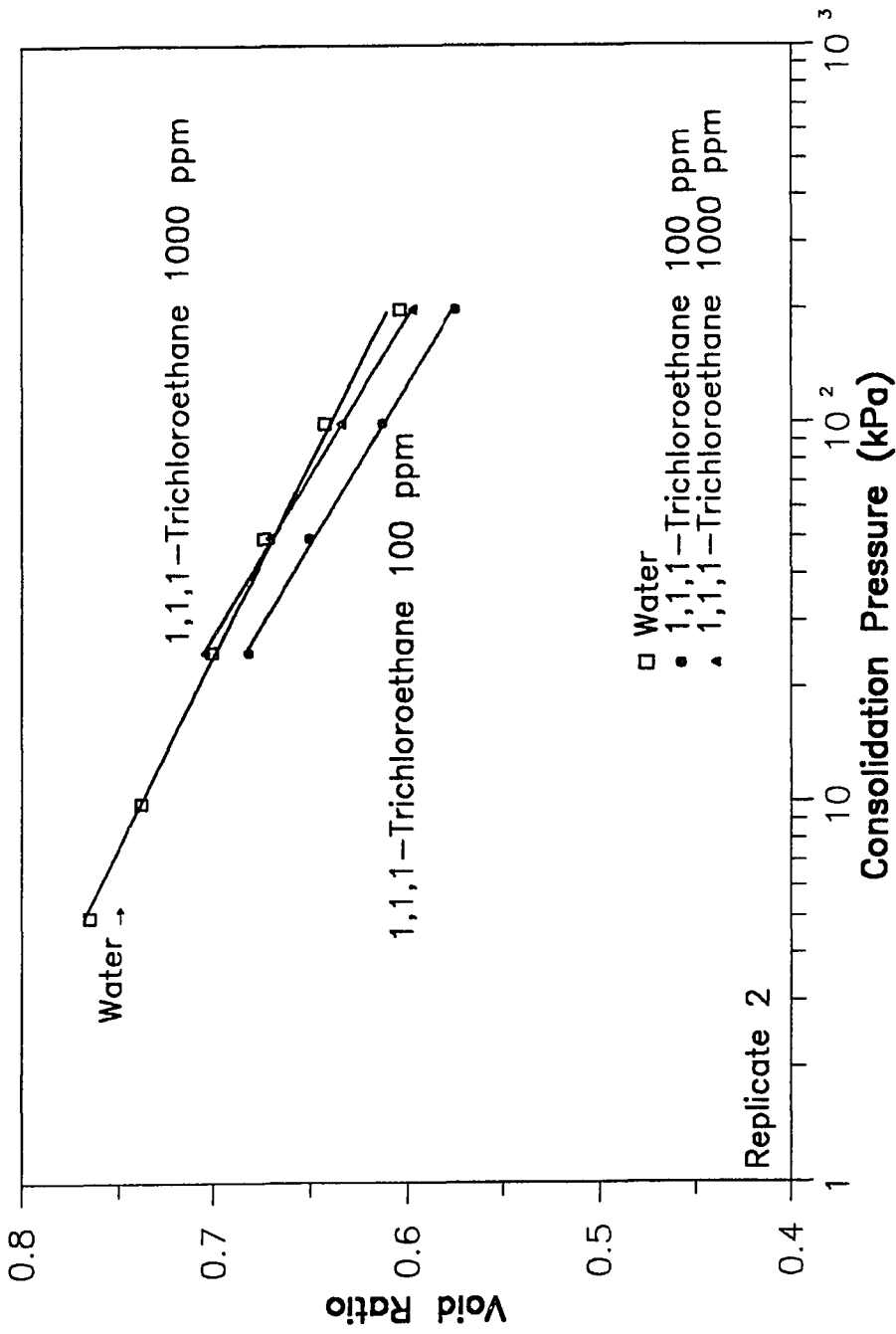


Figure 4-19H Consolidation Behavior of a 10SS Sample with 1,1,1-Trichloroethane (Replicate 2).

CONSOLIDATION TEST

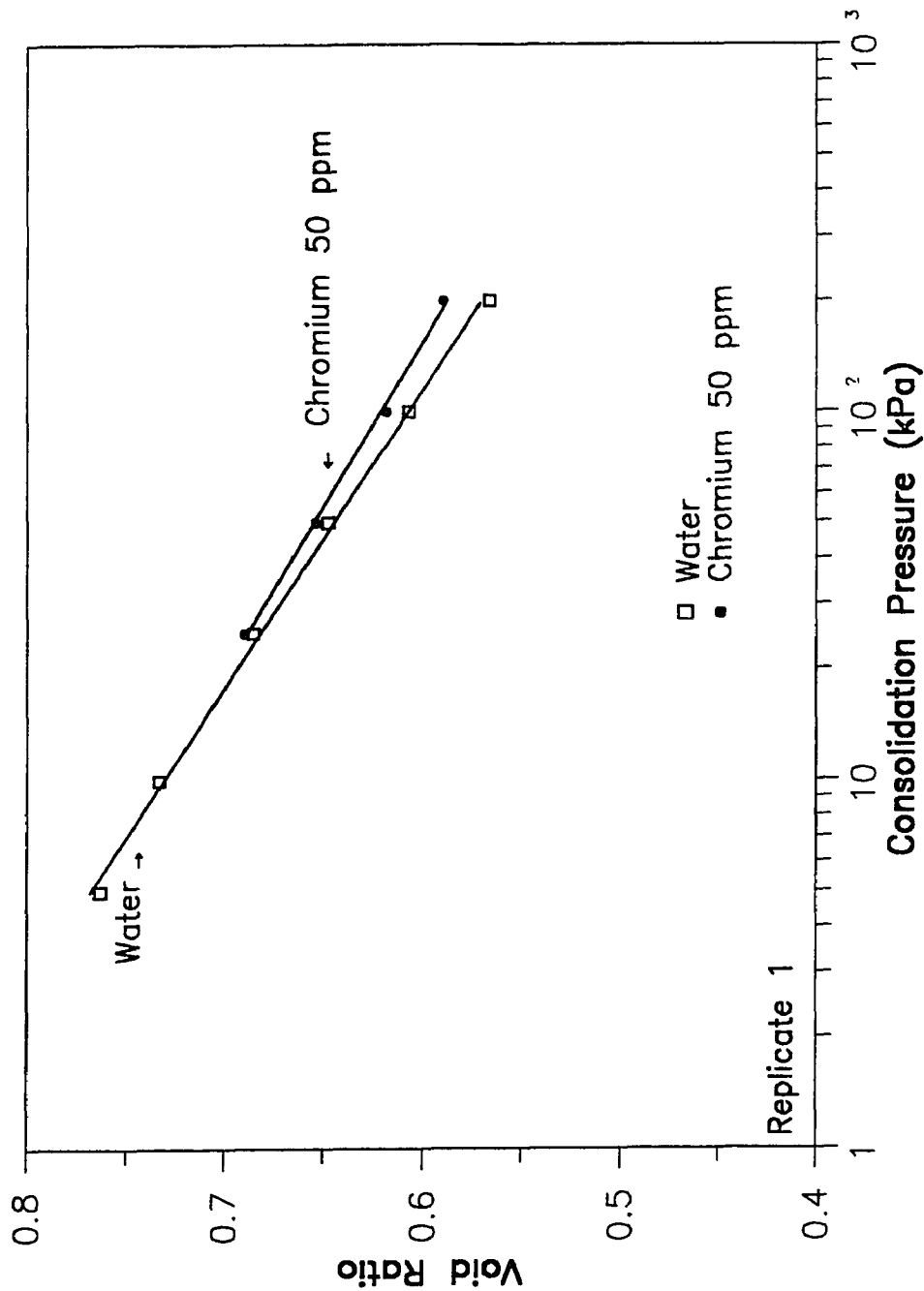


Figure 4-19I Consolidation Behavior of a 10SS Sample with Chromium (Replicate 1).

CONSOLIDATION TEST

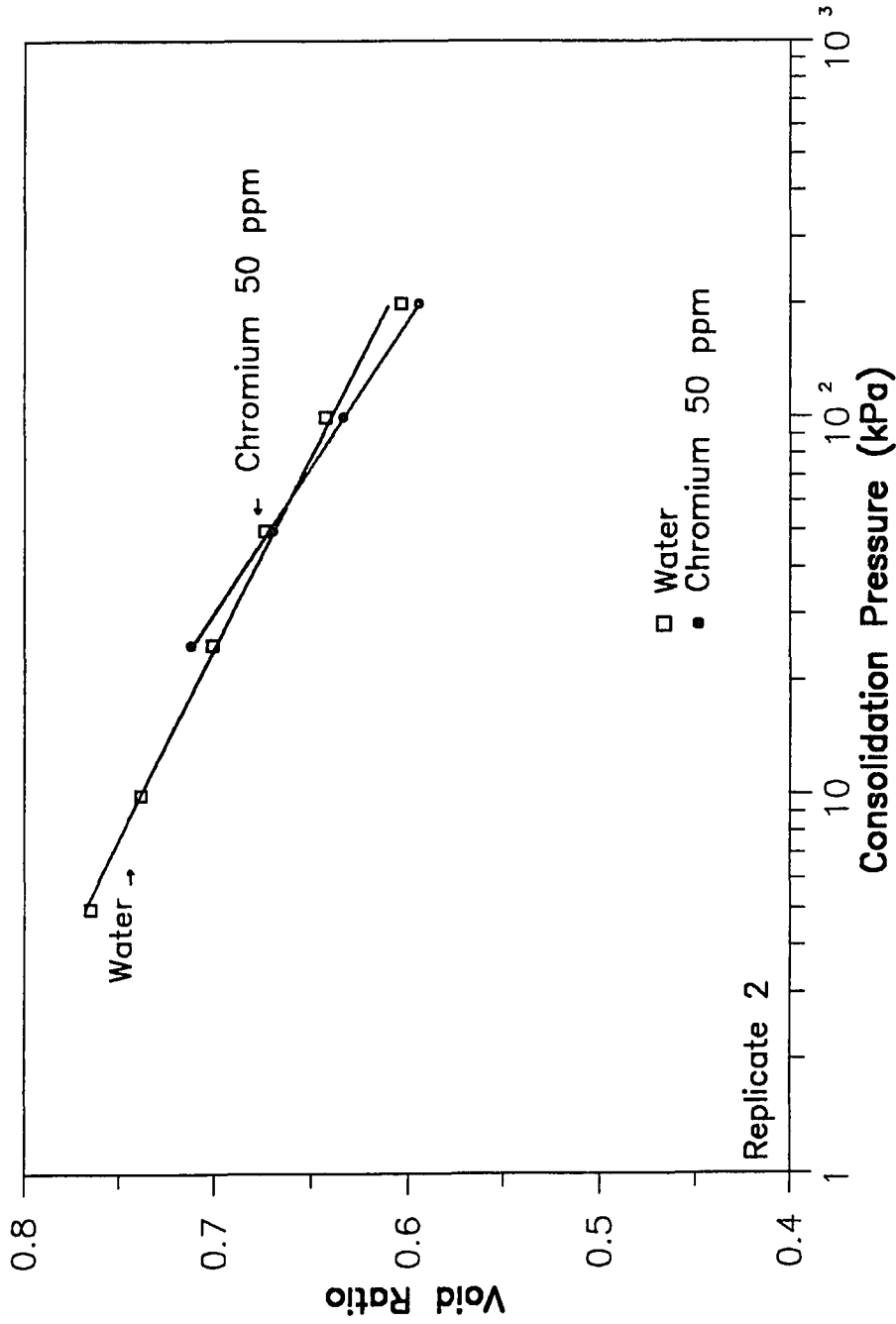


Figure 4-19J Consolidation Behavior of a 10SS Sample with Chromium (Replicate 2).

CONSOLIDATION TEST

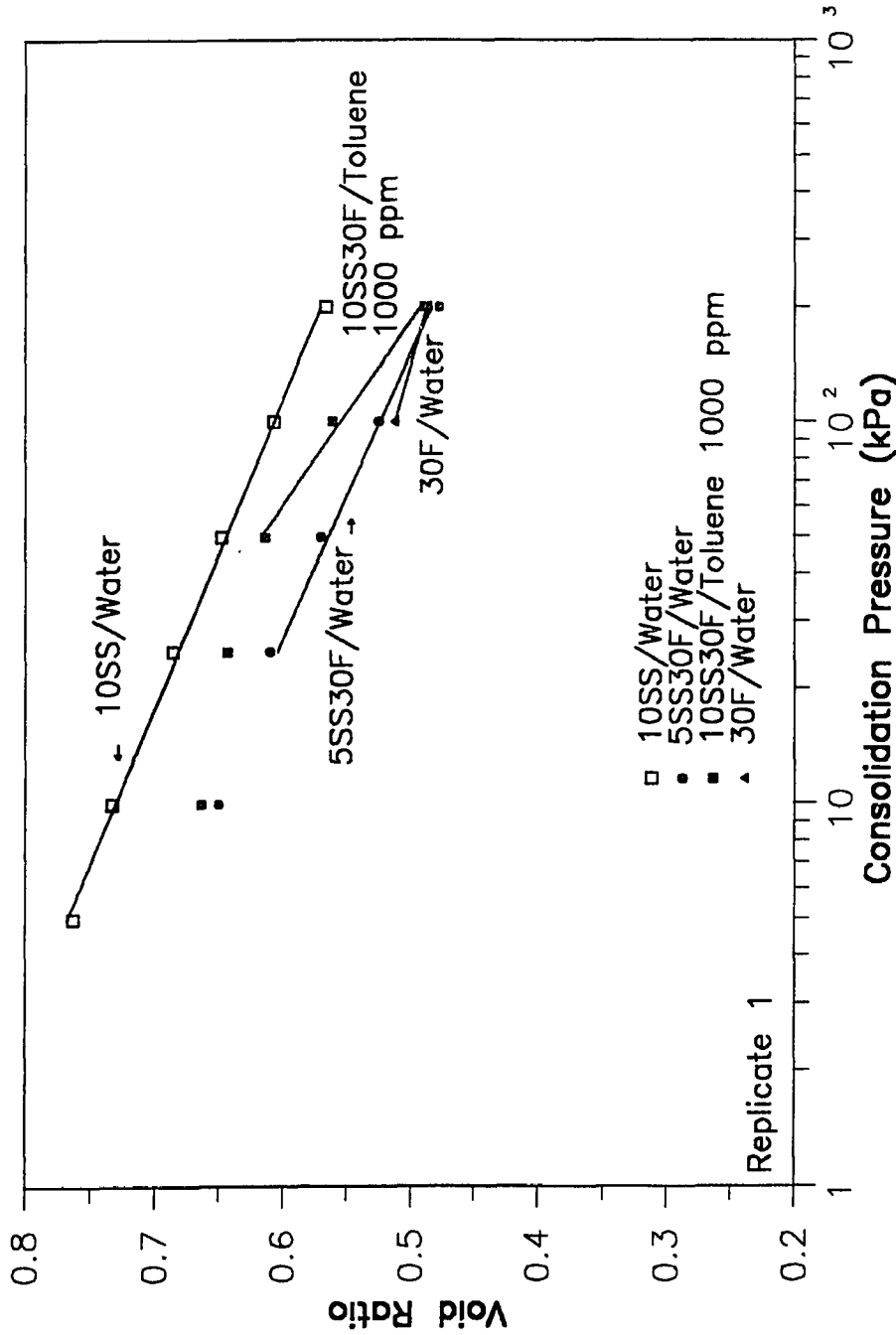


Figure 4-19K Consolidation Behavior of Soil Mixtures with Various Proportions of Bentonite, Fly Ash, and Sand (Replicate 1).

CONSOLIDATION TEST

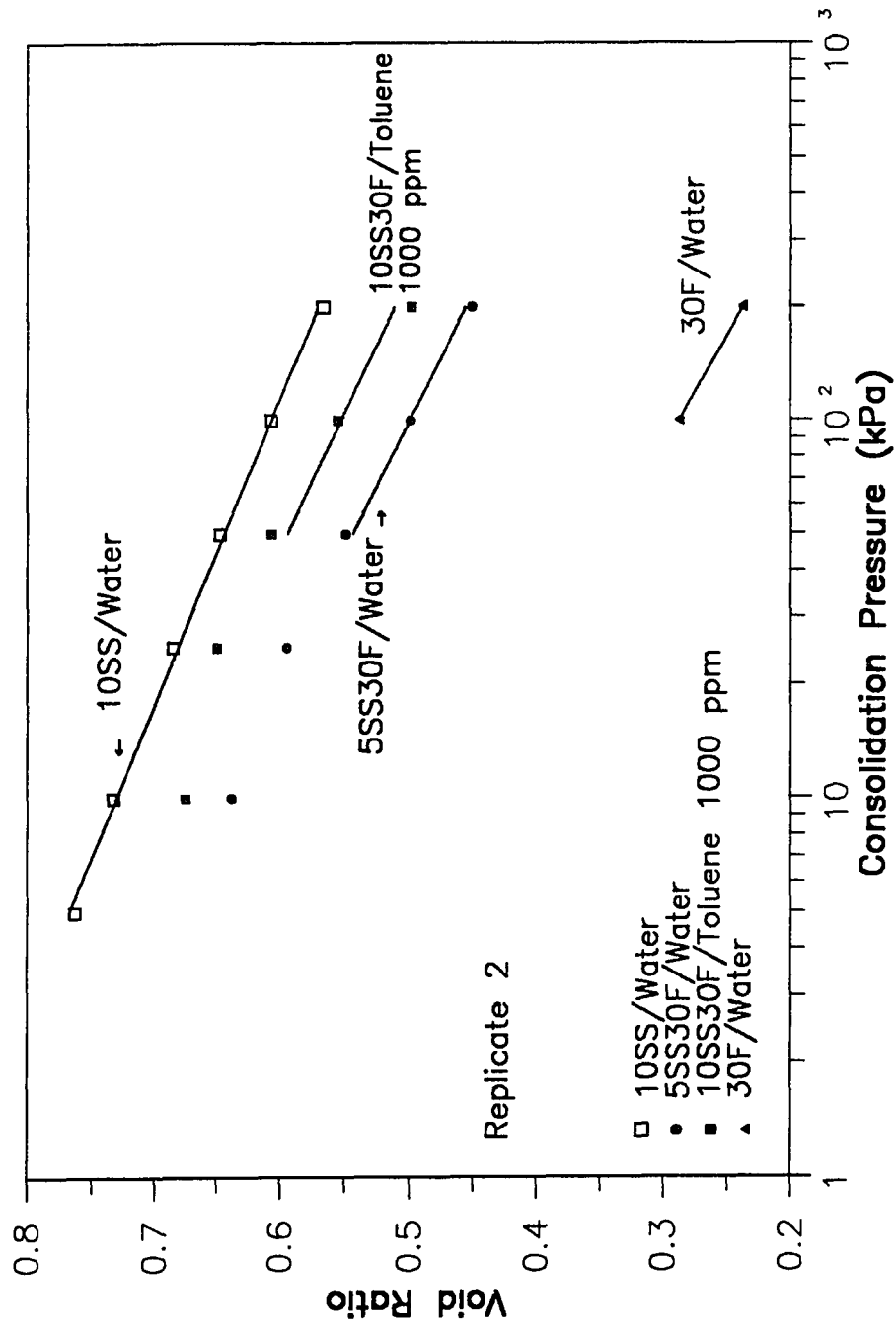


Figure 4-19L Consolidation Behavior of Soil Mixtures with Various Proportions of Bentonite, Fly Ash, and Sand (Replicate 2).

considered to be of low compressibility (Holtz and Kovacs, 1981). In general, C_c increases with increasing chemical concentration. The variations are within 40 percent. The liquids appear to soften the soil matrix and cause an increase in C_c . However, for C_c with such a low value, the compressibility of the specimen is not likely to be further degraded.

Table 4-5 Summary of Compression Index

Type of Soil	Type of Permeant	Concentration (ppm)	Compression Index
10SS	Water	-	0.12
	Aniline	500	0.13
		1000	0.12
	Phenol	500	0.13
		1000	0.13
		10000	0.15
	Toluene	100	0.14
		1000	0.15
	1,1,1-Trichloroethane	100	0.13
		1000	0.13
Chromium	50	0.13	
10SS30F	Toluene	1000	0.14
5SS30F	Water	-	0.14
30F	Water	-	0.09

Fly ash samples showed about a 30 percent increase in C_c as compared with 10SS sample. The higher values are associated with the higher fines contents. The fly ash

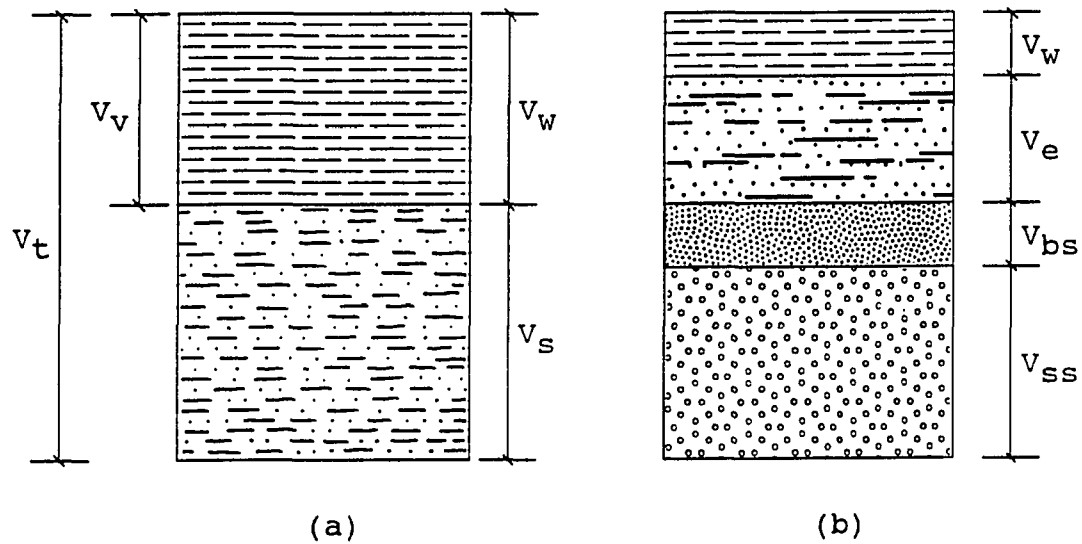
particles are predominantly silt-sized. Therefore, the results are reasonable.

4.2.9 Concept of Expansion

During hydration of bentonite, water molecules are adsorbed by the clay crystals due to the attraction between the hydrogen atoms on the water molecules and the hydroxyles or oxygens on the clay particles. The adsorbed water is held so strongly by the clay that it may be considered as an immobile, semi-crystalline substance. Even the water molecules that do not directly contact the clay surface are influenced by the bentonite crystal and form what may be thought of as a semi-rigid structure (EPA, 1984).

The 10SS sample was prepared with an approximate liquid content of 30 percent. The free expansion of SS-100 in those permeants varied from 1,080 percent for water to 44 percent for sodium hydroxide 4 percent. The expanded volume is known to be generated by the adsorbed liquid. Though the bentonite in the soil mixture may expand quite differently from that in the free expansion condition, it is possible that most of the liquid in the soil matrix is adsorbed by the bentonite and behaves as a semi-rigid solid, thus reducing the permeability.

As shown in Figure 4-20(a), the concept of void ratio is based on the conventional two-phase diagram from a permeability viewpoint. Samples are normally to be saturated before any permeability values are measured. The volume of the voids will thus be occupied by liquid. However, the added



Legend:

- V_w = Volume of free liquid.
- V_e = Expanded volume of bentonite.
- V_{bs} = Volume of bentonite.
- V_{ss} = Volume of sand.
- V_s = Volume of solid.
- V_v = Volume of void.
- V_t = Total volume of the specimen.

Figure 4-20 Illustration of Phase Diagram for a Specimen subjected to Permeability Test: (a) Conventional Model and (b) Proposed Model.

liquid can not be considered as void completely for a bentonite mixture. Dependent upon the potential of the bentonite to expand, the amount of the liquid added, and the initial soil density, the volume of the void can be substantially reduced due to the adsorbing of liquid by the bentonite. Based on this hypothesis, a four-phase diagram is proposed as shown in Figure 4-20(b). Two expansion parameters are developed to further delineate the relationships among the phases.

$$\text{Expansion Ratio, } e_r = V_e / V_s \quad (4.3)$$

$$\text{Expansion Function, } e_f = V_e / V_v \quad (4.4)$$

where: V_e = expanded volume of bentonite;

V_v = apparent volume of void, sum of the volume of air, free liquid and semi-solid, if any;

V_s = volume of total solid.

The expanded volume is limited by the amount and the type of bentonite and liquid in a soil. The expansion ratio is an indicator of the expansion with respect to a particular soil matrix. The expansion function represents the amount of semi-solid in contrast to the apparent volume of the voids. Since permeability is a function of the volume of the void, e_f can show the effect of expanded volume on permeability.

Based on the above equations, values of expansion ratio and expansion function were calculated for samples used for the permeability tests and presented in Appendix A. For 10SS samples, the initial e_r and e_f are about 0.72 and 0.95, respectively. As the consolidation progresses, the soil

structure tends to change. The free liquid and possibly part of the adsorbed liquid may dissipate. It is difficult to differentiate between the amount of adsorbed liquid from the total reduced volume. Therefore, it is assumed that the initial volume changes include free liquid only. This is because it is relatively easy to displace. As the volume decreases, e_f increases and e_r remains constant. This tendency of change is valid until the soil has been consolidated to contain essentially particle solids and adsorbed liquid. At this point, e_f reaches its maximum value of 1.0 and e_r is approximately equal to the void ratio. Because solid particles and the semi-rigid adsorbed liquids are relatively impermeable, the permeability of the sample is reduced drastically. Further consolidation of the soil should displace more adsorbed liquids. However, permeability shows little change as a result of the reduction of the micropore volume.

Figure 4-21 shows the permeability values for all 10SS specimens with void ratios less than their initial expansion ratios. A specimen with its void ratio less than its expansion ratio implies that the expanded volume becomes more significant for permeability control. As can be seen in Figure 4-21, most of the data points fall in the range of 10^{-9} cm/sec or slightly higher. When the void ratio further decreases, i.e., e_f is close to the maximum, the variation of the permeability tends to become asymptote. A soil with a permeability of 10^{-9} cm/sec may be considered as impermeable

PERMEABILITY TEST

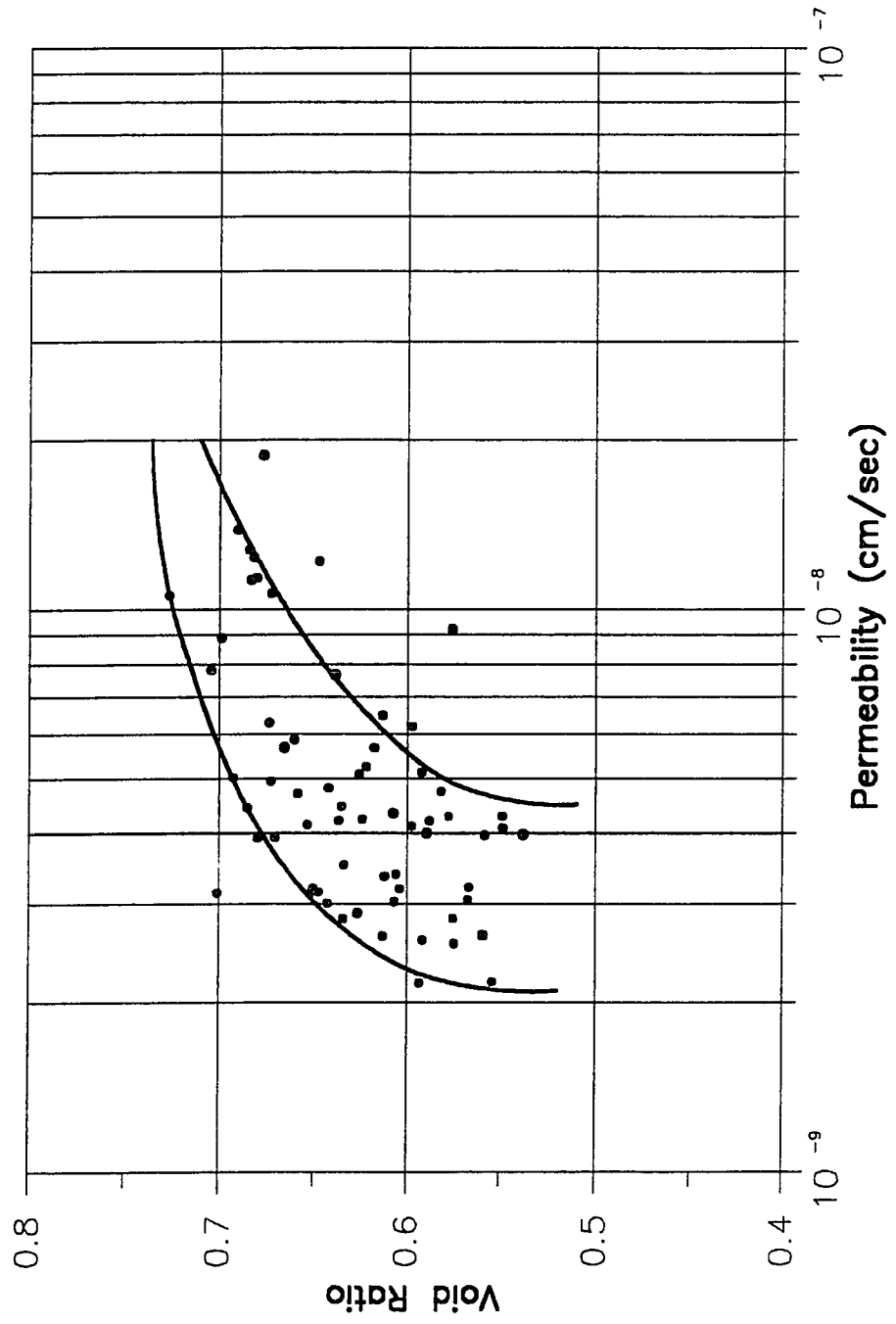


Figure 4-21 Variation of Permeability for 10SS Samples at a Void Ratio Less Than the Initial Expansion Ratio.

for all practical purposes.

The initial values of e_r and e_f of a 5SS30F sample are 0.4 and 0.57, respectively. These values are considerably less than those for 10SS samples. They indicate that 5SS30F sample contains less expanded volume and greater volume of voids. The higher observed permeability of this soil mix supports the previous argument.

For 10SS30F sample, e_r and e_f are 0.66 and 0.96, respectively. These values are close to 10SS soil mixture. However, their permeability values are considerably higher. The proposed hypothesis must be further validated with additional soil mixtures.

The concept of expansion also can be used to explain why phenol 60,000 ppm did not increase the permeability for 10SS samples. Though the chemical caused some reduction of the expansion, the specimen contained sufficient bentonite and expanded to an amount that was able to impede the seepage.

As noted in the consolidometer permeability and flexible wall permeability tests, the concept of expansion appears to be helpful in explaining the permeability response. Therefore, additional studies were conducted to further verify this concept by using triple ring permeameter. Various amount of water or sodium hydroxide solution was added to saturate the 10SS samples to yield different values of expansion ratio and expansion function. Because of the lower expanded volume, the void ratio for the samples prepared with solutions of higher concentrations were limited.

As shown in Figure 4-22, it is clear that for all 10SS samples at a given bentonite content, the void ratio does not show a clear correlation with permeability. The permeability appears to be dependent upon e_r and e_f , and the void ratio thus is not reliable for the permeability prediction. The Spearman analysis indicate that permeability has a strong correlation with e_r and e_f yielding R value of -0.9759 and -0.9104, respectively. Specimens with higher e_r and e_f tend to have lower permeability. The values of e_r and e_f are limited by the amount of expanded volume and the available space for expansion within the sample. For a 10SS sample with sufficient space to accommodate the fully hydrated bentonite, e_r could be 1.0 which is its maximum value. A sample with a void ratio over 1.0 favors this condition. In a sample with a void ratio less than 1.0, e_r will be limited by the pore volume available. Similar conditions apply when sodium hydroxide solutions are used. The expansion ratio drops as a result of the reduction of the expanded volume. It ranges from 0.9 for 0.4 percent solution to virtually zero for a 4 percent solution.

Figure 4-23 shows that a sample of each particular expansion ratio tends to have a minimum permeability when e_f is close to its maximum. The permeability decreased with the increasing e_f . Note that the void ratio is the quotient of the expansion ratio and the expansion function. Samples with an e_r of 1.0 and an initial void ratio greater than 1.0 should have an e_f less than 1.0. This is not true when water

PERMEABILITY TEST

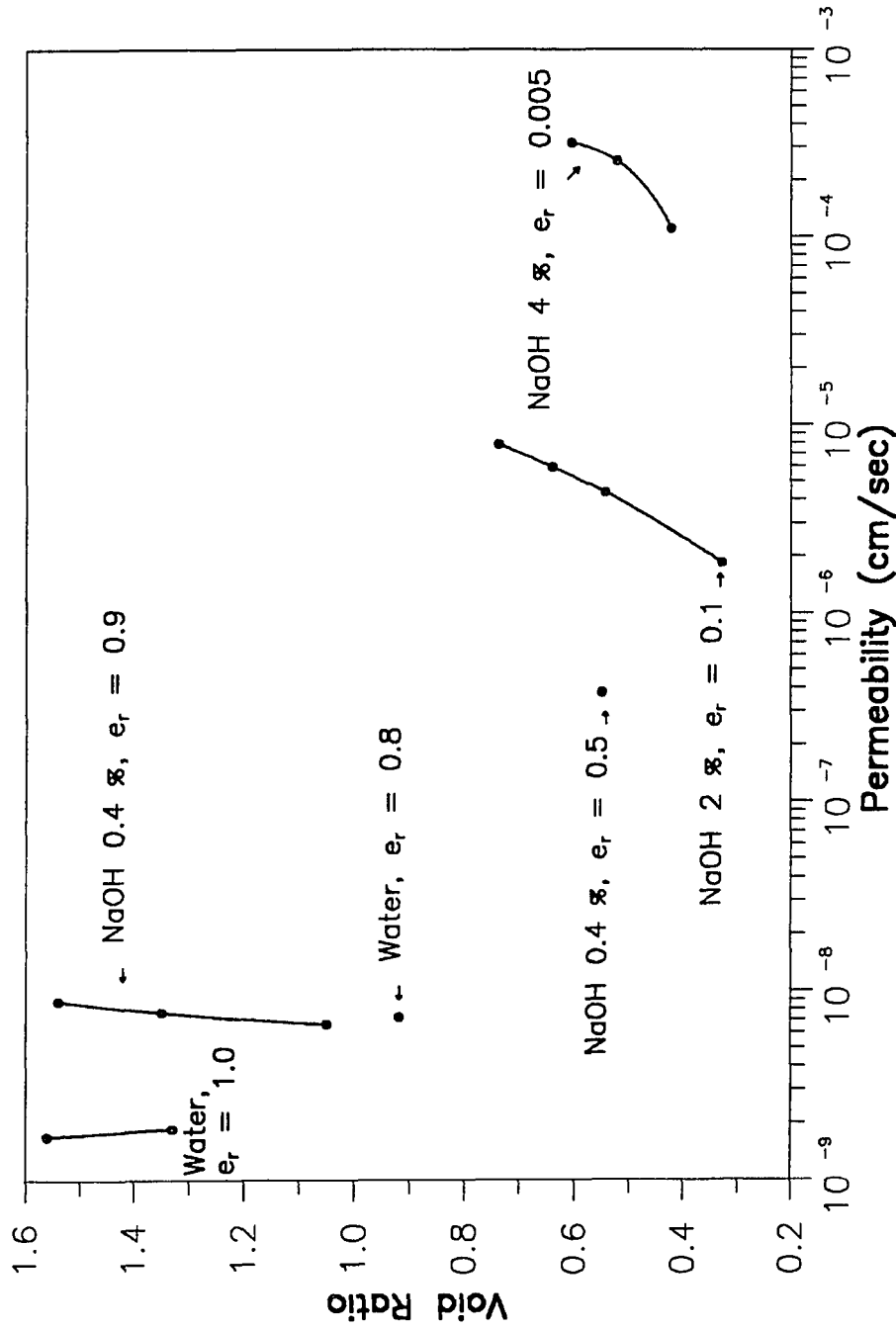


Figure 4-22 Effect of Expansion Ratio on the Permeability of 10SS Samples in a Triple Ring Permeameter.

PERMEABILITY TEST

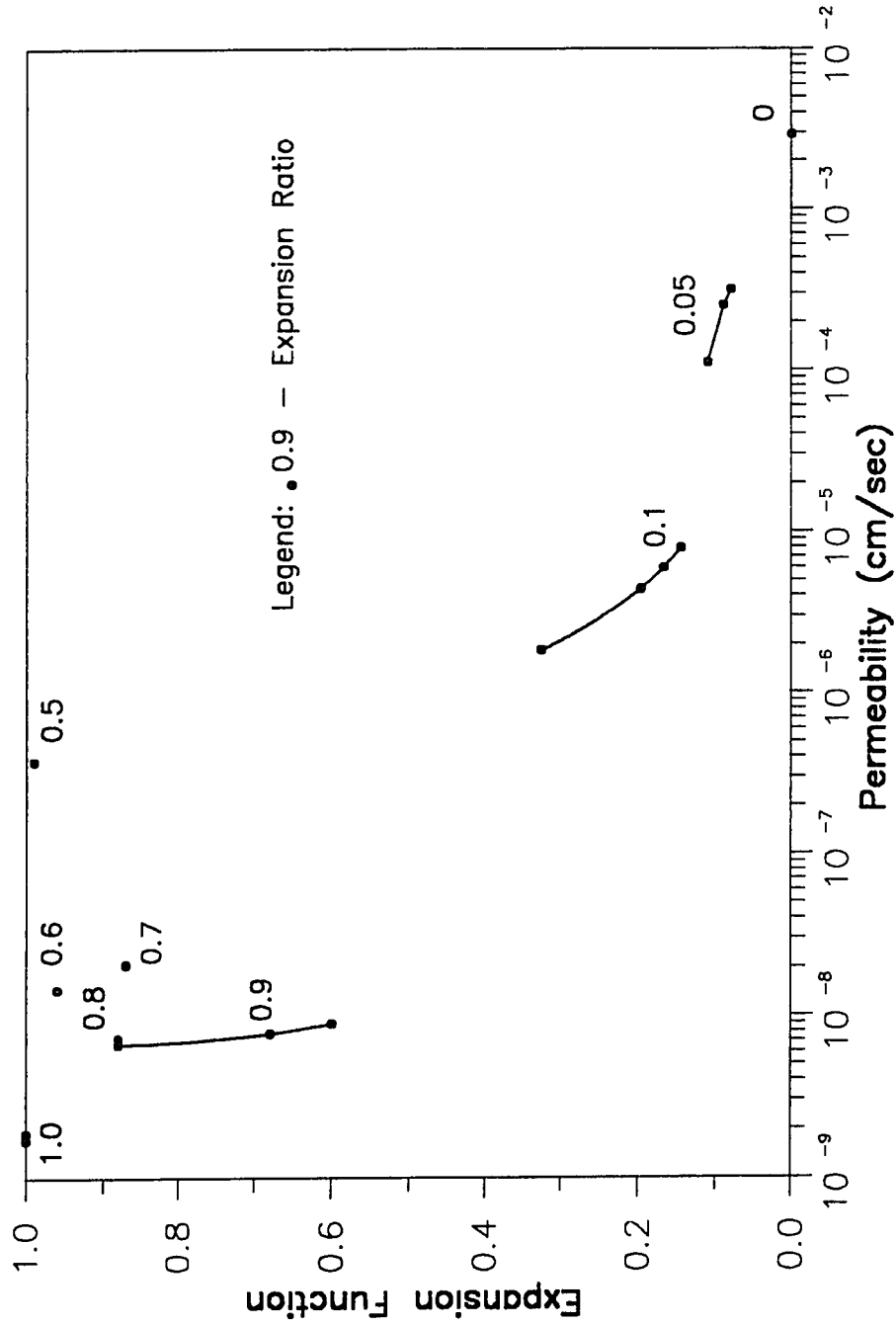


Figure 4-23 Effect of Expansion Function on the Permeability of 10SS Samples in a Triple Ring Permeameter.

is the permeant. When a 10SS sample was prepared at a void ratio greater than 1.3, it appeared to be over-saturated. Slurry was observed at the surface of the sample resulting from the excessive water. Observations made after the completion of the permeability test indicated that the sample volume had reduced and its void ratio was close to 1.0. A layer of filter cake was found at the influent end. This most likely resulted from the slurry that penetrated into the porous granular medium during the hydraulic gradient enforced. As the filtration continues, the hydrated bentonite accumulates in the pores and thus forms a tightly packed layer of filter cake (Xanthakos, 1979). This gelled material provides a positive contribution to impede the seepage and reduce the permeability (D'Appolonia, 1980). When sodium hydroxide solutions were used, filter cake did not appear and the volume of the samples was also found not to change. Apparently, the sodium hydroxide inhibited the forming of the filter cake and made the permeation behave as those samples at lower void ratios.

The concept of the expansion appears to be very helpful in evaluating the permeability behavior of a bentonite mix. For example, in order for a 10 percent bentonite backfill mix to have a 10^{-9} cm/sec permeability, it should be prepared with a minimum e_r of 0.90 and an e_f of at least 0.70. Free expansion tests should be used to determine the potential of bentonite expansion in the proposed permeants. Though the application of this concept has been verified for 10SS

samples only, it should be similar for other bentonite backfill mixes.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

The volume change behavior and its influence on the permeability of bentonite-sand mixes in a contaminant environment were studied. Emphasis was placed on the observations of permeability changes resulting from diluted chemicals. The effect of consolidation on permeability was also evaluated.

Modified index tests were conducted and physical parameters were developed to evaluate the soil-chemical compatibility. Present practices in determining permeability for slurry samples were compared. Based on the evaluation, a new permeameter was developed to compensate for the limitations of conventional apparatus. The properties of fly ash were reviewed and the possibility of its use as a backfill component was examined. Statistical technique was applied where appropriate. Its usefulness for data analyses was noted.

Based on the results of this research, conclusions and recommendations are presented in this chapter.

5.1 Index Tests

The liquid limit tests performed with diluted organic liquids provided a fair indication for the effect of a specific liquid on the bentonite. However, its correlation with the exposure time was not clear. The liquid limit test is relatively inconvenient to perform, and the results are not reliable due to its test methodology.

The use of a free expansion test exhibited that it is an easy and reliable technique for evaluating the soil-chemical interaction qualitatively and quantitatively. It is much more sensitive to chemical type, its concentration, and exposure time. In most liquids, bentonite expanded less than it was in water. The higher the concentration, generally the less the expansion. SS-100 generally showed lower expansion potential than CS-200. It also appears not to be able to resist attack by most of the chemicals tested. CS-200 tends to be more vulnerable in the composite leachate.

Equilibrium time generally correlated well with the expansion behavior in each liquid. Liquids that exhibited a more pronounced effect on the expansion generally reached equilibrium in a shorter time. The equilibrium time for CS-200 is generally much less than that for SS-100. The equilibrium time could serve as a guide to the selection of a proper testing period for obtaining more reliable values for permeability determinations.

The modified sedimentation analyses provided a good indication to show the effect of chemicals on bentonite. It generally presents the extent of flocculation of a soil in a liquid. Statistical regression is helpful in analyzing the data. Based on the results, two physical parameters were derived. The sedimentation index, S_i , is strongly correlated with the characteristics of the liquid. S_i varied little provided that the properties of the liquids are similar. Initial sedimentation, I_s , refers to the type of the soil and

may be used to indicate the possible extent of soil flocculation. I_s tends to drop if soil tends to flocculate or contain more coarse grained materials.

The values of S_i and I_s are altered by the chemical concentrations. As the concentration increased, S_i increased and I_s decreased. Lower I_s and higher S_i indicated a faster sedimentation condition. This in turn implies that a shorter time would be required to complete the soil-chemical interaction.

The results derived from the index tests generally correlated well with the Gouy-Chapman theory. The effect of ionic crowding overrides other physico-chemical factors and controls the behavior of sodium bentonite in the sodium hydroxide solutions.

Bentonite expansion plays a critical role in the effectiveness of impeding permeability in a soil matrix. Permeability is also related to the soil structure. A flocculated structure tends to increase permeability. Based on the results of expansion test and sedimentation analyses, one can conceptually infer the chemical effects on permeability.

Cracking evaluation provided an indication of the effect of different liquids on a soil at their initial contacts. However, observations made based on long term and cyclic conditions proved that no conclusion may be made concerning permeability change.

5.2 Permeability Tests

For organic chemicals, permeability varied little with their type and their concentration. However, the diluted organic chemicals generally showed an adverse effect on permeability. Permeability measured in a consolidometer permeameter for 10SS specimens appears not to have a linear relationship with a void ratio as conventionally described for sand and clay. Permeability varied little once the void ratio became less than 0.60. Test data indicated that for a void ratio at 0.60 or higher, permeability with diluted organic chemicals was generally greater than with water. The difference varied from negligible to two-to-three-fold. The higher the void ratio, the more the difference. When the void ratio decreased to 0.60 or less, the difference tends to be minimized. Since a void ratio less than 0.60 was caused by a consolidation pressure of 100 kPa or higher, the lesser influence of chemicals on permeability can be attributed to the effect of higher consolidation pressure.

Permeability tested with chromium showed a completely reverse tendency to that found with other permeants. This is attributed to the coupling effects of high electrolyte concentration and hexavalent of chromium. The shrinkage of the bentonite could induce significant sideleak and result in higher permeability values.

Sodium hydroxide solutions of higher concentrations strongly inhibited the bentonite expansion and caused an increase in permeability. When tested with 4 percent

solution, the bentonite was completely degraded and showed a permeability of the order of 10^{-4} cm/sec.

The change in permeability generally agrees with the results found in the index tests and can be explained by the Gouy-Chapman theory.

Permeability values from a consolidometer permeameter showed a medium to strong correlation with consolidation pressure. Permeability decreased with the increasing consolidation pressure. The decrease occurred relatively rapidly for stresses less than 25 kPa. For SB wall that consolidation usually is not significant, permeability test should be performed with a consolidation pressure no more than 25 kPa. At such a low pressure, the consolidometer permeameter is not able to prevent the occurrence of sidewall leakage. It is also impossible to apply an upward gradient without taking the risk of boiling the soft specimen. Use of consolidometer permeameter for testing soft SB material thus is not appropriate.

The specimen assembly is more troublesome for a flexible wall permeameter than that for a consolidometer permeameter. In addition, the flexible wall permeameter normally requires a higher gradient to complete the measurements. The greater hydraulic gradient tends to reduce permeability in a flexible wall permeameter. The flexible wall permeameter appears to be unsuitable for the soft backfill mix samples.

Long term observations made on 10SS samples tested with phenol and leachate showed that permeability decreased with

permeation. The higher consolidation pressure presented a pronounced effect on the permeability. Tests conducted on cracked samples also indicated that the consolidation pressure tends to close the internal fissures and cracks and leads to a lower permeability than one would expect. The consolidation pressure thus tends to yield a lower permeability value and not conservative for design.

Fly ash from a local source appears to have an adverse effect on permeability when it partially replaced sand. This is attributed to the coupling effects of ion replacements and higher electrolyte concentration resulting from the interactions of sodium bentonite and fly ash in the soil matrix. However, fly ash from other sources may not have the same detrimental effect on bentonite. This is because fly ashes are quite variable in nature and their properties are highly dependent upon the source of coal and the power plant operational characteristics.

For fly ash samples, there appears to be a fairly good agreement between permeability determined indirectly from consolidation tests and that determined from direct measurements in a consolidometer permeameter. For 10SS samples this correlation could not be established. This is possibly due to the granular nature of the sample.

The triple ring permeameter was verified to function properly. It has many advantages over other conventional apparatus. However, it requires more sophisticated instrumentation and additional effort to manage the

operation.

Based on the results from CP and FP tests, the void ratio according to the conventional three-phase diagram can not appropriately describe a bentonite mixture from the permeability viewpoint. Tests conducted with a triple ring permeameter further verified that the void ratio does not show a clear relationship with permeability for bentonite mixtures.

A four-phase soil model which accounts for the effect of bentonite expansion is proposed. Based on this model, two physical parameters, expansion ratio, e_r , and expansion function, e_f , are developed. The permeability is highly dependent upon e_r and e_f . It decreased with the increasing e_r and e_f . Each sample with a particular e_r tends to have a minimum permeability when e_f is close to its maximum value. Use of e_r and e_f can clearly identify the permeability behavior of bentonite mixtures.

The preparation of specimen with chemicals eliminated the potential sidewall leakage that has been reported in cases where samples were passively permeated with the chemicals. This technique improves the accuracy of the permeability measurements and expedites the testing.

The backfill mix is relatively incompressible. The chemicals are not likely to cause any degradation for the compressibility of the backfill mix.

5.3 Recommendations

Based on the results of this study, the following are recommended for further research.

- Conduct expansion test and sedimentation test on other types of expansive materials to verify the usefulness of physical parameters proposed in this study.
- Avoid using flexible wall permeameter to measure the permeability of soft materials such as SB backfill mixes.
- Use triple ring or consolidometer permeameter for slurry wall design.
- Consolidate the specimen at a pressure of 25 kPa or less.
- Study further the long term effects of composite leachate on the permeability of bentonite materials.
- Evaluate the properties of fly ash and further verify the compatibility of different fly ashes with bentonite.
- Observe the impact of cracks and fissures on permeability. Prepare the specimen so that cracks and fissures are parallel to the flow direction.
- Use statistical techniques to facilitate data analyses and establish testing criteria.

APPENDIX A

PERMEABILITY TEST RESULTS

Table A-1
Summary of Consolidometer Permeability Test Results¹

Type of Permeant	Consolidation Pressure (kPa)	Void Ratio	Expansion Ratio	Expansion Function	Permeability ² @ 20°C (cm/sec)	Permeability ³ @ 20°C (cm/sec)
Water (1) ⁴	0	0.77	0.72	0.94	-	-
	5	0.76		0.95	1.74×10^{-7}	-
	10	0.73		0.99	3.27×10^{-9}	7.34×10^{-9}
	25	0.69		1.00	3.38×10^{-9}	4.44×10^{-9}
	50	0.65		1.00	4.59×10^{-9}	3.15×10^{-9}
	100	0.60		1.00	4.93×10^{-9}	3.03×10^{-9}
	200	0.57		1.00	4.45×10^{-9}	3.21×10^{-9}
Water (2)	0	0.77	0.72	0.94	-	-
	5	0.76		0.95	5.77×10^{-9}	-
	10	0.74		0.98	8.85×10^{-9}	1.43×10^{-8}
	25	0.70		1.00	3.42×10^{-9}	3.14×10^{-9}
	50	0.67		1.00	5.32×10^{-9}	6.30×10^{-9}
	100	0.64		1.00	8.80×10^{-9}	3.01×10^{-9}
	200	0.60		1.00	4.89×10^{-9}	3.19×10^{-9}
Aniline 500 ppm (1)	0	0.74	0.70	0.95	-	-
	10	0.71		0.99	-	-
	25	0.67		1.00	7.48×10^{-9}	4.95×10^{-9}
	50	0.63		1.00	6.21×10^{-9}	5.10×10^{-9}
	100	0.59		1.00	6.36×10^{-9}	4.20×10^{-9}
	200	0.55		1.00	5.76×10^{-9}	4.29×10^{-9}
Aniline 500 ppm (2)	0	0.76	0.72	0.95	-	-
	10	0.73		0.98	-	-
	25	0.68		1.00	8.23×10^{-9}	3.94×10^{-9}
	50	0.64		1.00	7.58×10^{-9}	4.46×10^{-9}
	100	0.60		1.00	4.42×10^{-9}	4.11×10^{-9}
	200	0.56		1.00	6.14×10^{-9}	3.97×10^{-9}
Aniline 1000 ppm (1)	0	0.76	0.70	0.92	-	-
	10	0.71		0.98	1.63×10^{-7}	6.68×10^{-9}
	25	0.67		1.00	5.45×10^{-9}	5.69×10^{-9}
	50	0.63		1.00	2.69×10^{-9}	2.89×10^{-9}

1) All samples tested were 10SS except where noted.

2) Indirect permeability, calculated based on Terzaghi coefficient of consolidation.

3) Direct permeability, measured based on falling head technique.

4) Number of sample repetition.

5) Sample tested was 5SS30F.

6) Sample tested was 10SS30F.

7) Sample tested was 30F.

8) Sample tested was sand.

Table A-1
Summary of Consolidometer Permeability Test Result (Cont'd)¹

Type of Permeant	Consolidation Pressure (kPa)	Void Ratio	Expansion Ratio	Expansion Function	Permeability ² @ 20°C (cm/sec)	Permeability ³ @ 20°C (cm/sec)
Aniline 1000 ppm (1)	100	0.59		1.00	2.57×10^{-9}	4.01×10^{-9}
	200	0.56		1.00	5.36×10^{-9}	2.64×10^{-9}
Aniline 1000 ppm (2)	0	0.76	0.70	0.93	-	-
	10	0.68		1.00	1.36×10^{-7}	1.88×10^{-8}
	25	0.64		1.00	3.60×10^{-9}	7.68×10^{-9}
	50	0.61		1.00	3.45×10^{-9}	4.34×10^{-9}
	100	0.58		1.00	4.47×10^{-9}	9.20×10^{-9}
	200	0.54		1.00	3.84×10^{-9}	3.98×10^{-9}
Phenol 500 ppm (1)	0	0.76	0.71	0.93	-	-
	25	0.69		1.00	2.13×10^{-8}	1.39×10^{-8}
	50	0.65		1.00	6.65×10^{-9}	3.13×10^{-9}
	100	0.61		1.00	6.25×10^{-9}	3.36×10^{-9}
	200	0.57		1.00	3.89×10^{-9}	2.55×10^{-9}
Phenol 500 ppm (2)	0	0.78	0.74	0.95	-	-
	25	0.73		1.00	1.63×10^{-8}	1.06×10^{-8}
	50	0.69		1.00	4.64×10^{-9}	5.03×10^{-9}
	100	0.65		1.00	2.97×10^{-9}	4.14×10^{-9}
	200	0.62		1.00	3.74×10^{-9}	5.68×10^{-9}
Phenol 1000 ppm (1)	0	0.76	0.73	0.96	-	-
	25	0.67		1.00	1.53×10^{-8}	1.07×10^{-8}
	50	0.63		1.00	2.86×10^{-9}	2.82×10^{-9}
	100	0.59		1.00	3.61×10^{-9}	2.17×10^{-9}
	200	0.55		1.00	4.39×10^{-9}	2.18×10^{-9}
Phenol 1000 ppm (2)	0	0.77	0.75	0.97	-	-
	25	0.70		1.00	1.43×10^{-8}	8.89×10^{-9}
	50	0.66		1.00	3.04×10^{-9}	5.87×10^{-9}
	100	0.62		1.00	3.07×10^{-9}	5.26×10^{-9}
	200	0.58		1.00	4.79×10^{-9}	4.75×10^{-9}
Phenol 10000 ppm (1)	0	0.76	0.73	0.96	-	-
	25	0.68		1.00	1.70×10^{-8}	1.14×10^{-8}
	50	0.64		1.00	3.31×10^{-9}	4.21×10^{-9}
	100	0.59		1.00	3.75×10^{-9}	2.58×10^{-9}
	200	0.55		1.00	5.32×10^{-9}	4.09×10^{-9}
Phenol 10000 ppm (2)	0	0.77	0.72	0.94	-	-
	25	0.68		1.00	9.91×10^{-9}	1.13×10^{-8}
	50	0.64		1.00	2.07×10^{-9}	4.82×10^{-9}

Table A-1
Summary of Consolidometer Permeability Test Result (Cont'd)¹

Type of Permeant	Consolidation Pressure (kPa)	Void Ratio	Expansion Ratio	Expansion Function	Permeability ² @ 20°C (cm/sec)	Permeability ³ @ 20°C (cm/sec)
Phenol 10000 ppm (2)	100	0.61		1.00	2.46×10^{-9}	3.39×10^{-9}
	200	0.57		1.00	3.66×10^{-9}	3.05×10^{-9}
Toluene 100 ppm (1)	0	0.75	0.72	0.96	-	-
	10	0.71		1.00	3.55×10^{-7}	7.38×10^{-9}
	25	0.65		1.00	1.06×10^{-8}	4.67×10^{-9}
	50	0.61		1.00	5.13×10^{-9}	2.41×10^{-8}
	100	0.56		1.00	3.71×10^{-9}	1.13×10^{-8}
	200	0.52		1.00	3.22×10^{-9}	3.05×10^{-9}
Toluene 100 ppm (2)	0	0.74	0.71	0.96	-	-
	10	0.72		0.98	7.10×10^{-8}	1.42×10^{-8}
	25	0.68		1.00	3.55×10^{-9}	6.73×10^{-9}
	50	0.64		1.00	2.82×10^{-9}	7.22×10^{-8}
	100	0.60		1.00	2.88×10^{-9}	1.02×10^{-9}
	200	0.56		1.00	2.63×10^{-9}	4.67×10^{-9}
Toluene 500 ppm (1)	0	0.76	0.74	0.98	-	-
	25	0.66		1.00	1.25×10^{-8}	9.36×10^{-8}
	50	0.62		1.00	3.57×10^{-9}	1.00×10^{-8}
	100	0.59		1.00	2.10×10^{-9}	9.07×10^{-9}
	200	0.55		1.00	5.38×10^{-9}	7.73×10^{-9}
Toluene 1000 ppm (1)	0	0.74	0.67	0.91	-	-
	25	0.69		0.97	3.78×10^{-8}	1.41×10^{-8}
	50	0.66		1.00	3.43×10^{-9}	3.47×10^{-8}
	100	0.62		1.00	6.71×10^{-9}	1.01×10^{-8}
	200	0.57		1.00	4.71×10^{-9}	9.78×10^{-9}
Toluene 1000 ppm (2)	0	0.77	0.73	0.94	-	-
	10	0.73		1.00	1.21×10^{-7}	5.41×10^{-9}
	25	0.68		1.00	4.84×10^{-9}	6.19×10^{-9}
	50	0.63		1.00	4.36×10^{-9}	2.00×10^{-9}
	100	0.59		1.00	6.65×10^{-9}	4.30×10^{-9}
	200	0.54		1.00	3.58×10^{-9}	4.34×10^{-9}
1,1,1-Trichloroethane 100 ppm (1)	0	0.76	0.72	0.95	-	-
	25	0.70		1.00	6.00×10^{-9}	1.09×10^{-8}
	50	0.66		1.00	4.09×10^{-9}	4.71×10^{-9}
	100	0.62		1.00	5.09×10^{-8}	4.23×10^{-9}
	200	0.59		1.00	1.18×10^{-8}	5.14×10^{-9}

Table A-1
Summary of Consolidometer Permeability Test Result (Cont'd)¹

Type of Permeant	Consolidation Pressure (kPa)	Void Ratio	Expansion Ratio	Expansion Function	Permeability ² @ 20°C (cm/sec)	Permeability ³ @ 20°C (cm/sec)
1,1,1-Trichloroethane	0	0.76	0.74	0.97	-	-
100 ppm (2)	25	0.68		1.00	1.69 x 10 ⁻⁸	1.24 x 10 ⁻⁸
	50	0.65		1.00	4.83 x 10 ⁻⁹	3.20 x 10 ⁻⁹
	100	0.61		1.00	2.78 x 10 ⁻⁹	2.63 x 10 ⁻⁹
	200	0.58		1.00	5.30 x 10 ⁻⁹	2.83 x 10 ⁻⁹
1,1,1-Trichloroethane	0	0.76	0.74	0.97	-	-
1000 ppm (1)	25	0.68		1.00	2.04 x 10 ⁻⁸	1.28 x 10 ⁻⁸
	50	0.65		1.00	4.92 x 10 ⁻⁹	1.22 x 10 ⁻⁹
	100	0.61		1.00	6.04 x 10 ⁻⁹	6.49 x 10 ⁻⁹
	200	0.58		1.00	4.26 x 10 ⁻⁹	4.29 x 10 ⁻⁹
1,1,1-Trichloroethane	0	0.77	0.73	0.95	-	-
1000 ppm (2)	25	0.70		1.00	8.75 x 10 ⁻⁹	7.83 x 10 ⁻⁹
	50	0.67		1.00	3.76 x 10 ⁻⁹	3.94 x 10 ⁻⁹
	100	0.63		1.00	4.85 x 10 ⁻⁹	3.52 x 10 ⁻⁹
	200	0.60		1.00	3.23 x 10 ⁻⁹	6.20 x 10 ⁻⁹
Chromium	0	0.76	0.74	0.97	-	-
50 ppm (1)	25	0.69		1.00	8.48 x 10 ⁻⁹	1.28 x 10 ⁻⁸
	50	0.65		1.00	4.25 x 10 ⁻⁹	3.94 x 10 ⁻⁷
	100	0.62		1.00	6.78 x 10 ⁻⁹	3.94 x 10 ⁻⁷
	200	0.59		1.00	7.92 x 10 ⁻⁹	2.07 x 10 ⁻⁷
Chromium	0	0.78	0.76	0.97	-	-
50 ppm (2)	25	0.69		1.00	9.72 x 10 ⁻⁹	-
	50	0.65		1.00	4.51 x 10 ⁻⁹	4.85 x 10 ⁻⁶
	100	0.62		1.00	4.49 x 10 ⁻⁸	3.62 x 10 ⁻⁶
	200	0.59		1.00	4.60 x 10 ⁻⁸	5.15 x 10 ⁻⁶
Chromium	0	0.77	0.75	0.97	-	-
50 ppm (3)	25	0.69		1.00	1.06 x 10 ⁻⁸	-
	50	0.65		1.00	5.42 x 10 ⁻⁹	1.17 x 10 ⁻⁵
	100	0.61		1.00	3.05 x 10 ⁻⁹	8.47 x 10 ⁻⁶
	200	0.57		1.00	7.00 x 10 ⁻⁹	4.13 x 10 ⁻⁶
Chromium	0	0.77	0.75	0.97	-	-
50 ppm (4)	25	0.71		1.00	4.96 x 10 ⁻⁹	9.75 x 10 ⁻⁹
	50	0.67		1.00	3.64 x 10 ⁻⁹	1.22 x 10 ⁻⁷
	100	0.63		1.00	2.65 x 10 ⁻⁹	3.97 x 10 ⁻⁷
	200	0.60		1.00	3.64 x 10 ⁻⁹	3.67 x 10 ⁻⁷
Water (1) ⁵	0	0.78	0.49	0.63	-	-
	10	0.65		0.76	8.77 x 10 ⁻⁷	7.91 x 10 ⁻⁷

Table A-1
Summary of Consolidometer Permeability Test Result (Cont'd)¹

Type of Permeant	Consolidation Pressure (kPa)	Void Ratio	Expansion Ratio	Expansion Function	Permeability ² @ 20°C (cm/sec)	Permeability ³ @ 20°C (cm/sec)
Water (1) ⁵	25	0.61		0.81	5.15 x 10 ⁻⁷	6.79 x 10 ⁻⁷
	50	0.57		0.86	4.40 x 10 ⁻⁷	5.13 x 10 ⁻⁷
	100	0.53		0.94	3.88 x 10 ⁻⁷	3.61 x 10 ⁻⁷
	200	0.48		1.00	2.67 x 10 ⁻⁷	2.81 x 10 ⁻⁷
Water (2) ⁵	0	0.75	0.49	0.66	-	-
	10	0.64		0.77	8.27 x 10 ⁻⁷	8.13 x 10 ⁻⁷
	25	0.60		0.83	3.66 x 10 ⁻⁷	6.49 x 10 ⁻⁷
	50	0.55		0.90	4.55 x 10 ⁻⁷	4.84 x 10 ⁻⁷
	100	0.50		0.99	4.18 x 10 ⁻⁷	3.35 x 10 ⁻⁷
	200	0.45		1.00	3.24 x 10 ⁻⁷	2.78 x 10 ⁻⁷
Toluene 1000 ppm (1) ⁶	0	0.68	0.66	0.96	-	-
	10	0.66		0.99	1.78 x 10 ⁻⁷	2.15 x 10 ⁻⁷
	25	0.64		1.00	1.14 x 10 ⁻⁸	1.88 x 10 ⁻⁷
	50	0.61		1.00	7.83 x 10 ⁻⁸	1.94 x 10 ⁻⁷
	100	0.56		1.00	8.61 x 10 ⁻⁷	1.53 x 10 ⁻⁸
	200	0.49		1.00	1.06 x 10 ⁻⁷	9.84 x 10 ⁻⁸
Toluene 1000 ppm (2) ⁶	0	0.69	0.70	1.00	-	-
	10	0.68		1.00	6.51 x 10 ⁻⁸	2.00 x 10 ⁻⁷
	25	0.65		1.00	1.16 x 10 ⁻⁸	1.90 x 10 ⁻⁷
	50	0.61		1.00	6.80 x 10 ⁻⁸	1.63 x 10 ⁻⁷
	100	0.56		1.00	7.65 x 10 ⁻⁸	1.29 x 10 ⁻⁸
	200	0.50		1.00	6.09 x 10 ⁻⁸	9.31 x 10 ⁻⁸
Water (1) ⁷	0	0.70	-	-	-	-
	100	0.51		-	-	3.58 x 10 ⁻⁵
	200	0.49		-	-	2.76 x 10 ⁻⁵
Water (2) ⁷	0	0.33	-	-	-	-
	100	0.29		-	-	2.42 x 10 ⁻⁵
	200	0.24		-	-	1.43 x 10 ⁻⁵
Water (1) ⁸	0	0.79	-	-	-	-
	100	0.77		-	-	2.93 x 10 ⁻³
	200	0.75		-	-	2.83 x 10 ⁻³
Water (2) ⁸	0	0.85	-	-	-	-
	100	0.79		-	-	3.10 x 10 ⁻³
	200	0.78		-	-	3.05 x 10 ⁻³

Table A-2
Summary of Flexible Wall Permeability Test Results¹

Type of Soil	Type of Permeant	Consolidation Pressure (kPa)	Void Ratio	Permeability @ 20°C (cm/sec)
10SS	Water	0	0.90	-
		50	0.88	6.31 x 10 ⁻⁹
		100	0.84	3.86 x 10 ⁻⁹
		200	0.81	1.91 x 10 ⁻⁹
10CS	Water	0	0.90	-
		50	0.79	1.28 x 10 ⁻⁸
		100	0.74	9.02 x 10 ⁻⁹
		200	0.73	6.72 x 10 ⁻⁹
10CS ²	Water	0	0.80	-
		50	0.73	1.84 x 10 ⁻⁸
		100	0.72	9.45 x 10 ⁻⁹
		200	0.71	5.64 x 10 ⁻⁹
5SS	Water	0	0.78	-
		50	0.72	9.84 x 10 ⁻⁷
		100	0.71	7.02 x 10 ⁻⁷
		200	0.70	2.72 x 10 ⁻⁷
5SS30F	Water	0	0.89	-
		50	0.66	2.74 x 10 ⁻⁷
		100	0.62	2.15 x 10 ⁻⁷
		200	0.57	1.64 x 10 ⁻⁷
10SS30F	Water	0	0.90	-
		50	0.74	1.77 x 10 ⁻⁷
		100	0.68	1.07 x 10 ⁻⁷
		200	0.63	7.89 x 10 ⁻⁸
10SS ³	Water	0	0.94	-
		100	0.79	9.10 x 10 ⁻⁸
		200	0.71	2.15 x 10 ⁻⁸
		300	0.69	1.88 x 10 ⁻⁸

- 1) All samples were prepared at normal condition and tested with latex membrane except where noted.
- 2) Sample experienced one cycle of wet and oven dry treatment.
- 3) Sample was tested with neoprene membrane.
- 4) Sample experienced three cycles of wet and cool dry treatments.

Table A-2
Summary of Flexible Wall Permeability Test Results (Cont'd)¹

Type of Soil	Type of Permeant	Consolidation Pressure (kPa)	Void Ratio	Permeability @ 20°C (cm/sec)
10SS	Phenol 60,000 ppm	0	0.90	-
		50	0.77	2.72×10^{-9}
		100	0.74	2.43×10^{-9}
		200	0.70	1.91×10^{-9}
10SS ⁴	Phenol 60,000 ppm	0	0.91	-
		50	0.73	2.38×10^{-9}
		100	0.69	1.79×10^{-9}
		200	0.64	1.48×10^{-9}

Table A-3
Summary of Triple Ring Permeability Test Result¹

Type of Permeant	Void Ratio	Expansion Ratio	Expansion Function	Permeability @ 20°C (cm/sec)
Water	1.56	1.04	1.00	1.71×10^{-9}
	1.33	1.04	1.00	1.88×10^{-9}
	0.92	0.82	0.88	7.24×10^{-9}
	0.83	0.72	0.87	2.08×10^{-8}
	0.64	0.61	0.96	1.46×10^{-8}
NaOH 0.4 %	1.54	0.92	0.60	8.90×10^{-9}
	1.35	0.92	0.68	7.72×10^{-9}
	1.05	0.92	0.88	6.68×10^{-9}
	0.55	0.54	0.99	3.79×10^{-7}
NaOH 2.0 %	0.74	0.11	0.15	7.92×10^{-6}
	0.64	0.11	0.17	5.99×10^{-6}
	0.55	0.11	0.20	4.39×10^{-6}
	0.33	0.11	0.32	1.85×10^{-6}
NaOH 4.0 %	0.61	0.05	0.08	3.17×10^{-4}
	0.52	0.05	0.09	2.54×10^{-4}
	0.42	0.05	0.11	1.11×10^{-4}
Water ²	0.75	0.00	0.00	2.93×10^{-3}

1) All samples tested were 10SS except where noted. Tests were conducted with a consolidation pressure of 10 kPa.

2) Sample consists of clean sand only.

APPENDIX B

STATISTICAL ANALYSIS OUTPUT

STATS+: B A S I C S T A T I S T I C S

01-17-89 09:36:22

CORRELATION OF VOID RATIO, CONSOLIDATION PRESSURE, AND PERMEABILITY

main input data file is raw data file: a:10ss.CSS

[261 cases with 4 variables]

Correlation of e, P, k-measured, and k-computed -- 10SS Samples

VARIABLE LIST and missing data values:

1	p (-9999.)	2	e (-9999.)	3	kcv (-9999.)
4	km (-9999.)						

square correlation matrix

cases in the file: 261

mode: standard

accepted cases: 93

precision: single

MD deletion: pairwise

stats+:	Correlations r(x,y)			
basic	N. of CASES = 93 [from 261]			
stats	(MD pairwise deleted)			
standard mode	p	e	kcv	km
p	1.0000	-.8655	-.2768	-.4146
e	-.8655	1.0000	.4301	.4579
kcv	-.2768	.4301	1.0000	.2461
km	-.4146	.4579	.2461	1.0000

CORRELATION OF VOID RATIO, CONSOLIDATION PRESSURE, AND PERMEABILITY

main input data file is raw data file: a:10ss.CSS

[261 cases with 4 variables]

Correlation of e, P, k-measured, and k-computed -- 10SS Samples

VARIABLE LIST and missing data values:

1	p (-9999.)	2	e (-9999.)	3	kcv (-9999.)
4	km (-9999.)				

OPTIONS:

current case selection condition is:

INCLUDE if:

v2>=0.6

square correlation matrix

cases in the file: 261

selected cases: 67

accepted cases: 67

mode: standard

precision: single

MD deletion: pairwise

stats+:	Correlations r(x,y)			
basic	N. of CASES = 67			
stats	(MD pairwise deleted)			
standard mode	p	e	kcv	km
p	1.0000	-.7405	-.2997	-.3822
e	-.7405	1.0000	.5021	.4799
kcv	-.2997	.5021	1.0000	.2360
km	-.3822	.4799	.2360	1.0000

CORRELATION OF VOID RATIO, EXPANSION PARAMETERS, AND PERMEABILITY

main input data file is raw data file: b:10ss.css

[261 cases with 4 variables]
 Correlation of e, P, k-measured, and k-computed -- 10SS Samples

VARIABLE LIST and missing data values:

1	p (-9999.)	2	e (-9999.)	3	kcv (-9999.)
4	km (-9999.)				

OPTIONS:

current case selection condition is:

INCLUDE if:
 v2<0.6

square correlation matrix

cases in the file: 261
 selected cases: 194
 accepted cases: 26

mode: standard
 precision: single
 MD deletion: pairwise

stats+:	Correlations r(x,y)			
basic	N. of CASES = 26 [from 194]			
stats	(MD pairwise deleted)			
standard mode	p	e	kcv	km
p	1.0000	-.5932	.1928	-.3001
e	-.5932	1.0000	.1993	.0705
kcv	.1928	.1993	1.0000	-.0426
km	-.3001	.0705	-.0426	1.0000

STATS+: B A S I C S T A T I S T I C S

01-17-89 09:39:52

CORRELATION OF VOID RATIO, CONSOLIDATION PRESSURE, AND PERMEABILITY

main input data file is raw data file: a:10ss30F.CSS

[261 cases with 4 variables]

Correlation of e, P, k-measured, and k-computed -- 10SS30F Samples

VARIABLE LIST and missing data values:

1	p (-9999.)	2	e (-9999.)	3	kcv (-9999.)
4	km (-9999.)						

square correlation matrix

cases in the file: 261

mode: standard

accepted cases: 10

precision: single

MD deletion: pairwise

stats+:	Correlations r(x,y)			
basic	N. of CASES = 10 [from 261]			
stats	(MD pairwise deleted)			
standard mode	p	e	kcv	km
p	1.0000	-.9801	-.3421	-.9646
e	-.9801	1.0000	.3420	.9576
kcv	-.3421	.3420	1.0000	.4425
km	-.9646	.9576	.4425	1.0000

STATS+: B A S I C S T A T I S T I C S 01-17-89 09:41:30

CORRELATION OF VOID RATIO, CONSOLIDATION PRESSURE, AND PERMEABILITY

main input data file is raw data file: a:5ss30F.CSS

[261 cases with 4 variables]
 Correlation of e, P, k-measured, and k-computed -- 5SS30F Samples.

VARIABLE LIST and missing data values:

1	p (-9999.)	2	e (-9999.)	3	kcv (-9999.)
4	km (-9999.)						

square correlation matrix

cases in the file: 261

mode: standard

accepted cases: 10

precision: single

MD deletion: pairwise

stats+:	Correlations r(x,y)			
basic	N. of CASES = 10 [from 261]			
stats	(MD pairwise deleted)			
standard mode	p	e	kcv	km
p	1.0000	-.9354	-.6979	-.8892
e	-.9354	1.0000	.8091	.9778
kcv	-.6979	.8091	1.0000	.8391
km	-.8892	.9778	.8391	1.0000

CORRELATION OF VOID RATIO, EXPANSION PARAMETERS, AND PERMEABILITY

main input data file is raw data file: a:triple.CSS

[17 cases with 4 variables]

Correlation of e, er, ef, and k-measured -- 10SS Samples

VARIABLE LIST and missing data values:

4	km (-9999.)	1	e (-9999.)	2	er (-9999.)
3	ef (-9999.)				

correlation matrix

cases in the file: 17

mode: standard

accepted cases: 17

precision: single

MD deletion: pairwise

stats+:	Correlations r(x,y)		
basic	N. of CASES = 17		
stats	(MD pairwise deleted)		
standard			
mode	e	er	ef
km	-.1091	-.3635	-.4144

STATS+: NONPARAMETRIC STATISTICS 01-23-89 16:41:10

CORRELATION OF VOID RATIO, CONSOLIDATION PRESSURE, AND PERMEABILITY

main input data file is raw data file: a:10ss.css

[261 cases with 4 variables]
 Correlation of e, P, k-measured, and k-computed -- 10SS Samples

VARIABLE LIST and missing data values:

4 km	(-9999.00)	1 p	(-9999.00)	2 e	(-9999.00)
3 kcv	(-9999.00)				

OPTIONS:

current case selection condition is:

INCLUDE if:

v2>=0.6

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with p			
	Valid N	Spearman R	Z	signif.
Result	65	-.54222	4.33779	.00009

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with e			
	Valid N	Spearman R	Z	signif.
Result	65	.51449	4.11592	.00016

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with kcv			
	Valid N	Spearman R	Z	signif.
Result	65	.50263	4.02102	.00020

STATS+: NONPARAMETRIC STATISTICS 04-21-89 15:32:36

CORRELATION OF VOID RATIO, EXPANSION PARAMETERS, AND PERMEABILITY

main input data file is raw data file: b:10ss.css
 [261 cases with 4 variables]
 Correlation of e, P, k-measured, and k-computed -- 10SS Samples

VARIABLE LIST and missing data values:

4 km	(-9999.00)	1 p	(-9999.00)	2 e	(-9999.00)
3 kcv	(-9999.00)				

OPTIONS:

current case selection condition is:
 INCLUDE if:
 v2<0.6

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with p			
	Valid N	Spearman R	Z	signif.
Result	26	-.14447	.72235	.47712

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with e			
	Valid N	Spearman R	Z	signif.
Result	26	.04834	.24169	.79575

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with kcv			
	Valid N	Spearman R	Z	signif.
Result	26	.11318	.56591	.57857

STATS+: NONPARAMETRIC STATISTICS 01-23-89 16:46:13

CORRELATION OF VOID RATIO, CONSOLIDATION PRESSURE, AND PERMEABILITY

main input data file is raw data file: a:10ss30F.css

[261 cases with 4 variables]

Correlation of e, P, k-measured, and k-computed -- 10SS30F Samples

VARIABLE LIST and missing data values:

4 km	(-9999.00)	1 p	(-9999.00)	2 e	(-9999.00)
3 kcv	(-9999.00)				

OPTIONS:

NONE

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with p			
	Valid N	Spearman R	Z	signif.
Result	10	-.93550	2.80649	.00524

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with e			
	Valid N	Spearman R	Z	signif.
Result	10	.92075	2.76225	.00591

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with kv			
	Valid N	Spearman R	Z	signif.
Result	10	.39394	1.18182	.23532

STATS+: NONPARAMETRIC STATISTICS 01-23-89 16:49:59

CORRELATION OF VOID RATIO, CONSOLIDATION PRESSURE, AND PERMEABILITY

main input data file is raw data file: a:5ss30f.css

[261 cases with 4 variables]

Correlation of e, P, k-measured, and k-computed -- 5SS30F Samples.

VARIABLE LIST and missing data values:

4 km	(-9999.00)	1 p	(-9999.00)	2 e	(-9999.00)
3 kcv	(-9999.00)				

OPTIONS:

NONE

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with p			
	Valid N	Spearman R	Z	signif.
Result	10	-.98473	2.95420	.00350

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with e			
	Valid N	Spearman R	Z	signif.
Result	10	.98788	2.96364	.00341

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with kcv			
	Valid N	Spearman R	Z	signif.
Result	10	.83030	2.49091	.01226

STATS+: NONPARAMETRIC STATISTICS 01-23-89 16:23:35

CORRELATION OF VOID RATIO, EXPANSION PARAMETERS, AND PERMEABILITY

main input data file is raw data file: a:triple.css

[17 cases with 4 variables]
Correlation of e, er, ef, and k-measured -- 10SS Samples

VARIABLE LIST and missing data values:

4 km	(-9999.00)	1 e	(-9999.00)	2 er	(-9999.00)
3 ef	(-9999.00)				

OPTIONS:

NONE

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with e			
	Valid N	Spearman R	Z	signif.
Result	17	-.70552	2.82209	.00502

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with er			
	Valid N	Spearman R	Z	signif.
Result	17	-.97589	3.90356	.00027

stats+ nonpar. stats.	SPEARMAN RANK ORDER CORRELATION Variables: km with ef			
	Valid N	Spearman R	Z	signif.
Result	17	-.91043	3.64172	.00054

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