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SLURRIES IN GEOTECHNICAL ENGINEERING

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Geotechnical engineering encompasses problems ranging from rock mechanics to slurry mechanics – and perhaps even one step further to suspensions. Most geotechnical engineers deal with the broad range of soils – sands, silts, clays, and mixtures thereof – between these two extremes, and they are typically concerned with designing and constructing foundations in or on these soils, tunnels through them, or earth dams with them. For reasons that were not part of a master plan, much of my career has been spent working with slurries or suspensions. When invited to prepare this talk, I chose this subject because it is one that the average geotechnical engineer has had relatively little opportunity to experience and hence might provide some interesting and thought provoking different outlooks. Following overviews of (a) how most industrial waste slurries are generated, (b) the material properties of slurried wastes, and (c) general approaches to the disposal of such slurries, the presentation will focus primarily on research studies undertaken at Northwestern University over the past four decades on different slurries and the major perspectives that have been gained. The intent is not to delve into the details of these various studies (these details have been explained at length in a few dozen theses and more than a hundred technical papers published in different journals and proceedings), but rather the objective is to simply provide enough background to enable one to “understand the general nature of the problem” and then to share as succinctly as possible the major findings in each case.

The slurries were very different in nature and the purposes of the studies were quite varied. The materials range from monomineralic clays to dredged materials to industrial wastes (flue gas desulfurization [FGD] sludge and bauxite residue [red mud]) to microfine cement grouts. A few other wastes, such as phosphate slimes and taconite tailings, were also investigated, but these studies were considerably less intensive than the foregoing studies and are not included here. Each of the five major studies discussed spanned a number of years (more than a decade in four cases) over the past forty years and involved quite a few talented students, who labored diligently and contributed many excellent ideas to enhance the value of the results, and their names are given in the acknowledgements.

Before getting into the specifics of the various slurries involved in these studies, it is useful to review some of the overarching principles that are applicable to all slurries and which will help to place in context the subset of slurries studied. Since many of the so-called waste slurries are generated by the processing of a parent ore, it is instructive to review the different processes most frequently employed, because in some cases changing the process might change the characteristics of the slurry and make it more amenable to handle. Next will be a presentation of reasonably well established correlations that have been developed for the engineering properties of a variety of slurries. Following these correlations will be brief discussions of the sedimentation-consolidation process and definitions of the rheologic parameters that govern the flow of slurries. Then, large-deformation mathematical models will be discussed because they lie
at the heart of predicting the magnitude and rate of settlement of these slurries when placed in a disposal area and subjected to self-weight consolidation (dewatering). And finally, various approaches and associated problems encountered when depositing these slurries in disposal areas will be reviewed. Other than waste slurries, geotechnical engineers are concerned with clay slurries (the formation of clay strata in a geologic setting and the laboratory preparation of reproducible clay samples for testing) and cement grouts. Although quite different in purpose, the same principles apply to these slurries.

**GENERATION OF WASTES**

Most slurried wastes are produced by processing an ore and extracting from it the constituents of interest (such as gold, copper, iron, aluminum, phosphate, etc.). To better understand the nature of the wastes produced, it is helpful to have a basic knowledge of how the ores are processed to remove the minerals of value. Although the extraction process may vary considerably with each individual ore – and even from plant to plant for essentially the same ore – certain steps in the process are fundamental for many ores (Vick, 1990; Krizek, 2000).

**Crushing and Grinding**

At the heart of most milling processes are crushing and grinding the ore, after which various different techniques are used to remove the minerals of interest. Crushing is usually performed in stages to reduce the rock to a size that can be fed into grinding equipment; secondary crushing reduces half-meter size fragments from primary crushing to particles on the order of 1 or 2 mm. Then, grinding primarily by rod mills or ball mills further reduces the size of the particles produced by crushing; since ball mills generally produce smaller particles, they are often used sequentially after rod mills to obtain a particle size gradation on the order of 0.5 mm or less. Grinding is the final operation in the physical reduction of ore, and the final gradation depends on the degree of particle breakdown in the hard rock and the clay content in the ore. For example, copper tailings often consist largely of silicate particles produced by grinding the parent rock, whereas phosphate slimes consist primarily of clay particles from the ore rather than particle breakdown from grinding. Particles produced from hard rock are usually hard and highly angular, while those from shales and similar materials with a high clay content have a hardness and shape compatible with the clay particles in the ore.

**Concentration**

Since the particles produced by grinding will vary in mineral content, the goal of concentration is to separate those particles with high mineral value from those with little or no value. This is usually accomplished by gravity separation, magnetic separation, or froth flotation. Gravity separation is normally performed with water and requires that the mineral and its parent rock have very different specific gravities; the desired particles (either lighter or heavier) are separated and collected, and remaining particles are discharged as tailings. Magnetic separation is used frequently for the extraction of iron
particles; the magnetic particles are collected via a belt or drum separator, and the nonmagnetic particles become tailings. Froth flotation is the most widely used concentration method; it is a highly complex physiochemical process in which mineral-bearing particles in a water suspension are made water repellant and receptive to attachment to air bubbles. Particles with higher mineral content then rise to the surface of a froth which is skimmed off, and the remaining particles become tailings. The chemicals used as flotation agents sometimes cause environmental problems with the tailings, such as cyanide used to froth the lead in galena.

**Leaching**

Leaching involves the removal of minerals from the ground particles of an ore by direct contact with a solvent – usually a strong acid or alkaline solution – and often leads to environmental problems. For example, acid leaching of uranium or copper oxide ores produces a tailings effluent with a pH in the range of 1 to 3, and sodium cyanide with lime as a pH modifier is a common reagent for extraction of gold and silver. Leaching may also change the physical characteristics of the tailings. As one example, the acid leaching of a uranium ore converted the montmorillonite clay minerals in the original ore to predominantly kaolinite in the tailings as the result of a calcium-sodium replacement.

**Heating**

Heating either the slurry suspension or the ground itself is sometimes used to extract minerals. Examples include the extraction of oil from oil sands and the production of phosphoric acid fertilizer from phosphate rock concentrate. The resulting tailings are handled in slurry form.

**Thickening**

The final step in the milling process is to remove some of the water from the tailings-water slurry before it is discharged into the disposal area. This is commonly done with thickeners, which consist of a tank with rotating arms that convey the settled solids to the center of the tank where they are collected and pumped to the disposal area. This thickening process is sometimes accomplished with hydrocyclones or with drum or belt filters, which use vacuum suction to dewater the slurry through a cloth or screen. After the thickened tailings are transported in slurry form and deposited in an impoundment area, the coarser particles settle from suspension and the supernatant water with reagents and colloidal particles is collected, whenever possible, and returned to the mill for reuse as process water. However, the presence of contaminants sometimes precludes recirculation because it would reduce the extraction efficiency. What is left in the disposal area is a high water content soil-like waste material which must be dewatered in some fashion.
Cleaning Processes

In some cases slurry wastes are generated by a cleaning process. For example, about 70 percent of society-generated sulfur dioxide emissions in the United States come from coal-burning, electric-power generating plants, with the rest coming from industrial sources and residential/commercial heating systems. Depending on the combustion process, sulfur cleaning methods can be generally categorized into three groups: (a) before combustion (coal cleaning, coal liquefaction, and gasification), (b) during combustion (fluidized bed combustion), and (c) after combustion (flue gas desulfurization).

Coal Cleaning

Coal contains sulfur in two forms: inorganic mineral sulfur in the form of pyrite and organic sulfur which is chemically bound to the coal. Most of the mineral sulfur can be removed by mechanical coal cleaning processes, but the removal of organic sulfur requires chemical processing. In physical coal cleaning the coal is crushed, washed, and then separated from the impurities by a settling process and the sulfur is removed with the impurities. The chemical coal cleaning process involves methods which are still in their developmental stage.

Fluidized Bed Combustion

Fluidized bed combustion involves the combustion of pulverized coal in a bed of crushed limestone or dolomite while air is forced upward to cause the bed of coal and sorbent to become suspended and move in a fluid-like motion. As the coal burns, the SO2 is absorbed by reacting with the limestone, and the spent sorbent (waste sulfate) and ash are continuously removed.

Flue Gas Desulfurization

The most common technology to control SO2 emissions are a variety of flue gas desulfurization (FGD) processes. FGD systems can be classified as (a) non-regenerable or throwaway systems, in which the sulfur material generated through scrubbing or absorption is disposed of as a waste product, and (b) regenerable or recovery systems, in which the sulfur materials, such as elemental sulfur, liquid SO2, and sulfuric acid, are marketed as salable products.

MATERIAL PROPERTIES OF SLURRIED WASTES

This section summarizes much of the available data on the engineering properties that govern the post-depositional behavior of slurried wastes. Indices and properties are compared for a variety of materials over ranges that extend well beyond those normally encountered in conventional geotechnical engineering practice. Virtually all finite-strain theories describing the consolidation and desiccation of these wastes are strongly dependent on the determination of an “initial” or “zero effective stress” void ratio and
relationships among the effective stress, void ratio, and permeability. In their excellent paper on the design capacity of slurried waste ponds, Carrier, Bromwell, and Somogyi (1983) presented a comprehensive set of figures and equations that describe these relationships well. In some cases, classical small-strain consolidation theory can be used advantageously, and guidelines for estimating the compression index and coefficient of consolidation are discussed.

**Sedimentation – Consolidation**

When a thin slurry or suspension is deposited in a containment area, Imai (1981) has defined the ensuing densification process to occur in three stages (floculation, sedimentation, and consolidation), as follows:

“In the first stage, no settling takes place, but flocculation yields flocs. In the second stage, the flocs gradually settle and form a layer of sediment, which undergoes consolidation and reduction of water and the sediment is the birthplace of new sediment. While the sediment grows, the settling zone becomes thinner and finally vanishes. In the last stage, all of the sediment thus formed undergoes self-weight consolidation and finally approaches an equilibrium.”

A descriptive illustration of this process is given in Figure 1. The duration of each of these stages is highly dependent on the characteristics of the slurry or suspension, and the demarcation between the stages is usually not able to be well defined from a practical point of view.

![Figure 1. Illustration of Flocculation, Sedimentation, and Consolidation Stages (after Imai, 1981)](image-url)
Plasticity

Figure 2 shows a plasticity chart with approximate ranges for the liquid limit and plasticity index for a wide variety of clays and slurried wastes, as well as several naturally occurring clays and specifically prepared samples of calcium and sodium montmorillonite. As can be seen, the values for most of these materials plot slightly above the A-line in the zone typically characteristic of highly compressible materials. In addition to the wastes already discussed, including a variety of dredged materials, this figure contains data from oil sand sludges and China clay tailings. The oil sand sludge consists primarily of kaolinite clay with some smectite, together with some fine silt and residue bitumen, and the China clay tailings (from the manufacture of fine china) consist of micaceous silt and are generally nonplastic. The Maumee River dredgings and the FGD sludge tend to be “rather silty”, and the applicability of the Atterberg tests is questionable, thus suggesting a reason for their position below the A-line. The red muds, which also plot below the A-line, contain little to no mineral components. Dredged materials from a given location tend to exhibit little variability, but taken collectively, their range becomes quite large; note in particular, the differences between fresh water and sea water dredged materials.
Initial Void Ratio

When considering a dilute slurry of suspended particles, the term “void ratio” which may be on the order of 50 or 100 makes little sense, and a preferred term is “solids content.” The term “void ratio” becomes meaningful only when the slurry suspension becomes a “soil” or “soil-like material.” This occurs at the end of sedimentation and the onset of self-weight consolidation, or when the particles come into contact with each other and initiate the transfer of effective stress; this value is termed the “zero effective stress” void ratio, and it provides the origin for measuring strains in the material of any point during its consolidation history. Although simple to define conceptually, it is difficult in practice to determine when this situation actually occurs, and the selection of a “zero effective stress” void ratio is somewhat arbitrary. This void ratio can be determined experimentally by measuring the void ratio of a sample (a) recovered from the surface after sedimentation is considered to be complete or (b) sedimented in a container from a slurry having an initial solids content similar to that in the field. Alternatively, Carrier, Bromwell, and Somogyi (1983) suggest that the water content corresponding to the value of the “zero effective stress” void ratio is about seven times the liquid limit, and pursuant to a study of large-strain consolidation of kaolin slurries, Monte and Krizek (1976) suggest five times the liquid limit. Some typical values for the initial void ratio of a few wastes are about 10-12 for red mud and FGD sludge, 10-20 for freshwater dredgings, and 15-30 for sea water dredgings and phosphate slimes. Although the choice of a value for the “zero effective stress” void ratio may affect the predicted time to complete a given amount of consolidation by weeks or months relative to the total consolidation time of many years, it has little effect on the calculated storage capacity of an impoundment area because the permeability and associated rate of consolidation are very high for high void ratios and decrease quickly as the void ratio decreases.

Compressibility

The compressibility of various fine-grain wastes and a few clays for comparison is summarized in Figure 3. These materials span an enormous range in compressibility with void ratios from about 20 to less than 1, and many involve material behavior at much lower effective stresses and much higher void ratios than normally encountered in geotechnical engineering. Some of the available mathematical models can accommodate experimental compressibility data (i.e. observed void ratios at known effective stresses) directly and employ various interpolation schemes to calculate the appropriate compressibility. However, most models require that some form of mathematical relationship be specified. Monte and Krizek (1976) used a power relationship of the form $\sigma = Me^N$, where $\sigma$ is the vertical effective stress, $e$ is the vertical strain, and $M$ and $N$ are empirical coefficients. According to Carrier, Bromwell, and Somogyi (1983), the compressibility of many wastes can be approximated by $e = A\sigma^B$, where $e$ is the void ratio and $A$ and $B$ (which is negative) are empirical coefficients that depend on the particular material and vary over a wide range. For high values of stress, this equation predicts unreasonably low values for the void ratio and is therefore invalid; in this case, the more conventional compression index, $C_e$, tends to govern the
compressibility. From this equation, it follows that the coefficient of compressibility, $a_v$, is given by:

$$a_v = -\frac{de}{d\sigma} = -AB\sigma^{B-1}$$

where $a_v$ is always positive (since $B$ is always negative) and varies greatly (perhaps three orders of magnitude or more) with effective stress and type of material. The most useful laboratory device for measuring the compressibility of high void ratio sediments is the slurry consolidometer (Sheeran and Krizek, 1971). Although the compressibility of a material is sufficient to predict the final volume, the time rate of volume change requires information on the relationship between the permeability and the void ratio.

![Figure 3. Compressibility of Fine-Grained Mineral Waste Materials and Remolded Clays (after Carrier, Bromwell, and Somogyi, 1983)](image)

**Permeability**

Except for a few models that either incorporate experimental data or use the coefficient of consolidation, all others require a relationship between the permeability and the void ratio. Of all the components that comprise the problem under consideration, the permeability relationship is arguably the most important and the most difficult to quantify. Permeability values for a wide range of waste materials and two montmorillonite clays are summarized in Figure 4. Note that this graph spans eight orders of magnitude for the permeability from essentially impermeable to approximately the permeability of silt, with the waste materials spanning somewhat more than four
orders of magnitude. Among the many empirical relationships used for the permeability are: \( k = (1 + e)(S + Te) \) (Monte and Krizek, 1976); \( k = Ce^D \) (Somogyi, 1979); and \( k = Ee^F/(1+e) \) (Carrier, Bromwell, and Somogyi, 1983), where \( e \) is the void ratio and \( S, T, C, D, E, \) and \( F \) are empirical constants. If laboratory permeability tests are conducted, sufficiently small gradients must be used to avoid seepage-induced consolidation during the test process.

Figure 4. Permeability of Fine-Grained Mineral Waste Materials and Remolded Clays (after Carrier, Bromwell, and Somogyi, 1983)

Alternatively, Znidarcic (1982) presented a method for estimating the permeability from constant rate of deformation slurry consolidation test results, and Huerta, Kriegsmann, and Krizek (1988) presented a means for back-calculating the permeability from the results of seepage-induced consolidation tests.

Parameters for Classical Consolidation Theory

The magnitude and rate of consolidation of some tailings can be predicted reasonably well by use of classical Terzaghi consolidation theory. For these cases the compression index, \( C_c \), usually lies in the range from 0.1 to 0.3 (although values from 0.05 to 0.4 have been measured) and the coefficient of consolidation, \( c_v \), may vary over several orders of magnitude. While these ranges are similar to those exhibited by many natural clays, the values for tailings usually manifest little consistency in their variation with void ratio, \( e \). Guidelines for the estimation of \( c_v \) and \( C_c \) values may be obtained from Figure 5 (Vick, 1990) and Figure 6 (Krizek, Parmelee, Kay, and Elmaggar, 1971), respectively. The consolidation relationships in Figure 5 represent a variety of different tailings and convey an appreciation for the variability involved. Although the phosphatic clays show an essentially constant value for \( c_v \) with large changes in void ratio, the extremely high void ratios suggest that a finite strain formulation utilizing a relationship of the form \( \bar{\sigma} = f(e) \), instead of \( c_v \), would be preferable. The compressibility data in Figure 6 were obtained from a variety of inorganic and organic clays and silty soils, and the indicated straight-line regression equation was chosen to best describe those data with an initial void ratio less than two.
Figure 5. Variation in Coefficient of Consolidation with Void Ratio (after Vick, 1990)

Figure 6. Compressibility Factor versus Initial Void Ratio (after Krizek, Parmelee, Kay, and Elnaggar, 1971)
Rheologic Parameters

Slurries and suspensions are often modeled as Newtonian or Bingham fluids (in reality most are neither), the mechanical behaviors of which are depicted by Curves A and B in Figure 7. The viscosity is defined as the slope (coefficient of proportionality) of the tangent or secant to the curve of shear stress versus strain rate. In the case of a Newtonian fluid, the behavior is idealized as a straight line through the origin, and the so-called dynamic viscosity, $\eta_d$, is constant for all strain rates. The somewhat more general behavior of a Bingham fluid is represented by a sloped straight line starting at some yield stress, $\tau_y$; this implies that no flow takes place until the shear stress reaches some yield value, after which flow occurs at a constant viscosity, termed the plastic viscosity, $\eta_p$. The behavior of actual slurries and suspensions may be better approximated by a pseudo-Bingham fluid for which the constitutive response usually follows a curved path depicted by Curve C in Figure 7; thus, the viscosity (slope of the tangent to the curve) varies with strain rate, and the yield stress is not well defined. However, to provide a basis for comparison, the apparent viscosity, $\eta_a$, at a given strain rate is taken as the slope of the secant between the origin and the point on the curve corresponding to the given strain rate; the plastic viscosity is taken as the slope of the more-or-less linear portion of the curve for high strain rates; and the yield stress is taken as the point where the projection of this linear portion of the curve intersects the shear stress axis.

Figure 7. Definition of Viscosity and Yield Stress Parameters
Consolidation

Although analytical models comprise the heart of most predictive techniques for the rate of settlement, the physical process to be simulated is often not adequately understood, and the complexity of the governing natural phenomena usually dictates the incorporation of several simplifying assumptions to achieve mathematical tractability. The major assumptions in conventional consolidation theory (namely, small strains, constant material properties, and no self-weight) are generally recognized as excessively restrictive for analyzing the large volume changes observed in slurried waste materials. The two most popular approaches to circumvent this restriction utilize either “incremental small strain” or “finite strain” formulations (Been and Sills, 1981; Schiffman, Pane and Gibson, 1984; Krizek and Somogyi, 1984; Toorman, 1996).

**Incremental Small Strain Models**

Incremental small strain models maintain the simplicity of the classical Terzaghi-type formulation and constitute a logical next step to extend its applicability. The solution usually involves dividing a deposit into a series of layers, each with different material properties, and satisfying continuity at the interfaces between layers. These models can handle nonuniform material properties (whether caused by dissimilar materials, self-weight, or other temporal or spatial variations in effective stress), but the need to continuously update both the material properties and the positions of the layers makes the solution computationally laborious.

**Finite Strain Models**

Finite strain formulations are usually based on the pioneering work of Gibson, England, and Hussey (1967) and result in nonlinear second-order partial differential equations. Self-weight and material nonlinearities can be directly incorporated into the equations, which are usually developed in terms of material, rather than spatial, coordinates. This formulation has significant mathematical advantages because it transforms a boundary value problem with a moving boundary whose location is unknown into one with a fixed known boundary (no filling) or one in which the boundary location is always known (during filling or accretion).

**Mathematical Techniques**

In the finite strain models, the equation describing the consolidation process is cast in terms of either void ratio or pore pressure. The void ratio based models assume that both effective stress and permeability can be described as functions of void ratio, whereas the pore pressure based models assume that the void ratio depends on the effective stress and the permeability depends on the void ratio. Explicit numerical techniques advance the solution from known (previously computed) values of the dependent variable and corresponding material properties, but stability and convergence criteria are often unattainable for nonlinear equations and can only be deduced after appropriate linearization of the equation. On the other hand, implicit numerical
techniques are computationally more efficient and unconditionally stable for linear equations, but criteria for their stability and convergence may be equally difficult to prove for highly nonlinear problems; in addition, a major limitation for nonlinear problems is that the equations require material properties corresponding to unknown values of the dependent variable (i.e. at the “new” time step), so these must be either estimated or allowed to lag one time increment behind the solution.

**Seepage-Induced Consolidation**

Because the changes in void ratio and permeability with effective stress are very large in the early stages of consolidation, the effect of seepage-induced consolidation can be important. In particular, seepage-induced consolidation makes it difficult, if not impossible, to measure the material properties in this region of low effective stresses, because the very process of applying a hydraulic gradient to measure permeability results in a varying effective stress and a resulting nonuniform void ratio in the direction of the gradient. To address this problem, several seepage-induced finite strain consolidation models have been developed over the years (Imai, 1979; Huerta, Kriegsmann, and Krizek, 1988; Abu-Heijleh, Znidarcic, and Barnes, 1996; Fox and Baxter, 1997). Ideally, such a model can employ known or assumed material property relationships to determine the final thickness of a sedimented slurry subjected to a constant piezometric head, or alternatively the inverse solution can use the final settlement, steady-state flow data, and void ratio of the solids at the bottom of the laboratory sample or field layer to deduce permeability and compressibility relationships. One point of interest is that the permeability influences both the required time to reach a steady-state condition and the steady-state itself (i.e. the final height of the consolidated deposit depends on the variation of the permeability with the void ratio).

**Deviations from Primary Consolidation**

“Secondary” consolidation may be regarded as simply that portion of the settlement response that is not described by some appropriate theory of “primary” consolidation; similarly, any deviation from the straight-line relationship usually observed between the volume change and the logarithm of time in “secondary” consolidation may be termed “tertiary” consolidation. However, there is no implication that settlements due to secondary or tertiary consolidation are of negligible magnitude. Within the context of this definition, there would be no secondary or tertiary consolidation if a sufficiently descriptive primary consolidation theory were available. Notwithstanding this idealized concept, the unavailability of the requisite comprehensive theory dictates that consolidation data be interpreted in accordance with the traditional concepts of primary and secondary consolidation, with any deviations from secondary being termed tertiary. Figure 8 (typical for specimens of dredged material loaded for more than 200 days) shows clearly that some materials do exhibit significant “tertiary” consolidation (Salem and Krizek, 1975). It can be seen in Figure 8 that a so-called “standard” primary-secondary response curve was measured for about a week, after which the slope of the e-log t curve increased substantially for about two months and then began to decrease. Similar response curves were discussed by Lo (1961) for clay soils.
APPROACHES TO DISPOSAL SOLUTIONS

The following sections identify and discuss briefly many of the concepts that have been used with success and that should be considered when challenged with a disposal problem involving waste slurries. Only a few of the suggested ideas will usually be applicable in any particular situation, but this decision should not normally be made until each has been considered explicitly.

Process Modification

When addressing a problem of waste disposal, often not considered in sufficient detail are the modifications that might be made in the process by which the waste is generated. This is especially true for geotechnical engineers, because they are usually too quick to accept the wastes in whatever form the chemical or mechanical engineers in charge of the beneficiation process at the plant provide them. Oftentimes, however, a frank discussion by all concerned will reveal that some modification in the waste generation process at the plant will improve considerably the handling and disposal properties of the waste materials generated. For example, cyanide control is a major problem in ore deposits developed for precious metals, and increased emphasis is being placed on pretreatment and destruction of the cyanide prior to discharging the tailings into an impoundment. In some cases, chemicals can be added near the end point in the process to neutralize the waste or enhance sedimentation, vacuum filtration can decrease the water content, or perhaps fly ash should not be removed because its presence in the waste can be beneficial (Krizek, Christopher, and Scherer, 1980; Krizek, Chu, and Atmatzidis, 1987).
Dike Design

The design and construction of the dikes for an impoundment area usually comprises a substantial portion of the cost of the disposal operation. Accordingly, it is always desirable and economical to use the waste material, if possible, to construct the dikes. Although the waste materials are usually not ideal for this purpose, they have been used in many cases, and several excellent references, such as that by Vick (1990), are available to provide guidance. Surface impoundments can normally be categorized as water-retention dams and raised embankments. Water-retention dams are best suited for tailings impoundments with high water storage requirements; they are generally quite similar to conventional water storage structures and are usually constructed to their full height prior to the placement of tailings in the impoundment. Alternatively, raised embankment dams are staged over the life of the impoundment, beginning with a starter dike to contain the tailings for the first few years and then adding additional height on an as-needed basis. This scheme has significant advantages in distributing the costs over the life of the project (this is especially important in a start-up operation) and in providing flexibility in sizing the impoundment and choosing the materials (such as selected wastes from the mining operation itself) to construct subsequent stages.

Liner Design

Liners are inherently high cost and are usually used only in situations involving high toxicity wastes and stringent groundwater protection requirements. Typical liners consist of clay or various geosynthetic membranes, but some wastes have a sufficiently low permeability to possibly serve as a liner material, thereby enhancing the economics of the disposal process. For use as a liner, the waste must have about 40% or 50% passing the No. 200 sieve, and cycloning must usually be used to separate this fraction from the overall tailings. In other cases the waste must be placed and compacted at or near optimum water content to serve effectively, and this usually precludes their use because it is difficult to adequately dewater them. In addition, some waste materials placed in such a condition are brittle, and the liner might crack and become ineffective with differential settlements.

Drainage Blanket

Since the rate of dewatering is strongly dependent on the length of the drainage path for the escape of water, it may be highly advantageous in some special cases to install a drainage blanket and associated dewatering system at the bottom of an impoundment area prior to placing any wastes. Many wastes contain significant quantities of sand which can be separated and used to construct such a drainage blanket. For the past two decades or so, a wide variety of geosynthetic materials (albeit rather expensive) has been available to serve as an underdrain system for an impoundment area, but little can be done to install geosynthetic drains after an area is partially filled, because the materials are too soft to traverse. A partial vacuum in this drainage blanket would enhance the dewatering process considerably, especially during the early stages.
Sedimentation

When a low solids content waste is pumped into an impoundment area, the first physical process that occurs is sedimentation – presumably in conformance with Stoke’s law or some modification thereof. Accordingly, the particle size distribution of the solids in the suspension controls the rate of sedimentation. In a conventional hydrometer test, a dispersing agent is usually added to the suspension to prevent flocculation. However, such tests do not yield the proper particle size distribution for use in predicting the rate of sedimentation in the field, because a dispersing agent is not normally added to the slurry. Alternatively, a flocculating agent may be added to the waste slurry at some point in the generation and disposal process to enhance flocculation, accelerate the settling rate, and clarify the effluent.

Retention Time

Very often the supernatant water from the sedimentation in an impoundment area flows over a weir into a water course or is collected via a drain system for reuse as process water. The degree of clarity that is achieved in the supernatant water depends on the time available for the particles to settle from suspension, and the disposal area must be designed to provide a retention time commensurate with the water clarity desired. Sometimes this is handled by a successive series of ponds with each clarifying further the discharge from the preceding pond. Too short retention times for an impoundment area will result in too many solids, as well as pollutants adhering to the solids, in the discharged supernatant water. Guidelines for the design of impoundment facilities as solid-liquid separation systems have been presented by Krizek, Fitzpatrick, and Atmatzidis (1976).

Filters to Maintain Water Quality

Sometimes the discharged supernatant waters are passed through a filter to achieve further clarification. In some situations large portions of the surrounding dikes can be designed as filter elements to enable the discharge of large quantities of supernatant waters while maintaining the water quality. For lower flows filter “cartridges” may be installed at the overflow weirs to accomplish this purpose, but they would have to be exchanged periodically to avoid excessive clogging and decreased flow. Krizek, Fitzpatrick, and Atmatzidis (1976) have developed criteria for designing such filter systems.

Thickened Discharge

The thickened discharge method of tailings disposal originated from a concept advanced by Shields (1974) and developed by Robinsky (1979). The objective is to thicken the tailings-water mixture such that it behaviors more like a viscous mud than a liquid slurry. Under such a condition the discharged material will form a conical pile with side slopes of a few percent, as illustrated in Figure 9, thereby allowing any free water to readily flow to the toe of the slope where it is collected and removed.
Advantages of this method are the elimination (or significant reduction) of impoundment dams, reduction in pumping return water, reduction of seepage (because there is no decant pond), virtual elimination of embankment failure under static loading conditions, and simplification of the reclamation process. The cost savings realized by eliminating the impoundment dams may be largely offset by the higher costs for thickener construction and operation and the greater expenses associated with pumping. In many cases more surface area may be disturbed than for conventional impoundments, resulting in larger areas to be reclaimed. If not collected and diverted around the pile, runoff at the top of the slope may cause erosion and the transport of tailings. In addition, a dynamic excitation may cause a liquefaction flow slide in the lower (saturated) portions of the pile. Both liquefaction and runoff-handling problems become more severe if the thickened tailings are deposited on top of a conventional impoundment to augment its capacity. This method of disposal is best suited for materials containing a reasonable sand fraction without a major proportion of clayey fines in relatively flat topography at sites close to the plant (to minimize pumping costs) in areas of low seismicity.

![Diagram of Thickened Discharge Disposal Method](after Robinsky, 1979)

**Dry Stacking**

Dry stacking is a modified version of the thickened discharge method described above and is suitable in areas where the evaporation rate is high. In a typical scenario a dike encircles the impoundment area, and a pipeline with discharge ports every few hundred feet rests on the dike. Disposal begins by opening one of the discharge ports and
allowing the thickened slurry to form a “tongue” extending at a slight slope from the dike toward the center of the impoundment area. After the slurry “tongue” reaches a depth of perhaps one meter near the dike and extends a hundred meters or so toward the center of the area, the discharge port is closed, an adjacent discharge port is opened, and the process is repeated. This process continues for perhaps 30 or 40 discharge ports, each for a day or two, around the pipeline. By the time one cycle is completed, the “first tongue” of material deposited should ideally have desiccated to a rather low water content, at which time a “new tongue” of material is placed over the first, and so forth. The major limitation of this method is that the climate of the region must be such that each layer will dry sufficiently in the time available before an overlying layer is placed (Palmer and Krizek, 1987).

**Dry Disposal via Vacuum Filtration**

In an even more extreme case of reducing the water content of the waste at the plant, the dry disposal method endeavors to remove much of the water from the waste material before it is placed in an impoundment area. This is usually accomplished by vacuum filtration using a drum or belt, and the tailings come off the drum or belt as a relatively easily handled “dry cake.” However, there is considerable controversy over the feasibility, advantages, and economics of vacuum filtration. The nature of the material will affect substantially the efficiency of the process, and it may not work at all for some materials with a high clay content. Since both capital and operating costs for vacuum filtration are very high, the method can usually be justified only when the filtration is an integral step in the ore processing operation, but not when it is a supplemental dewatering method added to conventional thickeners. If dewatered in this fashion, the tailings can usually be placed and compacted in a disposal area in essentially solid form, and reclamation can proceed concurrently. Although the tailings are in “solid” form when handled and placed, their water content is often around 20% to 30%, which is usually higher than the optimum water content for compaction and will result in near saturation for some materials when placed at typical void ratios. Accordingly, the seepage from the saturated tailings may still be significant in the absence of liners or underlying natural materials of low permeability.

**Evaporation and Crust Formation**

The loss of water from the surface of a tailings pond is a two-stage process. In the first stage the conductivity of water is sufficiently large that water loss is controlled by the prevailing climatic conditions (radiation, air temperature, wind speed, humidity, etc.), and results are comparable to those measured in pan evaporation tests. This stage has been found (Brown and Thompson, 1977) to govern the response until the water content reaches a value slightly below the liquid limit. During the second stage, the converse is true, and evaporation losses are essentially independent of the environment. However, the presence of a water table near the surface may provide a sufficiently large supply of water to continuously rewet the surface and preclude the evaporative process from entering the second stage. In any case, when the overlying free water is decanted, evaporates, or drains through the layer into the underlying soil, the surface begins to
desiccate and a crust is formed. The formation of a crust has both advantages and disadvantages. One of the advantages is that it provides, in some cases, a layer with adequate strength to support workers and equipment. A second advantage is an increase in the dry density with a resulting settlement of the surface and increase in the storage capacity of the impoundment. A major disadvantage of a crust is that it greatly inhibits subsequent evaporation because the partially saturated surface layer has a drastically reduced permeability. Nature often provides the first solution to this problem by the development of “alligator” cracking as shrinkage occurs in the desiccating crust; this provides drainage channels for the horizontal movement of water and additional surface area for evaporation. When attempting to analyze the volume change of a partially saturated medium, a dichotomy is encountered. Geotechnical engineers usually assume 100% saturation and a deforming medium, whereas soil scientists usually assume that flow takes place in a partially saturated medium with a rigid structure (no volume change). This physical process of desiccation and crust formation was incorporated as an integral part of a mathematical model developed by Casteleiro, Krizek, and Edil (1981); solutions obtained from this model are able to (a) describe the water content distribution in the fill at any time after deposition, (b) predict the desiccation and consolidation behavior as a function of time, and (c) aid in evaluating different techniques for accelerating the dewatering process. An improved desiccation theory by Abu-Hejleh and Znidarcic (1995) includes, in addition to one-dimensional consolidation, desiccation under one-dimensional shrinkage, propagation of vertical cracks and tensile stress release, and desiccation under three-dimensional shrinkage. It has been found that slow evaporation rates form a thicker crust with more widely spaced cracks. In addition to evaporation, transpiration has an important effect on the dewatering process during the early stages of desiccation, but this effect tends to disappear as the water table approaches an equilibrium position.

Agitation

To overcome the impedance of a crust to the evaporation process and consequent dewatering of the wastes, various techniques have been utilized to agitate or break up the crust (Brown and Thompson, 1977; Haliburton, 1977). Since it is usually difficult, if not impossible, for equipment with any substantial weight to traverse the soft sediments, disc-like or plow-like tools are often pulled across the surface by cables operated from the tops of the impoundment dams. However, even under the best of circumstances, this process enhances desiccation for only a few feet of depth and its cost-benefit relationship must be evaluated carefully. Another limitation is that the climate must have a favorable evaporation rate.

Vegetation

For some types of materials conducive to the growth of vegetation, near surface dewatering can be accomplished by transpiration through the leaves and associated root systems of appropriate vegetation. Plants transpire large quantities of water during the growing season; the rate of water loss may exceed that of free water evaporation and continue long after the surface has become dry. In favorable climates and for wastes with
favorable composition (texture, fertility, and toxicity), the colonization of the site by volunteer vegetation or selected vegetation may occur by natural processes in a short period of time. However, if the waste is acidic or highly saline or contains high concentrations of heavy metals, the establishment of vegetation may be a lengthy, difficult, and costly process – and ultimately not justify the effort involved. As with agitation of the crust, dewatering by vegetation is limited in the depth of its influence (and evapotranspiration effects are much more complex to model than evaporation alone). However, vegetation can assimilate minerals and various organic toxic compounds, and much of this material can be removed by timely harvest. In the reclamation operation, those wastes incapable of sustaining vegetation must be covered with a layer of topsoil that is conducive to fostering vegetation (Vick, 1990).

**Surcharging**

Surcharging offers a time-tested procedure for accelerating the consolidation and dewatering process and increasing the rate of strength gain, but the low shear strength of many high water content waste materials usually makes it difficult to apply the surcharge without causing a stability failure and consequent mud wave. The use of geosynthetics can help considerably in preventing a mud wave, but extreme care must still be exercised in adding the surcharge slowly. The cost of the geosynthetic materials, the expense in applying and removing the surcharge, and the long times involved in the process preclude the use of this procedure except for special situations. The use of wicks to accelerate drainage can reduce substantially the time required, but there is usually a problem for the equipment to access the site; often a few feet of surcharge is necessary to form a “pad” on which the equipment can operate. If a drainage blanket or system of geosynthetic drains is installed at the bottom of the impoundment area, the dewatering process can be accelerated by applying a partial vacuum to the system, which enables advantage to be taken of atmospheric pressure as a surcharge load. While generally functional in principle, cost and time usually limit the use of this method.

**Spraying and Sandwiching**

Spraying involves using a floating pipeline equipped with spray nozzles to sprinkle sand tailings over the surface of the soft tailings (perhaps 10% to 15% solids). The sand slowly sinks through the tailings to form vertical channels (sand drains) which, together with the added overburden, serve to relieve pore pressures and accelerate dewatering; the result is a drier and stronger upper layer to support additional thicknesses of sand. This operation can be alternated with layers of tailings to form a sand-tailings sandwich effect (Bromwell and Oxford, 1977).

**Electro-osmosis**

Many studies have been done to investigate the effectiveness and feasibility of electro-osmosis as a means of dewatering high water content materials. The first limitation is the relatively narrow range of materials (primarily silts) over which it is effective, and the second limitation is that, except for certain very special situations, the
power consumption and associated cost are prohibitive. Even for one study where windmills were used to generate the electricity at a remote location, the use of electro-osmosis was not judged to be feasible.

Storage Capacity

The storage capacity of a given impoundment area depends to a large extent on the final dry density of the slurry placed within it; however, since it may require years to achieve the final dry density, a knowledge of the increase in dry density with time will aid in sizing the disposal area. Typical in-place dry densities of various slurries shortly after disposal range from 15 to 20 pcf (2.4 to 3.1 kN/m$^3$) for phosphate slimes and red muds to 50 pcf (7.9 kN/m$^3$) for dredged materials and FGD sludges to more than 100 pcf (15.7 kN/m$^3$) for taconite and copper tailings.

Reclamation

Current environmental regulations dictate that virtually all disposal sites must be reclaimed. While most of the preceding discussion has dealt with aspects of the disposal process during the period when the wastes were being deposited, closure is not possible until the materials deposited are considered to be permanently stable and environmentally innocuous. Toward this end there are many cases where more effort has been expended on remedial groundwater and toxicological studies for abandoned deposits than was ever allotted in the original design and operation of the site; similarly, there are cases where the costs for permanent stabilization of abandoned tailings are orders of magnitude greater than the value of all the ore produced. However, because of the land areas disturbed and the varying (and sometimes unknown) toxicities of the wastes involved, tailings impoundments usually provide the focal point for public opposition to mining projects. Since there have been few serious efforts at reclamation to date, there is little experience by which to judge the long-term success of such endeavors. Measures of success must be judged by the degree to which the following fundamental objectives are satisfied (Krizek and Atmatzidis, 1978; Vick, 1990):

- Long-term mass stability of the impoundment
- Long-term erosion stability
- Long-term prevention of environmental contamination
- Eventual return of the disturbed area to productive use

Typical costs to accomplish reclamation may be on the order of several thousand dollars per acre, and government authorities commonly require the posting of a reclamation bond at the outset to ensure that appropriate reclamation efforts are carried out after termination of the disposal operation. While “productive use” is a generally accepted goal, the term means “different things to different people,” and the specific definition influences considerably the manner in which the area is reclaimed. Generally, “productive use” is defined in the context of land use patterns that existed prior to the establishment of the disposal area.
CLAYS

The directionally-dependent mechanical properties of clay soils are governed to a large extent by their “structure” or “microstructure”, which consists of complex combinations of particle arrangements and interparticle bonds. The geometrical aspects of these particle arrangements at a microscopic level are usually termed “fabric” or “microfabric”, and fabric together with the time-dependent interparticle bonds that form is termed structure. Although structure is actually the primary characteristic that controls the engineering behavior of clay soils, fabric is an important component of structure and is much more amenable to study and evaluation; accordingly, the effect of fabric was investigated as an initial step to aid in understanding the overall anisotropic behavior of clay soils. Depending on the physical and chemical conditions that prevail during deposition, an “intrinsic fabric” is imposed on natural clays; in general, this intrinsic fabric may range from highly random (flocculated) to highly oriented (dispersed). Subsequent to deposition, the application of stresses (due to overburden or external loading in the field or a system of stresses in the laboratory) causes a “stress-induced fabric”, which represents a change in the intrinsic fabric.

Overview of Study

Summarized here are the major results of a long-term research effort to relate in a quantitative manner the microfabric of a kaolin clay and its mechanical properties. This was accomplished by preparing block samples with preconceived fabrics that were controlled by appropriate combinations of pore fluid chemistry and consolidation stress path. Flocculated and dispersed slurries at several times the liquid limit were consolidated either isotropically or anisotropically to various values of the maximum effective consolidation stress to obtain a wide range of fabrics extending from highly random to highly oriented. A comprehensive appraisal of the clay fabric at its various levels of organization was obtained by examining each sample with the combined use of scanning electron microscopy, optical microscopy, and X-ray diffractometry. During the entire process of anisotropic slurry consolidation, pore pressures and lateral stresses were measured. Upon completion of anisotropic slurry consolidation and while the clay samples were still subjected to their ambient consolidation stresses, horizontally and vertically oriented vane shear tests were conducted. Then, the consolidation stresses were released from both isotropically and anisotropically consolidated samples, the clay blocks were removed from the slurry consolidometers, mutually orthogonal specimens were trimmed from the blocks, and a laboratory program of creep tests, triaxial tests, and uniaxial compression tests was conducted to investigate the directional dependence of the mechanical properties. Several different mathematical models were developed to characterize various aspects of the experimentally observed behavior.
Sample Preparation

Approximately twenty block samples (with final dimensions on the order of 10 to 20 cm and a final water content of approximately 50%) of water-washed kaolin clay (Hydrite 10, marketed by the Georgia Kaolin Company) were prepared by consolidating high water content (about 250%) slurries either isotropically in a flexible balloon-like container with diametrical drainage (Edil and Krizek, 1976) or anisotropically in a slurry consolidometer (Figure 10) with vertical drainage (Sheeran and Krizek, 1971) to various principal consolidation stresses up to a maximum of 1760 kN/m$^2$. The kaolin clay has a Liquid Limit of 62% and a Plastic Limit of 28% when mixed with distilled water, and its particle size distribution is characterized by 95%, 80%, 40%, and 10% of the particles having an equivalent diameter of less than 2, 1, 0.5, and 0.2 microns, respectively. Predetermined combinations of slurry chemistry (flocculated versus dispersed) and consolidation stress path (isotropic versus anisotropic) were used to obtain samples with a wide range of intrinsic microfabrics extending from highly random to highly oriented particle orientation distributions (Krizek, Edil, and Ozaydin, 1975).

Figure 10. Slurry Consolidometer
**Consolidation Model**

A mathematical model (Monte and Krizek, 1976) was developed to describe the one-dimensional large strain consolidation of a clay slurry. As discussed previously, the first challenge in the development of such a model is to determine the “starting point” or reference state at which the model becomes applicable. If the water content of a clay soil is increased beyond its Liquid Limit, the shear strength will essentially vanish at some water content which may be postulated to reflect the “stress free” state or “zero effective stress” void ratio of the clay-water system. In this study the water content at which a “slurry” transitions into a “soil” was found to be about five times the Liquid Limit – and was termed the Fluid Limit. Accordingly, the Fluid Limit is considered to be that water content associated with a “stress free” condition of the soil, and it is taken as the reference state from which strains are measured. The resulting boundary value problem involved a nonlinear partial differential equation with void ratio as the dependent variable, and the numerical solution was accomplished by a step-by-step procedure combined with a weighted residual technique which leads to a finite element discretization in a spatial variable and a finite difference discretization in the time variable. The model was exercised for four cases (two involving a salt flocculated kaolinite slurry and two involving a dispersed kaolinite slurry) in the stress range within which a “slurry” is transformed to a “soil”, and good agreement was found between theoretical and experimental results. For the particular clay investigated it was found that classical small strain consolidation theory can adequately describe the deformation-time response for all practical purposes after the effective consolidation stress on the slurry exceeded a value of about 8 psi (55 kN/m²).

**Fabric Identification**

The microfabric of each sample was identified by the combined use of scanning electron microscopy, optical microscopy, and X-ray diffractometry. Scanning electron microscopy yielded three-dimensional pictures of particle arrangements at about 2,000 to 10,000 power magnification; these visualizations were extremely helpful in identifying qualitatively the general nature of particle associations and the possible existence of multi-particle units and domains in the clay fabric. Optical microscopy, which involved the transmission of plane polarized light through a thin-section of optically anisotropic clay particles, gave an overall appraisal of particle orientation characteristics at about 100 to 200 power magnification by means of the degree of illumination or extinction observed. X-ray diffractometry with a pole-figure goniometer attachment furnished quantitative measures of particle orientation distributions over areas about 1 or 2 mm in diameter. Accordingly, the combined use of all three methods provided comprehensive assessments of micro-fabric at various levels of organization. A typical example showing the synthesis of these three techniques to characterize the fabric of a sample is given in Figure 11.
Figure 11. Fabric Characterization of a Typical Clay Specimen

CHARACTERISTICS OF CLAY SAMPLE

Mineralogy: Kaolinite (Hydrite 10)
Chemistry of Slurry: NaOH Dispersed
Consolidation Stress Path: Anisotropic
Maximum Effective Consolidation Stress: 2.3 kg/cm² (vertical)
Water Content of Slurry: 249%
Water Content After Consolidation: 53%
Fabric Reference Axis: Direction of Major Principal Consolidation Stress:

Orientation Distribution by X-Ray Diffraction

Plane Normal to Fabric Reference Axis

Plane Parallel to Fabric Reference Axis

Scanning Electron Micrographs

Extinction Illumination Extinction Illumination

Optical Micrographs
Figure 12. Scanning Electron Micrographs of Clay Fabrics

Figure 12 illustrates mutually orthogonal scanning electron micrographs for a highly random sample and a highly oriented sample, the extremes of the fabrics studied.
These micrographs readily convey a qualitative appreciation for the prevalent particle orientation distributions, as evidenced by the preponderance of particle edges or particle faces in the orthogonal micrographs from an anisotropic sample and the relative lack of any noticeable preferred particle orientation in the orthogonal micrographs from an isotropic sample. Optical micrographs were obtained for most of the samples and interpreted in accordance with conventional procedures (Kerr, 1959); the results supported the general trends of particle orientation observed in the scanning micrographs. X-ray diffraction data from three mutually perpendicular planes were used to construct a complete pole figure of particle orientations (reasonable axial symmetry was observed for all samples) and subsequently to quantify the overall particle orientation distribution existing in a given sample (Baker, Wenk, and Christie, 1969). Although each individual method was shown to provide a correct appraisal of the general nature of the fabric of each sample, a synthesis of the information from these three independent procedures has the advantage of being able to characterize the fabric at different levels of particle organization.

Fabric Orientation

Anisotropic consolidation in a slurry consolidometer (no lateral strain) was found to induce a preferred particle orientation that coincides with the major principle consolidation stress (Sheeran and Krizek, 1971). When consolidation under the desired major principal consolidation stress was complete, samples from dispersed slurries were found to exhibit greater orientation than those from flocculated slurries, and increasing the major principal consolidation stress resulted in increased particle orientation. The void ratio at which the structure of a soil equilibrates under a given external stress is smaller for a soil generated from a dispersed slurry than for one generated from a flocculated slurry. However, most of the orientation is introduced at low values of the maximum principal consolidation stress. Although the fabric for isotropically consolidated samples was highly random, a slight orientation of the fabric was observed (Edil and Krizek, 1976); two possible causes for this slight orientation are (a) the conditions that prevailed during the placement of the slurry and (b) the fact that the drainage during consolidation was diametrical.

Coefficient of Earth Pressure At-Rest

The coefficient of lateral earth pressure under conditions of no lateral deformation, $K_0$, is a parameter of great interest for geotechnical engineers. The direct measurement of vertical stress, lateral stress, and pore water pressure during the conduct of these slurry consolidation tests provided a unique opportunity to determine $K_0$ throughout the consolidation history as the clay consolidated from a slurry to a clay block under a maximum vertical stress of 1760 kN/m$^2$ (Abdelhamid and Krizek, 1976). At the outset, it was anticipated that $K_0$ would decrease from unity (hydrostatic pressure in a slurry) at the beginning to about one-half when consolidation was complete, with the final value being somewhat dependent on the fabric of the sample. However, despite the existence of well documented different fabrics, the results of this experimental program manifested no identifiable relationship between $K_0$ during loading and clay fabric. This
experimental evidence suggests that the lateral earth pressure at-rest during loading is not strongly affected by short-term interparticle micromechanisms (as opposed to long-term cementation bonds that prevail in a clay mass). The relationship between the vertical and horizontal effective stresses was essentially constant throughout the entire range of virgin consolidation; accordingly, the value of the coefficient of lateral earth pressure at-rest, \( K_0 \), was constant under these conditions and equal to about \( 0.70 \pm 0.05 \). The relationship between the vertical and horizontal effective stresses during unloading was nonlinear, and the value of \( K_0 \) was therefore not constant. Upon unloading, \( K_0 \) increased slowly from its value during loading until it exceeded unity at an overconsolidation ratio of about 2, after which it increased more rapidly until an overconsolidation ratio of about 8 and then approached the value of the passive coefficient of earth pressure. The use of the effective friction angle to determine \( K_0 \) indirectly appears to be quite practical; \( K_0 \) values computed from Jaky’s (1944, 1948) empirical approach showed good agreement with those measured directly, and the expression proposed by Brooker and Ireland (1965) was found to give relatively smaller, although still acceptable, values of \( K_0 \).

**Strength**

Two series of directional vane shear tests were conducted on these clay blocks; one series was performed while the blocks were still in the slurry consolidometer under the ambient stress conditions and the other was conducted on stress-released or unloaded blocks. In addition, conventional triaxial tests with isotropic and anisotropic reconsolidation were conducted on specimens of these same clays.

**Triaxial Tests**

Horizontally trimmed triaxial test specimens (that is, specimens with their longitudinal axes perpendicular to the direction of the major principal consolidation stress) from samples with an oriented fabric were typically 10% to 25%, and occasionally up to 50%, stronger than those specimens trimmed in the vertical direction, with the difference increasing as the particle orientation increased. For the sample with the maximum particle orientation, the horizontal specimen had a strength almost twice that of the vertical specimen. Particle orientation did not manifest a clear influence on the initial modulus.

**Vane Tests**

For a given value of the consolidation stress, clays with an intrinsically flocculated (random) fabric generally yielded higher vane shear strengths than those initially dispersed (oriented), although at low consolidation stresses (220 kN/m²) the strengths were essentially independent of intrinsic fabric. A vane oriented perpendicular to the direction of the principal consolidation stress yielded higher strengths than one positioned parallel.
Creep

When oriented anisotropically consolidated specimens were tested in consolidated undrained creep, vertically trimmed specimens exhibited higher creep deformations, higher strain rates at a given stress level, and a slower decay of strain rate with time than did the horizontally trimmed specimens, and the difference in the response of the vertical and horizontal specimens increased as the particle orientation of the sample increased. Furthermore, anisotropically consolidated (oriented) samples were more susceptible to higher creep deformations and experienced smaller rates of decay of strain rate than isotropically consolidated (random) samples. Based on the use of rate process theory to interpret creep data from specimens subjected to a rapid temperature change, it was concluded that the free energy of activation for a soil is probably independent of particle orientation and is a property of the clay mineral contacts only.

Electrical Resistivity

Particle arrangement does have an effect on the electrical properties of a laboratory consolidated clay, but factors such as partial saturation and aging may also be very important. The degree of anisotropy in the electrical conductivity of a clay might be used to estimate the degree of particle orientation, but the chemistry of the electrolyte will greatly influence the absolute values of the individual conductivities.

DREDGED MATERIALS

The need to protect the ecology of the Great Lakes and other waterways of the United States led to a variety of problems concerned with the dredging and disposal of increasing volumes of polluted dredge spoil in areas of high population density and industrial development. One commonly used alternative to open water disposal is to place these polluted sediments in diked containment areas to form landfills of marginal value. The dredged materials are typically pumped into the containment area as a slurry with 12% to 15% solids content; the containment area then serves as a settling basin for the solid particles, and the excess carrier fluid is returned to the waterway via a weir – sometimes passing through a filter to remove more of the suspended solids. However, due to the high costs involved, the scarcity of land, and other environmental and economic considerations, it is important to estimate reliably the storage capacity of a given containment area and the resulting landfill should desirably serve some useful purpose. Accordingly, in the 1970s and 1980s the Environmental Protection Agency and the U.S. Army Corps of Engineers Waterways Experiment Station supported considerable research to address these and other issues concerned with the disposal of dredged materials, and the results described here were part of this overall effort.

Extensive Field and Laboratory Study

During the first half of the 1970s a study was undertaken to evaluate quantitatively in an extensive field and laboratory experimental program the engineering characteristics of hydraulically placed fresh water dredged materials. Although
sediments from seven Great Lakes harbors were investigated, most of the research took place primarily at four disposal areas near Toledo, Ohio, as depicted in Figure 13. One of the four field sites was new and provided an opportunity to observe its behavior over the two years during which it was filled with dredged material; instrumentation included settlement plates and piezometers that were installed during the filling process, and time-dependent settlements and pore water pressures were monitored during the entire two-year history of this site. The other three sites had been essentially filled to capacity during the preceding eight years, and field research efforts at these sites were limited to (a) periodic vane shear strength determinations, (b) the measurement of in situ permeability values, and (c) obtaining undisturbed tube samples for subsequent laboratory tests. The laboratory testing program included the evaluation of time-dependent (a) volume changes by means of slurry consolidation, conventional consolidation, and long-term secondary compression tests, (b) strength determinations by use of a miniature vane, fall cone, and unconfined compression tests, and (c) various classification tests. Based on an extensive series of classification tests, it was ascertained that the characteristics of the dredged materials deposited in each of the four sites were essentially the same (as would logically be expected), thereby enabling data from the different sites to be synthesized and interpreted as representative of one large site spanning a time period of about a decade.

**Characterization of Dredged Materials**

Most of the dredged materials studied can be classified as a mixture of organic silts and clays of medium to high plasticity (OH) and inorganic clays of high plasticity (CH), with approximately 60 percent of the materials tested lying in the first category and 40 percent in the second. The organic content was usually between 4 and 8 percent, thereby enabling them to be classified as intermediate organic soils. Particle size analyses indicated that sand, silt, and clay were present in approximate proportions of 1:3:2. The liquid limit and plasticity index were found to exhibit reasonably linear relationships with the clay content, but the plastic limit did not correlate with percent clay. As indicated above, of considerable importance to this study was the fact that the materials deposited in each of the four sites were essentially the same.

**Engineering Properties**

To aid in placing these dredged materials in context with other soils, brief overviews of their engineering properties will be given. It is important to recognize, however, that the thrust of this study focused on fresh water dredged materials from the Toledo area, a narrow subset of the various dredged materials handled annually from our nation’s waterways.

**Primary Consolidation and Compressibility**

The compression index obtained from conventional tests lies between 0.3 and 0.7 and increases linearly with both water content and liquid limit; however, its value obtained from slurry consolidation tests is about 1. For all practical purposes values of
0.0006 cm$^2$/sec and 0.0001 cm$^2$/sec can be assumed to represent the average coefficient of consolidation obtained from conventional consolidation tests and slurry consolidation tests, respectively. The difference in the values of $C_c$ and $c_v$ from the two different tests is attributed primarily to the variation in the manner of deposition of the tested materials.

Figure 13. Aerial Views of Toledo Disposal Areas
Secondary Consolidation

After primary consolidation was complete, the secondary compression of virtually all samples tended to increase in essentially a linear manner with the logarithm of time for a considerable period of time. The coefficients of secondary consolidation from both slurry and conventional consolidation tests were practically the same for corresponding ranges of load, with values ranging between 0.002 and 0.013 for a natural water content range from 45 to 65 percent. The coefficient of secondary consolidation increased exponentially with the consolidation stress for a given natural water content and linearly with the compression index for a given load increment. Despite the significant difference in the proportions of secondary and primary compressions obtained from slurry and conventional consolidation tests, the total settlement per unit height associated with corresponding stress levels was about the same for both tests; however, the times necessary to reach ultimate settlements were shorter for slurry consolidation tests than for conventional tests, suggesting that the times needed to reach ultimate settlements in the field may be much shorter than those predicted from conventional consolidation tests.

Tertiary Consolidation

In many cases the rate of secondary consolidation did not remain constant with time, but rather increased significantly after a week or so, eventually reaching a maximum and then decreasing after several months. Figure 8, which is typical for eight specimens of dredged material loaded for more than 200 days, clearly shows this type of behavior, termed tertiary consolidation (Salem and Krizek, 1975). As seen in Figure 8, a so-called “standard” primary-secondary response curve was measured for about a week, after which the slope of the e-log t curve increased substantially for about two months and then began to decrease. This type of behavior suggests that a structural breakdown of interparticle bonds occurs at some value of strain for each particular load increment. This response pattern was observed for both slurry and conventional consolidation tests on dredged materials.

Permeability

The coefficient of permeability of these dredged materials was determined by a variety of testing methods; as illustrated in Figure 14, values were strongly dependent on the void ratio and decreased from about $10^{-4}$ to $10^{-9}$ cm/sec as the void ratio decreased from approximately 10 to 1. Most permeability values for the firmer materials, which had void ratios on the order of 1 to 2, were in the range of $10^{-7}$ to $10^{-8}$ cm/sec. However, two field infiltration tests yielded permeability coefficients approximately three orders of magnitude higher than those obtained from laboratory tests on undisturbed and remolded samples with comparable void ratios. As expected, vacuum drainage removed water from dredgings much faster than gravity drainage alone, but its effect diminished significantly over longer periods of time. The electro-osmotic coefficient of permeability was found to be about $3 \times 10^{-5}$ cm$^2$/volt-sec, which is approximately one-half the value determined for a large variety of soils.
Strength

The strength characteristics of the dredged materials from the Toledo area are comparable to those associated with fine-grained organic soils with similar water contents and dry densities. Although the shear strength of these materials is generally low shortly after deposition, it increases rather consistently with time. Dredged materials of this nature can be categorized as sensitive, where the sensitivity decreases with increasing dry density. As illustrated in Figure 15, the logarithm of shear strength varied exponentially with the natural water content; similar plots showed that the logarithm of shear strength increased linearly with the dry density and decreased linearly with the liquidity index. In most cases the strength values determined in the field and in the laboratory agreed reasonably well, but values measured by means of the unconfined compression test were considerably lower than those measured by all other tests and values measured by the fall cone test were the highest.

Stabilization

Among the many effective flocculants, calcium oxide and calcium chloride are among the most suitable from the viewpoint of coloration and clearness of the supernatant. Upon studying the test results of this program and comparing the engineering characteristics of the five different dredged materials tested, it was found that the coarser dredged materials were less affected by chemical additives. Based on an overall comparison of all additives tested, lime appears to be the most effective; it has the ability to decrease the long-term rate of volume change, increase the permeability, and retain some of the pollutants. Although the addition of lime caused a reduction in strength and an increase in the calcium ion concentration, it can alter the sediments favorably for further stabilization, such as compaction. The results of repeated-leaching tests indicated that the leachates of chemically stabilized dredged materials do not constitute a very serious potential pollution hazard; in particular, if additives such as lime are used, the pollution potential will likely be reduced. Previous experience suggests that the rate of water loss due to evaporation can be increased substantially by mixing the
sediments and destroying the crust that forms at the surface. If the sediments are not mixed, the exposed surface area per unit volume of dredged material for evaporative purposes will be dictated largely by the formation of shrinkage cracks and the depth of the layer.

Figure 15. Variation of Shear Strength with Natural Water Content
Desiccation/Consolidation Model

A one-dimensional mathematical model was developed by Casteleiro, Krizek, and Edil (1981) to describe the desiccation-consolidation response of a landfill composed of dredged material. This model has the capability of predicting the water content distribution and the settlement response of a landfill at any given time after deposition of the dredged materials, and it can serve to evaluate the various techniques that may be used to accelerate dewatering of the fill. Any type of soil at any degree of saturation can be analyzed, provided the material properties are reasonably well known.

The flow of water through a heterogeneous medium is described by a nonlinear partial differential equation based on the simplifying assumptions that (a) the generalized form of Darcy’s law holds throughout the process and (b) the velocity of the solids can be neglected with respect to the velocity of the fluid. Scale heterogeneity functions are incorporated into the model to account for spatially variable soil characteristics, and the combined effects of evaporation and transpiration at the top boundary, as well as drainage conditions at the bottom boundary, are handled in a general way. The solution to this boundary value problem was obtained by means of a step-by-step finite difference procedure. However, due to the nonlinear character of the governing equation, the time required to solve a practical problem will usually be very long.

As illustrated in Figure 16, settlement predictions calculated by means of this mathematical model were in reasonably good agreement with those actually measured at the Penn 7 disposal site in the Toledo Harbor area. Based on the results of a parameter study with varying boundary conditions, it was found that the effects of evapotranspiration on the dewatering process are quite substantial, and the benefits gained by using vegetation with high transpiration rates may be as good or better than those obtained by improving drainage conditions at the bottom boundary.

![Figure 16. Typical Results from Evaporation-Desiccation-Consolidation Model](image-url)
Storage Capacity

Topographic surveys of Penn 7 before and after the placement of dredgings verified that the ratio between the initial disposal site volume and the bin-measure volume is about 0.62; this ratio is useful when estimating the storage capacity of a given containment area. As seen in Figure 17, nine sets of density determinations from four disposal sites encompassing about an eight-year history indicated that the dry density of deposited dredgings increased at a rate of about 4 percent per year, thereby commensurately increasing the storage capacity of the containment area. Compaction of the dredgings in a disposal area has the ability to significantly increase the dry density of the dredgings and the related storage capacity of the site, but the costs of dewatering and compacting these materials would probably be prohibitive in all except the most unusual cases.

![Figure 17. Increase in Dry Density as a Function of Time](image)

Effect of Aging

The influence of aging on the engineering properties, especially the shear strength, of a “new” (recently deposited) soil has been of interest to geotechnical engineers for a long time, and it is certainly one of the major factors that governs the
long-term usefulness of a dredged material disposal area. As used in the context of this study, a “new” soil is one that has been deposited recently from a disturbed state and aging refers to changes that occur over a time of about 30 years, as contrasted to aging in the geological sense, which may span thousands of years or longer. Changes in shear strength due to aging over this period are those changes which cannot be directly attributed to densification, changes in water content, or similar effects which alter the state of the soil. As such, changes due to aging are those changes which occur at interparticle contacts as a function of time when the state of the soil is essentially unchanged. The results given here were reported in greater detail by Calmeau and Krizek (2001) and are based on experimental data gleaned from three dredged material landfills (Penn 7, Penn 8, and Riverside) in the Toledo area early in their history and 30 years later. The principal dependent variable is shear strength and the primary independent variable is time, with secondary independent variables being the spatial locations of the various data measurements. Averaging procedures were used to reduce the spatially distributed data to a value representative of the landfills at 30 years of age (the dredged materials in the various layers are between 27 and 33 years old – nominally considered to be three decades). Placing this 30-year value of shear strength in perspective with earlier baseline values enabled an estimate to be made of the effects of aging on the time-dependent development of shear strength.

The change in strength due to aging over the 30 years these dredged materials have been in place was deduced by the following rationale. The basic premise is that any measured change in strength may be attributed to some combination of three independent phenomena: (a) partial saturation, (b) densification and/or changes in water content, and (c) development of interparticle bonds (termed aging herein). The first step was to adjust the measured 30-year strength values for the samples above the water table (about 2 meters deep) to account for the effects of partial saturation. Then, the next step was to subtract from the remaining 30-year strength values that portion of strength that existed in the early age sediments at the same water content and/or dry density before any aging effects had been manifested. The resulting strength is attributed to the development of interparticle bonds or aging. Following this procedure, the strength gain due to aging for 30 years was found to lie between 21 kPa and 36 kPa, which values represent between 40% and 70% of the measured 30-year undisturbed strength. Other conclusions emanating from this analysis are (a) the undisturbed shear strength 30 years after placement in a containment area was about 50 kPa – more than an order of magnitude higher than the one-year strength and over 2.5 times the 5-year strength and (b) the 30-year values for the water content and dry density appear to be close to their long-term equilibrium values, whereas the shear strength will likely increase considerably more over the ensuing years. This suggests that consolidation is essentially complete, but aging effects are still exerting their influence.

**Water Quality Study**

A four-month water quality study was conducted at the Penn 7 disposal site in Toledo, Ohio, and an assessment of the fate of pollutants during the dredging disposal cycle was made. In general, it was found that (a) the use of a diked containment area as a
settling basin to retain the solids in dredged materials effectively improved the water quality of the mixtures that passed through it; many of the contaminants apparently associated with the solid particles, thereby settling out of suspension with the solids and reducing significantly the concentrations of polluting materials, (b) the quality of the effluent that was discharged from the disposal area was similar to that of the ambient river water and slightly better than that of the groundwater, and (c) the retained spoil in the diked enclosure was a concentrated source of various pollutants that may leach into the groundwater with the passage of time, thereby reducing to some extent the advantage gained by placing polluted dredged materials in confined disposal areas. Although considerable effort was expended to monitor the various components of the water budget, limited success was achieved because it was impossible to obtain accurate information on the quantity of influent materials and the seepage losses could not be measured.

**Effluent Filtering Systems**

In some cases involving the disposal of dredged material in diked containment areas, waterborne suspended solids and associated contaminants may render the effluents from these disposal areas unacceptable for discharge to the open waters. Accordingly, a study (Krizek, FitzPatrick, and Atmatzidis, 1976) was directed toward evaluating a myriad of filter devices, systems, and concepts and developing a methodology by which appropriate effluent filtering systems for dredged material confinement facilities can be selected and designed. In a broader context this problem consists of identifying, evaluating, selecting, and integrating processes for dewatering dredged material slurries and/or clarifying disposal area supernatants.

The results of an extensive experimental investigation, including both laboratory and field filtration tests on granular media, were used to develop new concepts for the design of nonmechanized filter systems to clarify disposal area supernatants. These systems, which consist of pervious dikes, sandfill weirs with or without backwash, and granular media cartridges, have a relatively wide range of application with respect to the concentration of suspended solids in their influents. Pervious dikes, which may be used for influents with concentrations of suspended solids up to 0.5 g/ℓ, constitute a low maintenance filter that is characterized by very large filter depths and intended for a long effective lifetime. Sandfill weirs without backwash require maintenance to replace clogged filter media at periods significantly shorter than pervious dike lifetimes; although the type of influent to be treated with this system is similar to that for pervious dikes, its mode of operation is much more flexible. For cases where the influents are expected to have suspended solids concentrations up to 1 or 2 g/ℓ, the sandfill weir offers an attractive alternative. Granular media cartridges can be used with waters having loads of suspended solids up to 10 g/ℓ; however, maintenance requirements will likely be excessive at loads higher than a few grams per liter.

Removal efficiency and expected lifetime are two important characteristics of filter media to be used in any design. To assist in the design of granular media filter systems, nomographs were developed to allow (a) the effective grain size or the depth of the filter medium and (b) the time before severe clogging occurs to be estimated from a
knowledge of the required removal efficiency and the concentration of suspended solids in the influent. Gravity sedimentation of dredged material is a natural process that dramatically affects the quality of the effluents from disposal areas. Classical sedimentation basin theories were adapted and nomographs were prepared to estimate the amount and gradation of suspended solids in the effluents of a disposal area (or the influents to a filter system) when the geometry of the area, the flow rate, and the pertinent characteristics of the dredged material slurry are known.

Based on an extensive literature review, it was found technically feasible to use (a) vacuum filtration for dewatering dredged material slurries with 10 g/ℓ or more solids content, (b) a special microscreen device to clarify waters with up to 1 or 2 g/ℓ of suspended solids, (c) special designs of deep bed filters (such as moving bed, upflow, or pressure) to clarify waters with up to 1 g/ℓ suspended solids, and (d) conventional mechanized deep bed filters to treat waters with suspended solids concentrations up to several hundred milligrams per liter. Nonmechanized surface filtration systems using fibrous media were found to be generally unable to provide high removal efficiencies and sustain long runs. Electrofiltration appears uneconomical and it is not yet developed to the stage of field applications. Although the technical feasibility of using the foregoing variety of filter systems has been reasonably well documented, field evaluations are considered necessary and cost-effectiveness studies will further clarify the potentials of each system.

BAUXITE RESIDUE (RED MUD)

Red mud is the residue obtained after aluminum has been extracted from bauxite using the Bayer process. The term “bauxite” was introduced by Berthier (1821) to describe deposits rich in alumina from the vicinity of Les Baux de Provence in the south of France. The aluminum oxide (alumina) in bauxite is combined with various silicates, quartz, and iron oxides, and the impure alumina in the ore is dissolved in caustic soda (NaOH) to form soluble sodium aluminate (Williams, 1975). This process yields a slurry containing NaA102 in aqueous solution and undisolved solids known as red mud with a pH typically on the order of 12.

The aqueous solution is then separated from the red mud and hydrolyzed to precipitate aluminum hydroxide, which is filtered and calcinated to alumina. Throughout the process, flocculents are added to the red mud which goes through a series of washers and thickeners. This increases the solids content of the slurry and allows recovery of as much caustic soda (NaOH) as possible. Depending on the amount of flocculent added to the slurry and the standing time in each thickener, the solids content of the red mud slurry at the end of this process may vary between 20% and 50% by weight. Fluids are then added to the bauxite residue in a mixing tank to obtain the desired discharge solids content, and the slurry is pumped into a disposal area.

Many parameters influence the chemical and physical properties of the bauxite residue. These parameters are usually adjusted to optimize the yield of alumina and the recovery of caustic soda, and little or no effort is expended to control the properties of the
red mud. Hence, the properties of the red mud produced at various plants might vary
tremendously, and the properties of red mud produced at a given plant might vary daily
and even throughout the day. This renders the evaluation of red mud properties and the
subsequent disposal process an especially difficult task.

Disposal Problems

As with many process tailings, the disposal of bauxite residue or red mud in diked
containment areas at a solids content of 10% to 20% by weight presents major technical
and environmental problems, such as low strength, slow dewatering rate, the need for a
large disposal area, groundwater contamination due to leaching of the carrier fluid,
possible failure of the surrounding dike followed by the inundation of neighboring areas
with the wastes, and the delayed and costly reclamation and closure of the disposal
facility. The extent of these problems would be diminished considerably if the mud were
placed in the disposal area in a much drier state, and the applicability of the dry stacking
method was investigated as a means to accomplish this goal.

Dry Stacking

As explained earlier, the dry stacking method was derived from Robinsky’s
thickening discharge work and consists of pumping a thickened slurry (solids content of
30% to 50% by weight) from a given discharge point on the dike perimeter into the
disposal area. After an appropriate layer of material has been deposited, the discharge
point is shifted to another location and this initial layer is left to dry, decrease in volume,
and gain strength. When this scenario has been completed at all discharge points along
the dike perimeter and the disposal area is fully covered by a first lift, the procedure is
repeated for a second lift, and then another, and so forth, until the disposal area is filled.
In such a scheme, little or no fluid covers the disposal area, drying of the slurry is
enhanced, the volume of the slurry is decreased, and the shear strength of the slurry
increases more rapidly. The major challenges with the implementation of this method are
(a) controlling the mechanical properties of the thickened slurry so that a layer of
appropriate thickness and expanse can be deposited and (b) assessing the desiccation rate
to determine the proper time for placement of the next lift. In late 1984 Northwestern
University personnel, in conjunction with the Aluminum Company of America
(ALCOA), investigated the feasibility of using the dry stacking method to dispose of
thickened bauxite tailings.

Overview of Research Effort

Pursuant to a comprehensive literature review and the sharing of ALCOA
background data, a preliminary flow model was prepared and predictions from this model
compared favorably with the results of small-scale field flow tests performed by Stinson
(1983, 1984). However, additional full-scale field and laboratory tests were needed to
further characterize the properties of the bauxite tailings and to study the flow behavior in
a field environment. During the summer of 1985, full-scale flow tests, together with
laboratory viscosity tests, in-situ shear strength tests, and drying tests, were performed at

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ALCOA’s Point Comfort facility in Texas. Further analysis of the flow model indicated that the yield stress is a critical input parameter, and an extensive laboratory testing program was undertaken to define the relationship between the yield stress and the solids content of the bauxite residue.

Visual observations of the full-scale field tests indicated that self-weight consolidation of the slurry occurred during the flow. Therefore, it was decided to investigate and quantify the effect of self-weight consolidation on the shear behavior of the flowing slurry. A series of self-weight consolidation tests were performed in the laboratory, and a computer model was developed to predict the self-weight consolidation of the red mud slurry. The effect of self-weight consolidation during flow on the final yield stress of the slurry was then calculated and incorporated into the flow model.

**Engineering Properties**

Summarized briefly in the following sections are the engineering properties of the red mud at the Point Comfort facility during the period of this study (Krizek et al., 1985).

**Soil Parameters**

The “true” grain size of bauxite residue is approximately 0.1 µm, but the “apparent” grain size is about 1 µm and the “flocculated” grain size may be on the order of 10 µm. When discharged in the field, flocculated and aggregated red mud has the index properties of a clayey silt.

**Yield Stress and Viscosity**

Red mud is highly thixotropic with rheological properties that are strongly dependent on its chemical composition and flocculent dosage. Complete breakdown of the red mud structure requires a few hours of agitation, and structural recovery may never be complete, even after several years of resting time. Furthermore, the time-dependent gain in strength may be partially attributed to a decrease in void ratio, as well as structural recovery. The slurry behaves as a non-Newtonian material with a yield stress typically between 5 Pa and 60 Pa for the muds studied here; below the yield stress, the mud behaves as a solid, but once the yield stress is exceeded, it behaves as a viscous fluid. Following extensive trial-and-error experimentation, it was determined that consistent and repeatable results could be obtained on slurries with a wide range of solids content by conducting viscosity tests using various measuring systems operated at increasing shear rates followed by decreasing shear rates. The yield stress was evaluated by using the flow curve generated during the decreasing shear rate stage of the test to minimize the effects of thixotropy, and a plot of yield stress versus solids content is shown in Figure 18. For similar materials and test conditions, these measured values for the yield stress are similar to those measured by other researchers, but variations in red mud can produce yield stresses which differ by an order of magnitude. Aging of the mud did not influence the yield stress as long as the mud was thoroughly mixed prior to the test.
Sedimentation and Self-Weight Consolidation

The observed dewatering of the mud during flow was attributed to sedimentation and self-weight consolidation. This is a very important factor in the flow problem because dewatering increases the solids content which, in turn, increases the yield stress which governs the final equilibrium or lateral extent of the flow. This phenomenon is especially important at the mud-bed interface because of the effect exerted on the lateral extent of the mud flow. The results of a series of laboratory self-weight consolidation tests on red mud slurries at solids contents ranging from about 20% to 50% by weight indicated that a significant amount of consolidation occurred, and the times to achieve one-half of the final settlement ranged from about an hour to a day (or even longer in some cases), depending on the initial solids content. Regardless of the initial solids content, the final solids content under self-weight consolidation was approximately 50% by weight. However, in such a situation the void ratio and related properties are not homogeneous throughout the slurry. Also, the relationship between the void ratio and the effective stress within the consolidating slurry is not unique, but depends on the initial solids content, slurry thickness, depth within the slurry, and structure of the aggregated slurry. Most computer models can predict reasonably accurate values for self-weight consolidation only if the compressibility and permeability relationships are not strongly dependent on structural changes in the slurry over the range of void ratio considered.

Figure 18. Yield Stress versus Solids Content for ALCOA Point Comfort Red Mud
Flow Behavior

During the early stages of this research program, two series of field flow tests were conducted at the ALCOA plant in Point Comfort, Texas. The first series consisted of nine small-scale flow tests (Stinson, 1983, 1984) wherein flows were confined in a narrow channel and essentially one-dimensional, and a second series of five full-scaled multi-dimensional tests was conducted by Northwestern University personnel (Krizek and Palmer, 1986; Palmer and Krizek, 1987).

The general shape of the disposal area was square with one side somewhat skewed; dimensions of the sides were on the order of 1000 meters. This disposal area was surrounded by a dike on which was placed a 12-inch diameter feed pipe. Approximately every 100 m, there was a 10-inch diameter discharge pipe fitted with a gate valve which, when opened, allowed the mud to be discharged into the disposal area. At the time the flow tests were conducted, the center area of the mud lake was partially submerged under red mud liquor and the gently sloping beaches were fairly dry, but very soft, especially at distances greater than about 130 m from the dike. Four of the tests were conducted adjacent to each other on one side of the disposal area, and the fifth test was performed on the opposite side. Shown in Figure 19 is a schematic diagram of the piping at a typical discharge point.

![Figure 19. Schematic Diagram of Discharge Point](image)

As the mud was discharged during a given test (presumably at a constant flow rate and a constant solids content), it flowed slowly in the direction of the largest gradient to form a tongue with a shape that depended mainly on the magnitude of the field slope, the surface characteristics of the beach, and the properties of the mud. During this process the velocity, depth, and approximate lateral extent of the “tongue” were measured at selected time internals, and a typical topographic survey of one of the test areas before
flow began, during flow, and after flow ceased is given in Figure 20. Once pumping stopped, the mud continued to flow until it reached a state of equilibrium, after which no further significant movement of the mud was observed. Based on the qualitative and quantitative observations made during the performance of full-scale field flow tests, it was concluded that a limit equilibrium model would predict the final configuration of the red mud to typically within 25% of the observed data. One example of measured and predicted flow patterns is given in Figure 21.

Figure 20. Topographic Map of Typical Field Flow Test
During the flow tests several observations deemed to be important were made. For example, once the thickness of the mud near the discharge point reached about 20 cm (typically after 1 or 2 hours of pumping), fluid appeared to run off the surface of the flowing mud. This fluid flowed toward the tip of the mud flow and prewetted the existing bed surface in front of the flowing mud; however, this fluid was not rapidly absorbed by the dry mud bed, thereby supporting the assumption that a negligible amount of fluid contained in the flowing mud was absorbed by the dry mud bed. Another observation indicated that fluid appeared at the mud surface only 1 or 2 hours after the beginning of the flow tests, thus suggesting that sedimentation or self-weight consolidation of the flowing mud needed to be investigated. It was also observed that vertical cracks formed within the flowing mud; these cracks appeared to be created by an upward fluid flow within the mud and resulted in preferred drainage paths. The observation that the flowing mud surface never appeared to be dry indicated that evaporation of fluid from the mud surface did not seem to affect the flowing mud significantly.
FLUE GAS DESULFURIZATION (FGD) SLUDGES

Each year tens of million tons of sulfur oxide pollutants are discharged into the atmosphere, and most of these pollutants come from the combustion of coal that contains sulfur as a natural contaminant. Sulfur dioxide (SO₂), which is the main pollutant, has been found to irritate the respiratory tract and can cause permanent injury to the lungs. When exposed to moisture in the atmosphere, much of the sulfur dioxide oxidizes into acidic sulfates (H₂SO₄), which then fall in the form of acid rain. The depletion of nutrients from soils and streams is a problem often caused by acid rain. About two-thirds of industrial sulfur pollution comes from coal-burning, electric-power generating plants, with an additional one-third from other industrial sources and residential/commercial heating systems.

FGD Processes

The most common technology to control SO₂ emissions today are a variety of flue gas desulfurization (FGD) processes, wherein the exhaust gases combine with a fine slurry mist of lime or sodium carbonate. More than fifty different FGD processes have been developed, but only a few have received widespread use. FGD systems can be classified as (a) non-regenerable or throwaway systems and (b) regenerable or recovery systems, but the large majority of FGD systems currently operational are lime or limestone scrubbers of the throwaway type. The most common throwaway FGD systems for near-term SO₂ control are (a) direct lime scrubbing, in which the sulfur dioxide reacts with lime to form calcium sulfite and water, (b) direct limestone scrubbing, which is similar to lime scrubbing, but also yields carbon dioxide, (c) double alkali scrubbing, which is based on second generation FGD technology and uses scrubbing soluble alkali salts (e.g., sodium) for SO₂ removal, (d) dry scrubbing, which is a modification of wet scrubbing FGD technology that takes advantage of the heat of the exhaust gases to dry the reacted slurry into particles of calcium sulfite and sodium sulfite, and (e) various other systems such as magnesium oxide scrubbing, the Wellman-Lord process, and citrate, carbon adsorption, and copper oxide adsorption systems. One problem is that the sludge produced at a given utility plant may differ considerably from day to day or even within the span of a day, and this imposes difficulties on generalizing any approach to a solution.

Sludge Disposal

There are three major alternatives for the disposal of FGD sludge; these are ponding, landfilling, and mine disposal. Ponding (wet disposal) involves pumping slurried solids directly to a disposal pond; it generally minimizes operational processing and may reduce transportation costs, but it has the potential for groundwater and surface water contamination and the degradation of a large area. Landfilling (dry disposal) involves hauling dewatered or chemically treated sludge to a disposal site or placing it in an excavated area or an above ground landfill. Although landfilling reduces the land deterioration and water pollution associated with ponding, the potential for leaching contaminants into the ground and surface water and the low strength of the sludge are
two major disadvantages, unless some stabilization or fixation process is used. Mine disposal, which is the least used of the three alternatives and will not be discussed here, simply involves disposing of the sludge in an abandoned mine; although typically below the surface and “out of sight”, the potential for polluting the groundwater clearly poses a problem.

**Ponding**

The disposal of an FGD slurry in a pond involves the rheological properties (flow characteristics, sedimentation rate, and sedimented dry density) of the slurry, as well as a number of engineering considerations (site selection, dike design and construction, sand blanket and/or liner, etc.) not addressed here. The design of a pipeline system to transport the slurry to the disposal area requires a knowledge of the quantity of slurry to be pumped, the percent solids in the slurry and their sedimentation rate, and the yield stress and apparent viscosity of the slurry. FGD slurries are clearly non-Newtonian and have often been modeled for simplicity as a Bingham material, although a few highly theoretical studies have employed more complex constitutive models. The engineering solution to the transport problem is further complicated by the fact that the slurries manifest a degree of thixotropy and under some conditions they tend to settle during the pumping process. Once placed in the disposal area, sedimentation and self-weight consolidation proceed simultaneously in accordance with the general process described by Imai (1981).

**Landfilling**

Landfilling typically refers to the disposal of “dry sludges, that is, sludges with a sufficiently high solids content (perhaps on the order of 40% or 50% by weight) that they can be handled with “standard” earthmoving equipment (trucks and low pressure dozers) and stacked at some stable slope without flowing. In addition to dewatering and perhaps mixing these sludges with additives, such landfills involve all of the normal considerations regarding compaction, settlement, permeability, liners, slope analysis, and erosion. Wetter sludges not amenable to stacking must be placed in trenches or existing pits to confine them; sometimes blending them with native soils or drying them by evaporation (provided favorable climatic conditions exist) will lower the water content sufficiently to allow stacking. Notwithstanding the idealized scenario depicted above, trucks hauling sludge to a disposal site often have difficulty in dumping the sludge because it tends to adhere to the bed and sides of the trucks and low-ground-pressure dozers must usually be used. These problems are aggravated for sulfite-rich sludges, because the microstructure of calcium sulfite consists of a rosette-type particle configuration which is conducive to holding large amounts of water around the small platelets; although this water flows very slowly under gravity forces, it is released much more readily when the sludge is vibrated or agitated.
Double Alkali System

Although data from several FGD systems were analyzed, most of the experimental work undertaken at Northwestern University was conducted on sludges produced by the double alkali system. In this system the spent scrubbing liquor is reacted with lime outside the scrubber system, thereby forming a slurry which, after drying, produces a caked waste product; at the same time, the alkali salts can be recovered, thus minimizing the reagent costs. In general, the waste product of most FGD systems usually includes calcium sulfite, which settles and filters poorly because of its morphology. To overcome this problem a process called “forced oxidation”, in which air is blown into the tank that holds the used scrubber slurry (composed primarily of calcium sulfite and water) was developed. The air oxidizes the calcium sulfite to calcium sulfate, and the calcium sulfate formed by this reaction grows to a larger crystal size than does calcium sulfite. As a result, the calcium sulfate can be easily filtered to a much drier and more stable material.

Sludge Characteristics

The pH values of all FGD sludges are high, but they are particularly high (> 12) for double alkali sludges, and these values are likely to remain unchanged for years if no other neutralizing chemicals are used. Most FGD sludges, including double alkali sludges, have a specific gravity of about 2.5 to 2.7, but gypsum sludge has a specific gravity on the order of 2.35. Morphology is influenced by the chemistry of the system and is important to the water-holding capacity of FGD sludges; the microstructure of calcium sulfite produced by a double alkali system has a predominant rosette configuration; spherical aggregates and flat platelets are common in lime and limestone systems, respectively, and gypsum (calcium sulfate) has prism-shaped crystals. Conventional hydrometer tests showed that the particle sizes (equivalent diameter) of sludge particles are primarily in the coarse silt range and about the same for both calcium sulfite and calcium sulfate sludges. Most FGD sludges are non-plastic in nature and not amenable to Atterberg limit tests (Chu, 1983).

Pozzolanic Reactivity

In most operations involving FGD sludges, fly ash is a by-product of the combustion process. Although several benefits are often gained by combining the sludge and fly ash, many utility plants separate them and dispose of them in separate disposal areas due to prevailing regulations. Fly ash is composed primarily of silicate, aluminate, and calcium particles, and it is non-plastic; its particle size distribution lies in the range of silt and fine sand. The specific gravity of fly ashes varies over a wide range (1.9 to 2.8, with the majority lying between 2.1 and 2.6) and decreases with an increase in the amount of SiO₂ and Al₂O₃. Lime is a general term used to describe several varieties of highly alkaline chemical compounds.

A pozzolan is a material which is capable of reacting with time in the presence of water at ordinary temperatures to produce cementitious compounds, and fly ash is a well
known pozzolan. The short-term strength can be loosely related to the carbon content, silica and alumina content, glass content, density, and specific surface of fly ashes, whereas the long-term strength (1 to 2 years) can be correlated most closely with their SiO$_2$ or (SiO$_2$ + Al$_2$O$_3$) content (Thorne and Watt, 1965). The time rate of development of strength in a pozzolanic reaction is increased strongly by a rise in temperature.

**Engineering Properties**

As with soils in general, the engineering properties of FGD sludges are highly variable and depend strongly on the specific sludge and the conditions to which it is subjected. Given here are some overall observations and measurements that were made over more than a decade on a number of different sludges, with an emphasis on double alkali sludges. Nevertheless, any quantitative values given must be accepted with an awareness that they represent measurements and/or trends for a limited number of specific sludges (Chu, 1983).

**Sedimentation**

Sludge slurries with a low solids content (about 5% by weight) have a high sedimentation rate (about 50 cm/hr or more); the slurries generally settle in accordance with Stoke’s law and have indistinctive solid-liquid interfaces. On the other hand, sludge slurries with a higher solids concentration (15% or more) have relatively low sedimentation rates (on the order of 5 cm/hr). Gypsum slurries tend to settle at a uniform rate (that is, the solid-liquid interface is quite distinct) by displacing water through small channels. For sludges with particles of similar morphology, those with larger crystals settle faster. Sedimented double alkali sludges with solids contents between 10% and 25%, when in slurry form, have a bulk density of about 1.3 g/cm$^3$, whereas most other FGD sludges exhibit a more-or-less exponential relationship between the bulk density and solids content of the slurry.

**Viscosity**

FGD slurries are usually non-Newtonian, shear-thinning, and thixotropic. In addition to the many material factors (concentration of suspended solids, particle size and gradation, particle shape, degree of dispersion, type and concentration of electrolyte, deformability of colloidal particles, temperature of the suspension, etc.) that could influence the viscosity, the viscometer itself and the manner in which the test is conducted can also have a substantial effect on the measured viscosity. It is especially difficult to investigate the effect on any one of these variables because of the need ideally to maintain all of the others constant during a test series. Notwithstanding this formidable challenge, the viscosity of FGD slurries increases more-or-less exponentially with an increase in the solids content. Fly ash, on the other hand, has a much lower viscosity (typically one order of magnitude or more) relative to sludges at the same solids content tested under the same conditions. Consequently, adding fly ash to a sludge slurry tends to reduce the viscosity of plain sludge for specified test conditions. This is because fly ash particles are spherical in shape, whereas double alkali sludge particles have a
“rosette” configuration; consequently, the surface area of fly ash is much smaller than that of sludge for a given concentration in a slurry.

Compaction

In general, gypsum sludges have much higher maximum dry densities (80 to 100\(^+\) pcf) and lower optimum water contents (15% to 30%) than sulfite-rich or double alkali sludges (~70 pcf and 30% to 45%, respectively) for the same Standard Proctor compactive energy. However, the compaction curves were very sensitive (peaked) for gypsum sludges and rather insensitive (flat) for double alkali sludges; recompacted sludges sometimes manifested an increase of up to 10 pcf in the maximum dry density. Since the as-produced water contents of FGD sludges are usually much higher than their optimum water contents, the maximum dry densities obtained in the laboratory cannot be duplicated in the field for the same compactive energy unless the sludge is dewatered to a water content close to its optimum water content. The easiest way to lower the water content of a sludge is to add a dry material (such as lime and/or fly ash or perhaps a local soil) to the sludge. The addition of fly ash to sulfite-rich sludges increased their maximum dry density and decreased their optimum water content, but the opposite result was found for gypsum sludges; for double alkali sludges the data were erratic, and no conclusion can be deduced. The addition of lime alone decreased the maximum dry density and increased the optimum water content of one double alkali sludge and one sulfite-rich sludge, but the addition of both lime and fly ash produced the opposite effect for both sludges.

Permeability

The permeability of FGD sludges spans more than four orders of magnitude, but the majority of the data lies between 10\(^{-6}\) and 10\(^{-4}\) cm/sec and is essentially independent of curing time. The addition of fly ash to the sludge tended to increase its permeability several-fold, but usually within the same order of magnitude, regardless of the type and amount of fly ash; some individual specimens showed increasing permeabilities with time, but this trend was not conclusive. The addition of lime to the sludge-ash mixture generally decreased the permeability, but this decrease was a rather erratic function of the lime content; since it was frequently difficult to obtain duplicate results (even within the same order of magnitude) from duplicate specimens, any apparent trend must be viewed with caution. Although permeability values on the order of 10\(^{8}\) and 10\(^{-7}\) cm/sec have been measured, such low values were not obtained consistently and were probably due to some test anomaly. Whenever the permeability could be measured with confidence, it was usually on the order of 10\(^{6}\) to 10\(^{-5}\) cm/sec, regardless of the amount of lime in the mixture. Due to variations in field placement procedures, field permeability may be different from laboratory permeability. Variables such as dry density and homogeneity (stratification) must be taken into account when extrapolating any laboratory permeability values to field problems.
Consolidation

The time for completion of primary consolidation of FGD sludges was very short compared to that for most natural clays, and this time was even shorter if the sludge was not fully saturated, as would normally be the case in the field. If the sludge is placed in lifts (layers) at a disposal site, ordinary compaction by conventional earth moving equipment may densify the sludge and virtually complete its consolidation process during the placement phase. The addition of fly ash to sludge reduced its initial void ratio and resulting settlement in the field. The addition of lime to a fresh sludge-fly ash mixture had little influence on the consolidation behavior of the mixture.

Strength

FGD sludges are primarily frictional in nature; they have small values of cohesion (usually less than 5 psi) and relatively large angles of internal friction (usually larger than 30 degrees) in a drained condition. In an undrained condition the friction angles were slightly larger than one-half the friction angles in a drained condition, but the cohesion values were of the same order of magnitude in both drained or undrained conditions. This suggests that FGD sludges behave similar to silts and, when subjected to an external load in a drained condition, much of the load will be carried by the internal particle-to-particle frictional forces instead of cohesion forces. The addition of fly ash to sludge had less effect on the friction angles when tested in a drained condition. The cohesion values of FGD sludges were erratic when an additive was mixed with the sludge. Aged sludge-fly ash-lime mixtures usually had a larger angle of internal friction (up to 8 degrees) than fresh mixtures in a drained condition.

The unconfined compressive strength of plain double alkali sludge was on the order of 5 psi and 8 psi for fresh and 28-day specimens, respectively. Dry additives, such as fly ash and lime, are often blended with the wet sludge to reduce the water content and improve the handleability of the mix, and the unconfined compressive strength is then influenced by the additive. In particular, this strength is strongly influenced by the lime content in any additive. For example, the strengths of sludge-fly ash mixtures were much higher when a highly reactive (high lime content) fly ash was added. The strength gain of a sludge-ash-lime mixture is related to the number of contacts between fly ash particles and lime particles within the mixture. Uniformly graded lime particle sizes around 0.2mm seem to have a better effect on the strength gain than more well graded distributions.

Freshly sedimented sludge with or without an additive usually had a very small (almost negligible) vane shear strength (on the order of 0.01 kg/ cm²), regardless of the solids content. The strength of sedimented sludge with fly ash and lime added increased with time due to the formation of chemical bonds; for example, 21-day strengths were sometimes found to be as large as 100 times the 7-day strengths.
MICROFINE CEMENT GROUTS

At the time this work was undertaken (circa 1980), microfine (sometimes termed ultrafine or superfine) cement grouts were relatively new in the United States. In the past two decades the availability of these cements and our experience working with them has increased significantly. Notwithstanding their increased availability, microfine cements are still considerably more expensive than ordinary portland cements due to their high manufacturing cost. Microfine cements are made by further refining conventional cement particles; this is usually accomplished by dry milling or using ventilation to separate only the target fineness fraction. Although there is no established reference standard in terms of grain-size distribution, microfine cements have particle sizes that are typically about an order of magnitude smaller than those of ordinary portland cement. This physical characteristic is beneficial for permeation grouting because the suspension is capable of penetrating into fine sands and rock fissures that are impenetrable by coarser portland cement suspensions. Therefore, microfine cement grouts can be very effective in specialized applications and offer a viable replacement for chemical grouts and their associated problems of toxicity and permanence.

Definitions of Terms

To assure effective communication, brief definitions of the terms used will be presented. The concentration of cement particles in a particular grout mix is designated by the water:cement ratio of the mix, where the components are proportional on a weight basis. The relatively small amounts of other additives (bentonite, dispersing agents, etc.) are expressed as percentages of the dry weight of cement. The setting process may be considered as having two stages – an initial stage in which the fluidity of the grout decreases to a level at which it is no longer pumpable, and a second stage, termed final set, in which the grout hardens and attains “significant” strength. The bleed manifested by a grout is a measure of its stability; bleed, which is measured in a sedimentation test, is defined as the volume of bleed water above the grout suspension, expressed as a percentage of the total initial volume of the grout slurry. Grout suspensions are often modeled as Newtonian or Bingham fluids (in reality, they are neither), the mechanical behaviors of which have been discussed previously.

Properties of Interest

Shown in Figure 22 is the probable permeation characteristics of several suspension and solution grouts into different soils. However, microfine cements must be used with great care, giving due respect to the effects resulting from the extreme fineness of their particles. In dry form, they tend to compact due to their sensitivity to humidity and electrostatic phenomena. When mixed with water, they tend to flocculate, thereby losing the fineness that is their fundamental characteristic. When preparing grout mixes, this latter problem is partially solved by the use of dispersing agents to prevent aggregation by making the particle potential repulsive. Accordingly, despite their high cost, microfine cement grouts can prove very useful and economical in special situations,
but they must be used with a full appreciation for their unique properties, some of which may prove problematic if ignored.

Figure 22. Comparative Permeation of Various Grouts

When designing a field grouting operation, engineers are concerned with several different properties of candidate grouts, not all of which are independent. Suspension grouts, such as microfine cement grouts, are usually characterized by the concentration of solid material (cement particles) in the grout slurry; this is generally expressed as the water:cement ratio. The set time or gel time is always of interest; if it is too short, it will be impossible to inject the grout very far into the formation, whereas, if it is too long, the grout may be diluted and/or elutriated from its intended location. In suspension grouts, the bleed or rate of sedimentation is important because the bleed must be held to a minimum prior to set. Another property of concern is the compatibility between the grout and the geologic formation, lest some undesired chemical reaction inhibit the intended placement of the grout. Two rheologic parameters of optimal importance are the viscosity and yield stress of the grout; the viscosity will control the rate at which the grout can be injected, and the yield stress will govern the distance that the grout can be injected into the formation. For suspension grouts the particle size distribution of the grout relative to the void sizes of the geologic formation is very important to prevent mechanical filtration, and the size and chemistry of the grout particles will dictate the extent to which they will be chemically filtered from the grout as it is injected. Some complex and interactive combination of all or most of the foregoing properties give a measure of the groutability of a particular geologic formation with a given grout (the groutability index is usually simplified – often involving only various particle sizes – but its true nature remains one of the many mysteries of the grouting profession). Assuming that a formation is successfully grouted in accordance with the plans and specifications, the permanence of the in situ grout frequently is an important concern. As a final requirement, the entire grouting operation must not introduce any toxic elements into the subsurface environment.
Effects of Mixing

The positive role of high energy shear mixing in cement grout technology has been well stated by Houlsby (1982), and it is widely accepted as an important factor in microfine cement grouting. To compare quantitatively the effects of shear mixing on the viscosity, sedimentation characteristics, set time, and compressive strength of microfine grouts, Schwarz and Krizek (1992) conducted an extensive experimental program involving Type I portland cement and three microfine cement grouts at different water:cement ratios (primarily in the range from 1:1 to 3:1) mixed in five different mixers (a Colcrete mixer, a custom-designed proprietary mixer, a standard Oster household blender, a laboratory-size Morehouse/Cowles Dissolver, and a laboratory-size Lightnin paddle mixer) for various periods of time (generally one or ten minutes). This issue is of particular concern when attempts are made to predict the performance of field-mixed grouts prepared in one type of mixer with data obtained from laboratory-mixed grouts prepared in a totally different type of table-top mixer. Grouts mixed for 10 minutes settled on the order of 100 minutes faster than those mixed for 1 minute, but the final sedimented volumes for grouts with corresponding water:cement ratios were nearly the same. However, grouts prepared in field-size high energy mixers manifested bleed volumes that were 40% to 60% lower than those of corresponding mixes prepared in table-top high energy mixers. As the mixing time increased from 1 minute to 10 minutes, the viscosity of the grouts substantially increased, with this effect being more pronounced for the low water:cement ratio grouts (double to six-fold for 1:1 mixes; approximately an order of magnitude for 2:1 mixes; and double to triple for 3:1 mixes); the type of mixing, however, exerted only a small influence. The set time was relatively insensitive to the type of mixer or the duration of mixing, except for a modest increase for the grouts mixed in one of the field mixers. Data from unconfined compression tests indicated lower strengths as the water:cement ratio of the grout increased; blender-mixed grouts were stronger than field-mixed grouts, but weaker than paddle-mixed grouts.

Rheological Parameters

Tests on microfine cement grouts at water:cement ratios of 1:1, 2:1, and 3:1 showed that (a) the yield stress decreased (sometimes substantially, with factors as high as 20) as the water:cement ratio increased, with most of the change occurring between the 1:1 and 2:1 mixes, (b) there was relatively little change in the plastic viscosity for a given grout at different water:cement ratios, and (c) the apparent viscosity decreased with increasing water:cement ratio in a proportion roughly equivalent to that for the yield stress. The effect of adding 1% bentonite to the grout mixes typically resulted in increasing the yield stress and apparent viscosity about two- to three-fold, with little to no change in a few cases. When the amount of bentonite added to the 2:1 mix of one of the grouts was increased incrementally to 8%, both the yield stress and the apparent viscosity increased dramatically, but the plastic viscosity exhibited little change. When the cement comes into contact with the prehydrated bentonite in the mixer, the slurry undergoes a rapid stiffening caused by mutual flocculation of the negatively charged bentonite particles and the positively charged cement particles; in addition, the cement releases calcium and other ions into solution, and this will also promote flocculation of the
bentonite. A combination of these mechanisms, together with particle size and concentration, determines the time-dependent fluidity of the grout mix (Schwarz, 1997).

**Injectability**

In addition to the important roles played by viscosity and yield stress, at least four other factors influence strongly the injectability of a microfine cement grout. These are mechanical filtration, chemical filtration, product variability, and reaction with the host medium. As usual with any complex phenomenon, the cause and effect associated with each of these factors are sometimes interrelated, but they will be discussed individually for simplicity.

**Mechanical Filtration**

Various empirical relationships can be found in the literature expressing the groutability of a given sand in terms of specific particle sizes of the medium and the suspension grout. Based on particle size relationships alone, microfine cement grouts should be injectable into sands with a $D_{10}$ on the order of 100 to 200 $\mu m$, depending on the water:cement ratio of the grout, additives, injection pressure, and grout preparation. As emphasized earlier, shear mixing is very important in the preparation of a grout, and the elapsed time prior to injection should not be too long to preclude excessive hydration of the cement particles. Also, thinner grouts with a dispersing agent, but without bentonite, are more injectable than the converse. A grout chemistry conducive to flocculation would effectively produce “larger” particles, despite the fine particle size distribution of “individual” particles.

**Chemical Filtration**

Research and field experience have shown that microfine cement grouts are sometimes unable to permeate a given soil, although their particle size distribution and rheologic properties satisfy reasonably well accepted groutability criteria. This is often because the colloidal nature of the cement particles is conducive to chemical filtration, which exerts an important influence on the injection of microfine cement grouts and the migration of particles within the porous medium. The filtration processes of suspended particles involve consideration of such factors as (a) particles coming in contact with retention sites, (b) fixating or capturing of particles on the sites, (c) breaking away of previously retained particles, and (d) flocculation and aggregation of suspended particles during flow. The degree to which each of these phenomena occur is influenced by the chemical attributes, such as ionic strength and zeta potential, of the cement-water system. Experiments involving the injection of various grouts having dramatically different rheological properties and electro-chemical characteristics into sands have shown that (a) selective filtration of specific colloid-size particles occurs during the permeation process and (b) the mass and size of particles deposited by chemical mechanisms varies with distance from the injection point. Indeed, analyses of the effluents from grouted columns of sand in the laboratory have confirmed that, for the “right” chemistry of the grout
suspension, the coarser cement particles actually pass through the sand column while the finer particles are captured (Schwarz, 1997).

**Product Variability**

Small variations in the microfine cement product, as has been experienced from time to time over the years of research, can significantly affect the injectability of the grout. For example, variations in viscosity and bleed for “identical” microfine cement grouts were found to be as large as 70% and 25%, respectively, for 2:1 water:cement ratio mixes and about 50% and 10% for 3:1 mixes. In some cases a microfine cement was found to be “contaminated” with less than 0.5% of coarser silt-sized particles (four to five times larger than the mean size), and these large particles, together with some smaller “filtered” particles, effectively blinded the pores near the injection point and prevented the injection of the grout (despite tripling the injection pressure) in a situation where all other conditions were consistent with successful injectability on numerous previous occasions. Hence, the injectability of a grout is controlled strongly by the coarsest 1% or 2% of the grout particles.

**Reaction of Host Medium**

Undesirable reactions between a microfine cement grout and its host medium can cause problems with bleed, set time, long-term stability, and overall effectiveness of the grouting operation. This is especially important when considering the use of these grouts to encapsulate contaminated sands, to form a curtain to intercept a plume of contaminated groundwater, or to grout a soil medium with an incompatible chemistry. When such a condition is encountered, a site-specific evaluation of the grout mix must be conducted to ascertain a satisfactory completion of the project.

**Anisotropy**

Grouting sand with a microfine cement grout reduces its pore sizes and alters its pore structure. After injecting the grout into the sand, the cement particles which are not trapped at the intergranular contact points or do not adhere to the particle surfaces begin to settle in the pore spaces. As a consequence of the sedimentation process, most of the cement particles are eventually deposited in the void spaces bounded by the contact points and upper surfaces of the sand grains, as shown in Figure 23, and the bleed water accumulates at the upper portions of the pores.

![Figure 23. Schematic Representation of Sand Injected with Microfine Cement Grout](image)

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The amount of this bleed water depends on the water:cement ratio of the grout, additives (silica fume, slag, fly ash, bentonite, etc.), dispersing agent, mixing time, and set time. The portion of the soil voids filled with hydration products was determined to be approximately 98%, 85%, and 65%, respectively, for 1:1, 2:1, and 3:1 water:cement ratio grouts. This introduces a preferred orientation and distribution in the resulting voids and an associated anisotropic behavior of the grouted soil. This effect was quantified by grouting Ottawa 20-30 sand with a microfine cement grout at different water:cement ratios and then subjecting the grouted sand to permeability, unconfined compression, and tensile tests, as illustrated in Figure 24 (Helal and Krizek, 1992; Krizek and Helal, 1992). In the description of these tests, “horizontally cured” and “vertically cured” refer to the position of the long axis of the test specimen during the curing process; since all specimens were tested in the direction of the long axis, this means that the test direction was either “coincident with” or “orthogonal to” the position of the specimen during the curing process or the direction of anisotropy.

![Figure 24. Schematic Representation of Test Conditions](image)
**Permeability**

The results of the permeability tests are given in Table 1. For all specimens injected with a 1:1 grout and for vertically cured specimens injected with a 1.5:1 grout, the coefficient of permeability was less than $10^{-7}$ cm/sec and could not be measured with the test set-up employed under a gradient of 50. For the other cases the coefficient of permeability of the vertically cured specimens grouted with a 2:1 or a 2.5:1 grout was about two orders of magnitude lower than that of the horizontally cured specimens, but little difference (and actually the opposite trend) was observed for specimens grouted with a 3:1 grout. In all cases the permeability decreased with curing time, approximately halving in value as the curing time increased from 7 to 28 days; for the horizontally cured specimens grouted with a 1.5:1 grout, the permeability decreased about two orders of magnitude between 7 days and 28 days. The test gradient in most cases was between 10 and 15 with no back-pressure, and tests were conducted for periods ranging from 15 minutes to 24 hours, depending on the permeability of the specimen.

Table 1. Data from Permeability Tests

<table>
<thead>
<tr>
<th>Water-to-Cement Ratio</th>
<th>V or H</th>
<th>Coefficient of Permeability $k$(cm/sec), after</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>7 days</td>
</tr>
<tr>
<td>1:1</td>
<td>V</td>
<td>Less than $10^{-7}$</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>Less than $10^{-7}$</td>
</tr>
<tr>
<td>1.5:1</td>
<td>V</td>
<td>Less than $10^{-7}$</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$8 \times 10^5$</td>
</tr>
<tr>
<td>2:1</td>
<td>V</td>
<td>$4 \times 10^5$</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$3 \times 10^3$</td>
</tr>
<tr>
<td>2.5:1</td>
<td>V</td>
<td>$8 \times 10^4$</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$7 \times 10^2$</td>
</tr>
<tr>
<td>3:1</td>
<td>V</td>
<td>$9 \times 10^2$</td>
</tr>
<tr>
<td></td>
<td>H</td>
<td>$6 \times 10^2$</td>
</tr>
</tbody>
</table>

V = vertically cured specimen  
H = horizontally cured specimen

This difference in the coefficient of permeability between horizontally cured and vertically cured specimens can be attributed to the development of interconnected pore space due to the sedimentation of the cement particles in the soil voids. Although the total void space occupied by bleed water may be considered essentially the same for both
horizontally cured and vertically cured specimens injected with grout at a given water:cement ratio, the interconnectivity of the void space for flow in a particular direction is the issue. In the horizontally cured specimens the “layers” that separate the bleed water from the hydration products are more-or-less parallel to the flow path, while they are generally perpendicular to the flow path in the vertically cured specimens, as illustrated in Figure 24(a). This situation prevails when there are sufficient cement particles in the grout suspension to form a “closed” aggregation of deposited particles in each pore space, but not so many as to fill almost the entire pore space.

When the grout is dilute (water:cement ratio of 3:1 or greater), a complete blockage of the voids by “layers” of hydration products does not occur, because most of the hydration products are concentrated at the contact points and on the surfaces of the sand grains. In these cases the coefficient of permeability manifests little directional dependence. When the grout is thicker (water:cement ratio lower than about 1.5:1), the concentration of cement particles, and hence the hydration products, is sufficiently high that sedimentation and the resulting bleed water exert a negligible influence within the range of permeability investigated. However, it is entirely possible that, if lower ranges of permeability values were studied, a similar anisotropy may be observed.

**Compressive Strength**

The results of unconfined compression strength tests illustrated in Figure 24(b), on specimens of grouted sand are given in Figure 25(a). Horizontally cured specimens with a water:cement ratio greater than 1.5:1 are clearly stronger than vertically cured specimens, with the strength difference increasing with increasing water:cement ratio and reaching about 35% for a 3:1 grout. The suggested reason for this observed anisotropy is that the hydration products in the horizontally cured specimens form “bridges” to transmit the stress from one sand grain to another in the direction of loading, whereas in the vertically cured specimens, the accumulated bleed water in the upper portions of the voids serves as a weak link for transmitting stress from one array of sand grains to a lower array.

![Figure 25. Compression and Tensile Strengths versus Water:Cement Ratio](image-url)
In a recent study of lightly cemented soils subjected to uniaxial loading, Gill, Miglionico, and Andrews (1990) found that the inter-particle bonds break under relatively small strains and, with increasing strain, the particles rearrange into a denser configuration before the specimen actually fails under a higher stress. In the vertically cured specimens the bleed water accumulates under the sand grains, and there is little cementitious material to inhibit the rearrangement of sand grains under low stresses. Alternatively, the horizontally cured specimens have “columns” of cementitious materials and sand grains to transmit the load, thereby enhancing their ability to carry higher loads without rearrangement of sand grains. This difference in the stresses required to initiate rearrangement of the sand grains decreases as the water: cement ratio of the grout decreases, until it reaches 1:1, at which point the effect of the bleed water is essentially negligible.

**Tensile Strength**

The tensile strength of grouted sand specimens was measured indirectly from a bending test, as depicted in Figure 24(c); in such a test the center portion of the specimen is subjected to pure bending and the tensile strength is deduced from a classical analysis of flexure. The test results shown in Figure 25(b) indicate a strong dependence of the tensile strength on the curing orientation of the grouted specimens. Horizontally cured specimens exhibit higher tensile strengths than vertically cured ones, and the strength difference increases as the water: cement ratio of the grout increases. Horizontally cured specimens injected with 3:1 and 2:1 grouts show strengths about 135% and 35% higher than those of corresponding vertically cured specimens. In contrast, there was no significant strength difference between horizontally cured specimens and vertically cured specimens injected with a 1:1 grout.

These data indicate rather clearly that the sedimentation of microfine cement particles in the pore spaces of grouted sand affect its tensile strength. The observed differences in tensile strength can be attributed to the fact that the failure plane in the vertically cured specimens passes through the weak “layers” formed by the bleed water in the grouted sand (the hydration products are deposited in “layers” parallel to the failure plane), whereas in the horizontally cured specimens the failure plane is forced to pass through the hydration products of the microfine cement or through the interfaces between the hydration products and the sand grains, because the hydration products are deposited in “layers” perpendicular to the failure plane. Decreasing the water: cement ratio of the grout reduces the tensile strength difference due to the reduction in the space occupied by the bleed water, which, in turn, forces the failure plane to pass through more of the hydration products or the interfaces between the hydration products and the sand grains.

**Microfine Cement / Sodium Silicate Grouts**

Sodium silicate based grouts have been used for a long time with much success to stabilize sandy soils, but the organic reagents used to cause gelation have engendered increasing concern with regard to contamination of the groundwater. Non-organic microfine cement can be mixed with sodium silicate and water (and sometimes a
surfactant) to produce a microfine cement/sodium silicate (MC/SS) grout that can be injected into medium to fine sands with virtually no contamination. Liao, Borden, and Krizek (1992) and Krizek, Liao, and Borden (1992) conducted an extensive experimental investigation to evaluate (a) the physical and mechanical properties of various grout mixtures involving MC-500 and sodium silicate and (b) the mechanical properties of sand specimens injected with these grouts.

For the grouts it was found that only certain proportions of cement, sodium silicate, and water show signs of strength development, as indicated in Figure 26 for the condition where equal portions of the silicate:water and water:cement components are mixed. In general, it was found that, for grout mixtures conducive to gelation, the gel time was on the order of minutes and depended primarily on the cement content, with the silicate content producing little effect; however, the hardening resulting from cement hydration required several days. Accordingly, the strength of the grout increased with curing time and required a nonlinear failure envelope to represent its response. Significant strength was obtained in a few days and 80% to 90% of the 28-day strength was attained in two weeks. The prepeak stress-strain behavior was essentially linear, and the modulus increased with cement content. The tensile strength obtained from a Brazilian test was about one-fifteenth of the unconfined compressive strength.

![Figure 26. Recommended Mixing Proportions for Microfine Cement/Sodium Silicate Grouts](image)

Specimens of Ottawa 20-30 sand were injected with these grouts, cured, and subjected to unconfined compression tests, Brazilian tensile tests, and triaxial compression tests. Both the unconfined compressive and Brazilian tensile strengths of the grouted sand reached 80% to 90% of their 28-day values within one week, with the tensile strength being approximately 10% of the compressive strength. Triaxial compression tests showed that the influence of increasing the initial density of the sand
from loose to medium resulted in a strength increase of up to 20%, while pore pressure response and volume change characteristics of grouted sand were similar to those of a very dense ungrouted sand. Unlike neat grout, a linear Mohr-Coulomb failure criterion can be applied to grouted sand, with the peak and residual friction angles virtually unaffected by curing time, initial density, or cement content; however, the Mohr-Coulomb cohesion intercept clearly increased with the cement content of the grout. The $7^\circ$ to $10^\circ$ increase in the friction angle of grouted sand relative to ungrouted medium density sand is due mainly to the adhesion of sporadic patches of grout to sand grains, thereby enhancing the interlocking mechanism.

**Usefulness**

Microfine cement suspension grouts provide a viable alternative to chemical solution grouts when attempting to grout finer sands, narrow rock joints, or voids unfilled by ordinary portland cement grouts. This is because microfine cement grouts (a) possess sufficiently small particle sizes that enable penetration into reasonably small voids, (b) exhibit relatively low viscosity and yield strength properties to enable injectability, (c) are compatible with a variety of additives used to manipulate set time and solids concentration of the mix, (d) manifest sufficient compressive strength and permeability reduction in most grouted formations, and (e) provide a chemically inert means to encapsulate hazardous and radioactive wastes. Although commercially available microfine cements are more costly than ordinary portland cements, there are special situations that justify this added expense; however, viable alternative production methods, as well as increased competition, may serve to reduce the costs difference in the not-too-distant future.

**EPILOGUE**

Slurries in geotechnical engineering are encountered in a variety of circumstances, ranging from the deposition of a clay to grouts for strengthening soils or decreasing their permeability to various wastes from industrial processes and cleaning operations. While the origin, use, and final condition of these slurries may be quite varied, the engineering principles that characterize their behavior are generally applicable to all of them. Notwithstanding the special cases of clay deposition and grouts injected into soils, most slurries in geotechnical engineering are wastes produced by a variety of mining and beneficiation processes. These slurry wastes are often discharged hydraulically into a diked impoundment area, and predicting the time-dependent large-deformation behavior of these sedimented high water content tailings subjected to self-weight consolidation and desiccation is an extremely challenging problem that encompasses new developments in mathematical modeling, laboratory testing and associated interpretations, material property formulations, and engineering judgment. Standard analyses are elusive, and experience dictates a cautious approach to empirical extrapolations. Despite the sophistication that might be incorporated into a given model or the apparent preciseness of its formulation, it is likely that inadequate material property relationships and improper boundary conditions will prevent an accurate prediction of the field response. However, even a correct prediction does not necessarily
validate the model or its input, because the complexity of the many factors involved allows the “right” overall behavior to be obtained by using a fortuitous combination of erroneous material properties, boundary conditions, and/or mathematical formulation. Because the analytical capability of many models exceeds our ability to specify correct boundary conditions and material properties, constantly improving computer technology is fostering a situation wherein we are undertaking “million-dollar solutions” to problems based on “a dollar’s worth of data”. Determining a solution to some particular problem will involve consideration of a broad range of alternatives and the decision must include a generous portion of engineering judgment.

REFERENCES


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