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THE RELATIVE EFFECTS OF INSITU DRYING AND SAMPLE PREPARATION DISTURBANCE ON THE COMPRESSIBILITY OF A COPPER MINE TAILING

By

Fida Hussain

A Thesis Submitted to the Faculty of the

DEPARTMENT OF CIVIL ENGINEERING AND ENGINEERING MECHANICS

In Partial Fulfillment of the Requirements
For the Degree of

MASTER OF SCIENCE

WITH A MAJOR IN CIVIL ENGINEERING

In the Graduate College

THE UNIVERSITY OF ARIZONA

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This thesis has been approved on the date shown below:

Edward A. Nowatzki
E. A. Nowatzki
Associate Professor of Civil Engineering

11 April 1986
Date

With Love,

to

My Parents and My Darling Wife Nadira

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ABSTRACT

The relative effects of insitu drying and sample preparation disturbance on the compressibility of a Southern Arizona copper mine tailings were investigated in the laboratory by conducting a series of consolidation tests. The tailings material tested in this study consisted of silty sand with highly angular grain shapes, rough grain surface texture and 45% by weight passing #200 sieve. The fines fraction contained a relatively small amount of expansive clay mineral, montmorillonite, which caused the tailings to swell when saturated in water.

The consolidation tests were performed on undisturbed as well as disturbed specimens. The undisturbed specimens were retrieved in the form of large size blocks from a preselected location in the tailings dam. Later, test specimens were carved into the consolidation rings from these undisturbed blocks. The disturbed specimens were prepared to a dry density of 95% of the maximum dry density and to the field dry density using impact and static compaction. All specimens were prepared and tested in triplicate. To study the effects of drying and wetting on the tailings (tailings undergo alternate cycles of drying and wetting) the consolidation tests were run on both partially and fully saturated specimens.

The consolidation test results indicate that the tailings do not have any past stress history because the e-log(p) curves did not reveal any preconsolidation pressure. However, the

preconsolidation pressure expected from the specimens during this study was not supposed to be due to a past stress history. Rather, it was to develop due to the effect of desiccation.

For a comparative analysis, the slopes of all the e-log(p) curves were measured. Generally, the slopes of the curves for the undisturbed and statically compacted specimens were greater than those for the impact compacted specimens. This suggests that the former had a relatively higher reduction in the void ratio and hence a greater compression than the latter. Further, all of the saturated specimens underwent more consolidation compression than the unsaturated specimens over the same range of loading. All of the saturated specimens were allowed to free swell under a surcharge pressure of 1/16 tsf prior to applying an additional load increment. The statically compacted specimens tested at an initial water content of 0% produced maximum swell of 6.62%.

CHAPTER 1

INTRODUCTION

Tailings and Tailings Impoundments

Several decades ago, when man felt the need for valuable metals to be used for his industrial growth and realized that they could be found underground, he started digging the earth without realizing that he was embarking on a new horizon of science which would become a well recognized branch of engineering, mining engineering, in the years to come. In the beginning, the operation was simple. It produced only limited quantities of waste products after the mineral value had been extracted. Disposal of these waste products was not a problem either; nearby streams and ditches came in handy and served as dump sites for these unwanted materials. It is human nature not to spend funds on non-profitable operations and this is exactly what prompted the past miners to treat the industrial wastes rather less diligently than good reason would dictate. However, as the mining techniques developed and large mining operations began to turn over huge quantities of waste products, the old techniques of disposal were rendered obsolete. The real turning point came about two decades ago when some of the largest waste disposal facilities failed disastrously with great loss of life. These disasters later caused the mining community to reassess its methods of mine waste disposal. Apart from the structural instability of such facilities, questions about the environmental hazards associated with the carelessly handled mining wastes began to be raised. It was a new situation altogether and, obviously, was bound to panic the whole industry. It warranted a whole new approach towards the design and construction of disposal facilities. However, new design and construction techniques were not possible to optimize without obtaining reasonably good information about the engineering properties of the waste products themselves.

The first factor to be considered in a study of mine waste products is the nature of the processes that produce the mineral wastes. Even a little familiarity with these processes can yield important clues to the nature and geotechnical properties of these waste products. The waste products turned over by a mining operation are called mine tailings. By volume they are usually the greatest. Mine tailings or simply tailings are finely ground mill or mineral processing wastes remaining after extraction of the mineral values from an ore. The copper tailings, used in this study, are an end product of a multiphase mining operation that mainly consists of crushing, grinding, and concentration of the ore which is mined by open pit methods. The last phase, concentration, further consists of several steps that are necessary before the extraction of mineral value can actually take place. The one that affects the physical properties of the tailings from a geotechnical engineering viewpoint the most is the one involving addition of water and chemical reagents, often called hydrothermal solutions, to the ground ore. The process is called concentration by froth flotation and is commonly used in copper mining industry. These hydrothermal solutions change the physical properties of the tailings. For instance, they can convert the feldspars into expansive clay minerals, such as montmorillonite, thus transforming a non-plastic and non-swelling material into a plastic and swelling soil medium.

Central to tailings planning are not only the nature and engineering behavior of the tailings, but also an understanding of the various methods available for tailings disposal. Of the various tailings disposal methods, surface impoundments are the most widely used. Such impoundments are usually built with one of the many raised embankment approaches, Figure 1.1 presents three most frequently used methods of embankment construction. Among the various embankment raising methods, down-stream construction is the most versatile and adaptable to areas of high seismicity and to conditions requiring storage of significant volumes of water, but it may be the most costly method. Upstream construction is the least expensive procedure, but it is suitable only under a very specific combination of tailings gradation, water storage, and seismic conditions. Centerline construction offers most of the advantages of downstream construction but at a more moderate cost. The selection of type of embankment should always be based on specific project conditions, characteristics of each individual mine, and mill site.

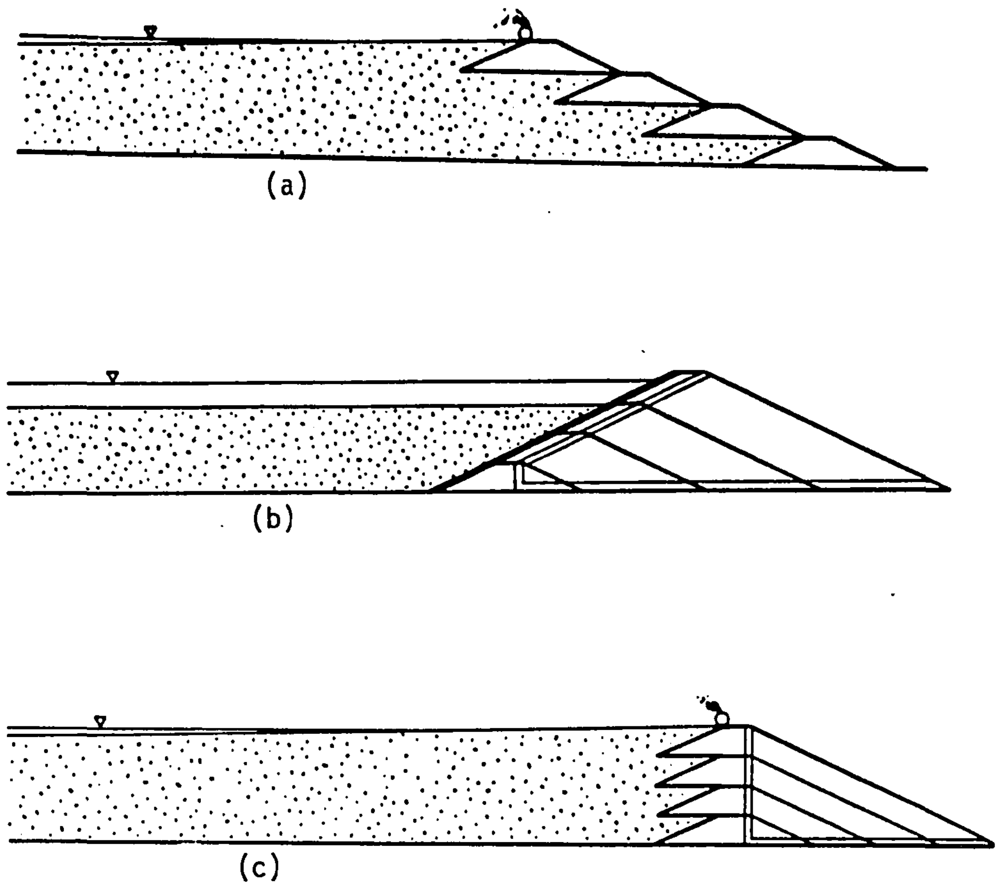

Figure 1.1. Sequential raising, a) upstream, b) downstream, and c) center line, embankments. (After Vick, 1983.)

Other methods of tailings disposal include underground disposal, which can be used only to supplement the surface inpoundments, thickened discharge, and "dry" tailings disposal. The later two methods are, however, untested and need to be carefully evaluated before their implementation.

The mode of deposition has an effect on the physical properties of tailings as well. A particular deposit generally has a specific behavior unique to the type of soil the deposit is made of. The copper mine tailings are deposited in the form of a slurry which is transported through concrete pipes of large diameter to a preselected site on the surface of an impoundment. The slurry is an abrasive material consisting of sand sized particles and "slimes". The "sand" consists of materials retained on a No. 200 sieve, while "slimes" pass No. 200 sieve and contain silt and, sometimes, clay fractions. The sandy tailings tend to settle near the discharge point and form an above-water beach. This is desirable for structural reasons. On the other hand, the slimes, due to their small particle size, remain suspended in the solution and travel farther into the "pond" where they settle as the solution comes to stand still. Where possible, the water is decanted and siphoned back to the mill for reuse. As the hydrothermal solutions accompany the slurry to the pond some physical changes may be expected to take place while the mixture is resting in the pond. It could be readily appreciated that the tailings impoundment can not be treated as one general soil mass. The physical characteristics of the tailings on the beach tend to differ from those of the tailings in the pond.

While quoting the work of a number of researchers, Vick (1983) reported that tailings derived from copper mines have a general particle size range of from 1.00 mm (No. 20 sieve) to 5 μ, which essentially encompases sand, silt, and clay size range. The crushing and grinding processes impart an angular shape and rough texture to tailings. In this respect they are generally different from naturally occuring fine-grained materials formed by chemical weathering processes. The physical properties of tailings could be defined using the phase relationships commonly used in soil mechanics except for a measure of density called "pulp density". Pulp density is defined as the weight of solids divided by the unit weight of the slurry. It is a very important parameter in that it is a crucial factor upon which the transport velocity of slurry in the pipes depends.

With this insight into the processes entailed in producing the tailings it is now worthwhile to look into the design and construction aspects of tailings impoundments.

It is typical of man's nature to treat a waste product with disdain; after all, tailings are a nuisance in all respects and seldom produce any revenue to anyone, unless, of course, they are remined. Consequently, there has been little incentive to spend funds on tailings' disposal systems. However, as it has been proven by past experience, the design and construction of tailings disposal systems should be treated with the same engineering diligence as any other part of the mill flow process.

Tailings impoundments, referred to hereafter as tailings dams, are generally constructed using the "hydraulic-fill" method. In the past some of the large sized tailings dams have been designed by an unintelligent use of the same basic principles as are used for an earth-filled dam. Experience has shown that this approach was incorrect. Unlike an earth-filled dam, constructon of a tailings dam is a continuing process for many years all during which there is an impoundment. Conventional dam design principles can be used only for an initial design which needs to be modified in the light of material behavior as the construction progresses. To accomplish this, other considerations such as metallurgical processes, the chemical nature of tailings, and the unique engineering behavior of tailings of various types must be taken into account. Only such a multi disciplinary approach will define the geotechnical behavior of tailings which, in turn, will evolve into a more practical and safe design and construction strategy.

General Statement of the Problem

In order to model a geotechnical problem successfully and predict field behavior, it is extremely important that a truly representative sample of the soil involved be collected and tested so that the correct values for the controlling soil parameters are used in the engineering computations. For many situations it is mandatory to have an exact account of the in-situ soil properties. This requires field testing or laboratory testing of undisturbed samples. Since the latter is usually too expensive, the former is generally used. The acceptable degree of sample disturbance depends on a specific situation for which the test results are required. There are tests for which sampling and sample preparation do not significantly change the measured soil properties. The more common situation, however, is to have a real or potential variation in the measured soil properties as a result of poor sampling and sample preparation techniques. The determination of tailings properties for stability analyses is an example of such a situation.

Vick (1983) points out that various types of tailings should be studied and treated separately since the geotechnical properties of tailings depend to a large extent on the type of the original ore and the processes that produce tailings after the mineral value has been removed. This implies that procedures of sampling and sample preparation cannot, and should not, be generalized. An individual treatment should be offered in each case.

In the past, only a limited amount of time has been spent on studying the effects of various sample preparation techniques on mine tailings in general and copper mine tailings in particular, Chen (1984) made a recent contribution in this area with respect to stress-strain and volume change characteristics of tailings materials.

Tailings dams situated in arid and semi arid areas like the State of Arizona, the Kingdom of Saudi-Arabia, etc., are subject to high temperatures and rapid variations in moisture content. Such factors appear to have a strong influence on the geotechnical properties of any soil and especially a chemically contaminated soil such as tailings. The current literature does not reveal any evidence that effects of moisture variation, specifically "drying", have so far been investigated together with the effects of sample preparation.

Objectives of Research

The primary objective of this research is to study the effects of drying, caused by natural temperatures, and sample preparation, imparted by some of the more commonly used techniques of sample preparation, such as compaction, on soil structure and compressibility. To accomplish this, a series of consolidation tests were run on undisturbed samples and disturbed tailings specimens which were prepared in the laboratory following the procedures for impact and static compaction. A well known geotechnical parameter, preconsolidation pressure, ρ_c, was chosen as a control parameter to study the effects of drying and sample preparation. Theoretically, highly desiccated, fine-grained soils should exhibit some amount of ρ_c due to surface tension forces induced in the soil structure during drying. It was anticipated that the magnitude of ρ_c would increase as the undisturbed samples dried from field water content to zero water content. At the end of testing, the consolidation test results were to be plotted on conventional e-log(p) coordinates. The preconsolidation pressure for various testing conditions was to be determined by using Casagrande's technique. The magnitude of the ρ_c was expected to be different for undisturbed and disturbed samples. The degree of sample disturbance was to be estimated based on this difference in ρ_c.

Scope of Research Work

Initially, the scope of this research included preparation and testing of both undisturbed and disturbed samples for consolidation characteristics. The study was to be conducted to evaluate the relative effects of in-situ drying of undisturbed samples of mine tailings and the effects of sample preparation on the magnitude of the preconsolidation pressure, ρ_c. When the testing was initiated and consolidation test results were plotted on e-log(p) coordinates, it was realized that, since primary consolidation took place very quickly, it was not possible to obtain the preconsolidation pressure ρ_c. Therefore the parameter selcted for the purpose of this study, ρ_c, was not useful. The problem was further aggravated by the unexpected development that the tailings investigated contained expansive clay minerals.

Therefore it was decided to perform a series of consolidation tests on both saturated and non-saturated specimens and use the slopes of the e-log(p) curves to study the effects of in-situ drying and sample preparation on tailings compressibility. Furthermore, it was also decided to investigate the swelling behavior of the tailings.

The actual scope of the work was set as follows:

1. Establish a moisture-density relationship using impact compaction by ASTM D1557 procedures.
2. Select two moisture contents, one dry of optimum and one wet of optimum, corresponding to 95% of $\gamma_{d(max)}$. This was done so as to duplicate Chen's (1984) procedure for preparation of laboratory samples.
3. Bring the moisture content of undisturbed samples to the moisture contents as selected in Step 2. Also prepare the laboratory specimens at the same moisture contents as selected in Step 2 using impact and static compaction. Figure 1.2 presents the range of these moisture contents.
4. Prepare consolidation specimens from both undisturbed and disturbed (laboratory prepared) samples and run standard as well as non-standard consolidation tests. The standard tests were to include loading of fully saturated specimens while the non-standard tests were to be run on as-prepared/molded partially saturated specimens. This was to be done to duplicate the field

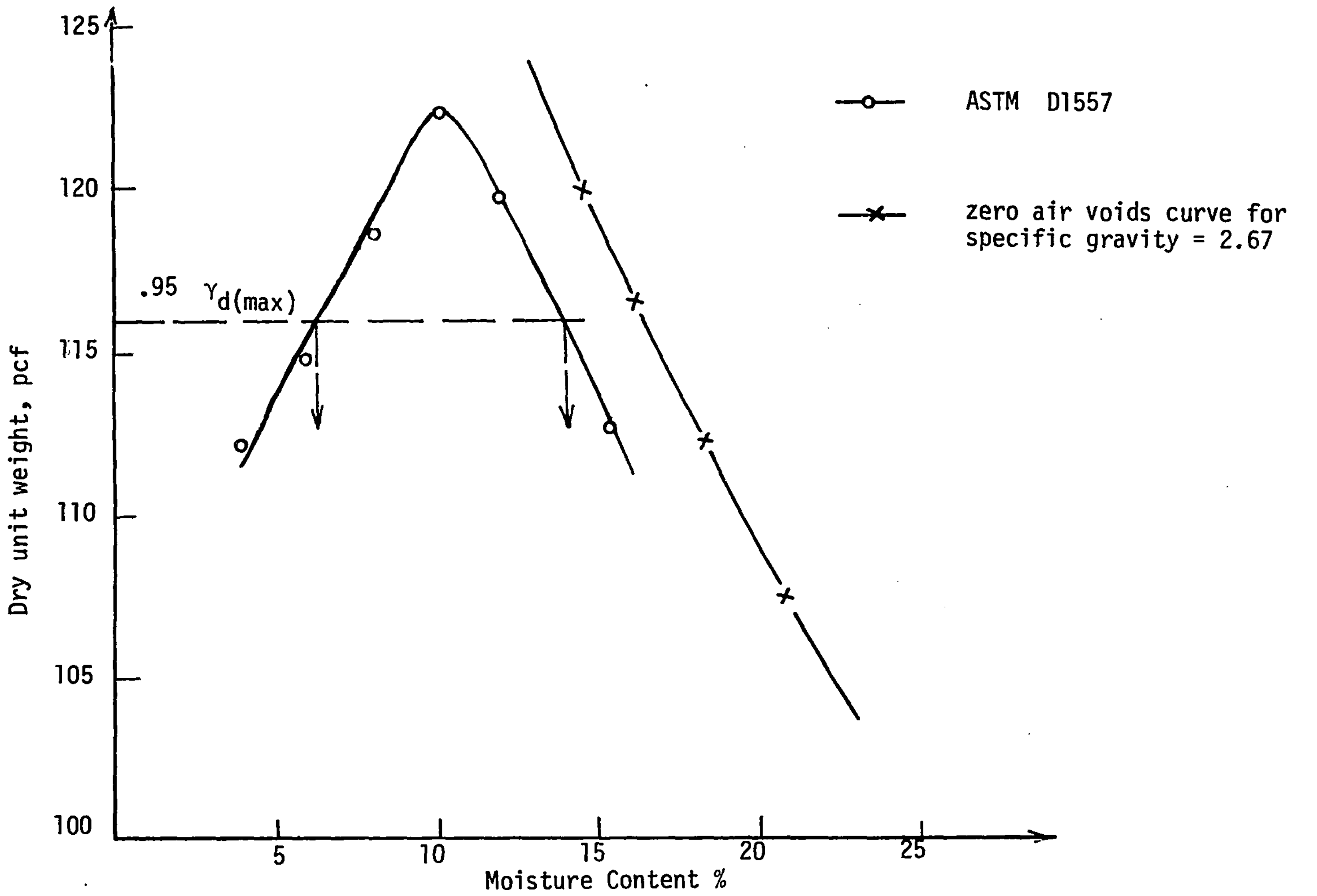

Figure 1.2. Range of moisture contents, corresponding to 95% of maximum dry density, for consolidation test specimens.

conditions. Triplicate samples were to be tested to ensure statistically meaningful data.

5. Evaluate the relative effects of drying and sample preparation disturbance on tailings structure and compressibility for both undisturbed and disturbed soil specimens.

CHAPTER 2

LITERATURE REVIEW

This chapter presents a review of the literature regarding:

1) The effects of drying and wetting on the physical behavior of soils;
2) The effects of sample disturbance on testing in general;
3) The effects of sample preparation on test results in general; and
4) Partially saturated soils.

Effects of Wetting and Drying on the Physical Behavior of Soils

The earth's crust in tropical and semitropical regions undergoes large fluctuations in its water content as the season changes. Literature suggests that such changes lead to degradation of the parent rock to form soil and to the formation of interparticle bonds that result in the aggregation of the soil particles and in interparticle bonding. These effects can impart an intrinsic effective stress to the natural soil. A tailings dam is similar to the earth's crust in that it undergoes a great number of cycles of wetting and drying as its construction progresses for many years, i.e., a layer of deposited slurry is permitted to dry and stabilize before an overlay of fully or partially saturated slurry is spread over it. Thus the tailings are subjected to cyclic wetting and drying. It could not be traced in the literature that a concerted effort to study this phenomenon was made in the past. However, some literature covering the effects of wetting and drying on soils in general is available.

The soils in large parts of the earth's land surface are partially saturated to considerable depths and are known as desiccated soils. These soils are subjected to repeated wetting and drying as a result of climatic changes. Russam (1965) points out that in arid areas with deep water tables, soil humidities are controlled by atmospheric humidities to depths of 20 feet or more. Drachev (1933) and Grant (1974) report that the action of water on the soil consists of solution, hydrolysis, and the decomposition of the soil compounds. Adlerfer (1964) reports that alternate wetting and drying result in the buildup and breakdown of soil granules and is partly responsible for a continuous increase in the water stable aggregates. The extent to which discrete or elementary, nonaggregated soil particles are grouped to form larger stable aggregates is a function of several factors such as the grain size distribution, amount of clay, type of clay minerals present, nature of the saturating cation populations, and presence and solubility of salts as well as organic decomposition and organic cementing agents.

Among several other factors, wetting and drying have been reported to cause grouping and aggregation of fine soil particles into large size particles. Lambe (1960) reports that wetting and drying are one of the reasons why individual soil particles can become permanently linked together and in effect grow into large particles. Ingles (1961) reports that chemical attack by chemical reagents on the majority of soil components is capable of producing covalently bonded, macrolatticed compounds possessing considerable strength, and under controlled conditions this can be achieved by alternate cycles of wetting and drying. Grant (1974) reports that repeated wetting and drying of some soils can result in increased strength due to aluminum oxide bonds being generated in the initial states, and also due to recrystalization of ferric oxide in the later stages. Kenney (1962) and Loiselle (1971) report that in certain cemented clays, Al, Fe, Mg, and Ca compounds act as cementing agents and influence their compressibility. Removal of these bonds by chemical treatment results in loss of the apparent preconsolidation pressure. Blight (1966) points out that the effect of desiccation is similar to that of heavy overconsolidation.

Allam and Sridharam (1981) have studied the effects of wetting and drying on the shear strength of a soil containing 70% sand, 25% salt, and 5% clay - the composition very much resembles that of the tailings materials investigated for this study. They prepared the specimens for $\overline{CIU}$ triaxial compression tests by using static compaction. To study the effect of interparticle bonds on the material properties, they treated some of the specimens with 250 ml of 0.2 N EDTA (disodium salt of ethylenediamine tetra acetic acid) solution after the specimens had undergone 50 and 60 cycles of rewetting. The EDTA tends to remove materials such as carbonates, gypsum, and iron components, that act as particle cementing agents. As suggested by Konder (1963) and Sridharam (1972), Allam, et al. (1981) transformed the deviator stress-strain curves obtained through laboratory tests ($\overline{CIU}$) into rectangular hyperbolas to obtain values of initial tagent modulus (ITM). The results of their investigation are summarized as follows:

1. Repeated cycles of wetting and drying increase the stiffness of the soil fabric so that the soil they tested eventually became brittle.
2. Compressibility of the soil fabric decreases due to increase in the intrinsic effective stress brought about by repeated wetting and drying.
3. The increased intrinsic effective stress increases the shear strength.
4. The cohesion intercept appears to have increased as the wetting and drying is repeated.
5. The specimens treated with EDTA show the increase in ITM ratio due to wetting and drying to be due to changes in soil fabric. However, the shear strength was found to be lower for these specimens.

Shrinkage of soils due to drying can also cause changes in the physical behavior of fine grained soils. Mitchell (1975) postulates that the drying shrinkage of fine grained soils result in particle movements that are due to pore water pressure tensions developed by capillary menisci. The shrinkage lowers the void ratio and results in a reduced soil volume. Shrinkage has a pronounced effect on preconsolidation pressure, ρ_c, too. The magnitude of ρ_c increases as the shrinkage intensifies.

Based on the preceding discussion, it could be concluded that soils subjected to alternate cycles of wetting and drying, as in case of tailings, experience a noticeable change in their engineering properties foremost among which are strength and deformation characteristics.

Effects of Sample Disturbance on Testing in General

The significance of the effects of sample disturbance on laboratory test results was recognized more than three decades ago. Hvorslev, as early as 1949, demonstrated that the sample disturbance caused by the sampler and handling, etc. affect the geotechnical parameters of the soil. For instance, an increase in the deformation modulus of the soil is found to be linked with sample disturbance. In their classic papers Skempton and Sowa (1963) and Noorany and Seed (1965) suggest that sample disturbance consists of two components, one associated with mechanical disturbance caused by the sampler, handling, etc. and the other due to the release of total stress. In practice, it is impossible to eliminate sample disturbance totally. Even if intensive care is exercised so that mechanical disturbance could be avoided, the release of the total stress is unavoidable. Ladd and Lambe (1963) define a sample that has not been disturbed by boring, sampling, and trimming but has been subjected to stress release as a "perfect sample" or a "virgin sample". In the following sections, the components of sample disturbance are discussed separately.

Mechanical Sample Disturbance

The mechanical disturbance is caused by a number of factors. Broms (1980) reports that the sample disturbance is caused by drilling, sampling, transportation, storage, sample preparation, and testing of the samples. The disturbance caused by the sampler depends on the ratio of the sampler wall thickness (t) to its diameter (d). The higher the t/d ratio, the higher the degree of disturbance of the sample will be. Kallstanius (1963) studied the disturbance caused by transportation; he vibrated and dropped samples of soft clay. He found that shock-loading effects produced by dropping the samples from a height of 1.0m on an asphalt floor caused an increasing reduction in the undrained shear strength with the magnitude of the reduction increasing with liquidity index, IL, of the soil. Some of vibrated samples showed reduction while others showed increase in the shear strength.

Sample Disturbance Due to Stress Release

In-situ, major and minor principal stresses generally differ. Hence a unit volume of soil in-situ is almost always under an anisotropic stress state. When a sample is removed from the ground, relief of stress takes place bringing about pore pressure and volumne changes. Holtz and Kovacs (1981) report that the various states of stress could be described schematically using the concept of stress paths. Figure 2.1 shows the stress paths for the stress states both before and after the sampling process. The stress path followed by line BC shows that the soil specimen ends up someplace on the hydrostatic axis where $q = 0$ representing $\sigma_1 = \sigma_3$. Brom (1980) reports, when the sample is brought to the surface during a field exploratory boring, the porewater expands and bears an effect on the deformation properties of the soil. However, this effect is minor as compared with the effect on deformability of a sample by escape of dissolved air or gas in the pore water.

The most important soil parameters liable to modification by either form of sample disturbance are:

1. The true cohesion and angle of internal friction;
2. The undrained shear strength of saturated clays; and
3. The compressibility coefficients.

Begemann (1977) reports that most of these parameters are largely dependent on the type of soil and are related to density. For sands, for instance, the density will have an appreciable influence on the

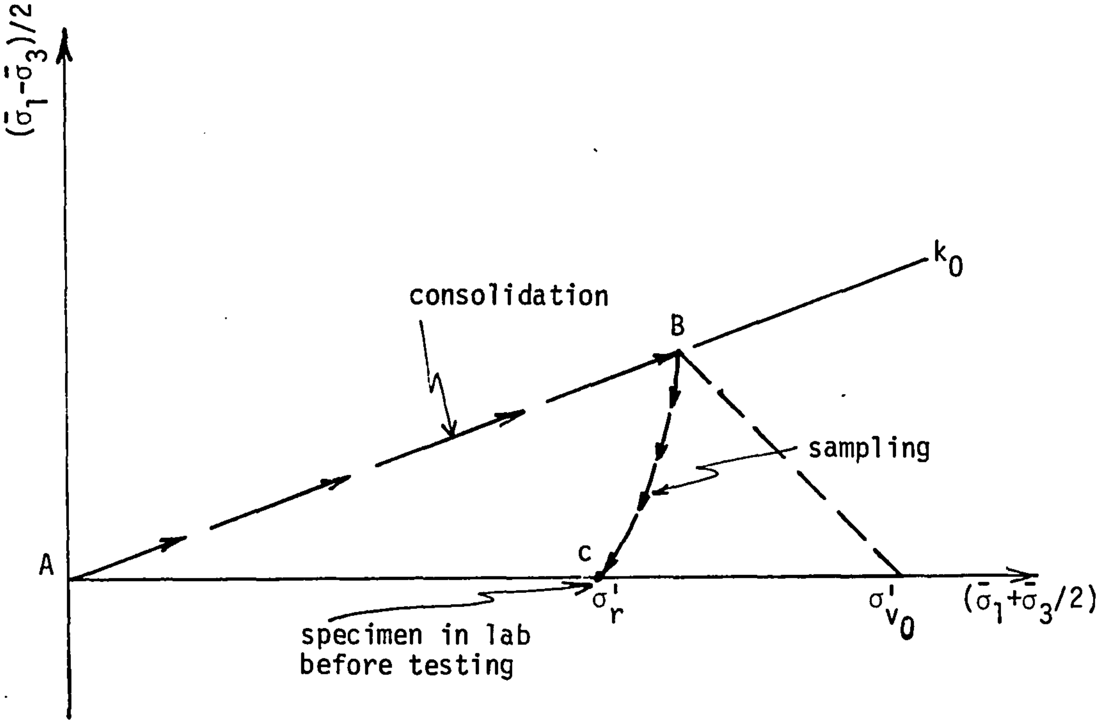

Figure 2.1. Stress paths during sedimentation and sampling of normally consolidated clay, where $K_0 < 1$. (After Holtz and Kovacs, 1981.)

true angle of friction. For clays however, within normal ranges of stress, there is a very small effect if any at all. So density can be a very important factor for evaluating test results in connection with sample disturbance. The in-situ preconsolidation stresses, which for instance define for clays the undrained shear strength (C_u), have become a more important factor. In both sands and clays, changes in densities will have a big influence on this in-situ preconsolidation pressure.

Begemann (1977) recommends that the following laboratory tests be performed in order to evaluate the presence of possible sample disturbance:

1. Density determinations;
2. Unconsolidated and consolidated undrained triaxial tests for saturated soils (with pore water pressure measurements),
3. Stress controlled triaxial tests, and
4. Oedometer tests.

The following discussion deals with some of the pronounced effects of sample disturbance on the engineering properties.

The two most important aspects of soil behavior from the geotechnical standpoint are strength and deformation. The parameters by which they are characterized are very much influenced by sample disturbance. Their erroneous evaluation may bring to bear negative effects on the structure in question. Sample disturbance reduces the shear strength as well as modulus of elasticity of a soil. An analysis based on the shear strength measured in the laboratory can be misleading. For example the bearing capacity of a shallow foundation under static loading can be underestimated by upto 30% if the shear strength parameter ϕ is underestimated by only 20%. Likewise, the calculated "immediate" settlement for a stiff clay in some cases may be three to six times larger than the real settlements. Broms (1980) reports that the settlements of structures founded in stiff to hard clay are often four to eight times larger than the measured settlements even if the results have been corrected for sample disturbance.

Marsland (1971) reports that the modulus of elasticity of samples that had been stored a few weeks was only half of that for samples tested four days after the sampling. It has been observed that large changes of the moisture content can take place during storage and transport if the "tube and/or block samples" are not secured carefully. This has a direct effect on the soil properties. Bjerrum (1973) found for quick clays that the undrained shear strength had decreased 15% after only three days of storage. This effect increased with decreasing plasticity of the clay. Jerbo, et al. (1961) reported a decrease in sensitivity of soft clays during storage.

Sone (1971) studied the effects of the force required to extrude a soil sample of clayey silt. They found that undrained shear strength of specimens taken was reduced by as much as 10 to 20%

by the extraction of specimens taken 10 cm to 20cm from the bottom of the sample tube. The maximum strain during the extraction was approximately equal to the failure strain.

Effects of Sample Preparation on Test Results in General

The precise mode of formation of the soil, both in naturel and in the laboratory, tends to have a significant effect on structure and hence deformation strength characteristics. The variation in the behavior of granular materials resulting from different methods of sample preparation have recently received considerable attention from the geotechnical engineering community. Though much of the work has been directed towards the use of laboratory tests to evaluate liquification potential of cohesionless soils, the findings also provide valuable insight into the more general behavior of granular deposits.

A review of the pertinent literature suggests that the variations in the physical behavior of a soil are greatly influenced by the "soil fabric" or "soil structure". "The arrangement of solid phase and the pore spaces located between its constituent particles is termed as soil fabric, on the microscale, or soil structure, on the macroscale" (Mitchel 1976). Here both of the terms namely soil fabric and soil structure are used to imply the same meanings. A number of researchers have consistently demonstrated that for the same density different methods of sample preparation generate different soil structures. This leads to causing a variation in the measured soil properties. Oda (1972) studied two sets of samples having the same density but prepared by two different methods of soil placement. In the "tapping" method of preparation, the sand was compacted by tapping the sides of the sample mold whereas the "plunging" method of preparation involved compaction with a small diameter rod. The tapped samples had a significantly higher modulus than the plunged samples. They also had a preferred orientation of interparticle contacts in the vertical direction whereas the plunged specimens did not. This behavior can therefore be explained by differences in the soil structure developed during sample preparation. Mahmood et al. (1975) conducted a similar study on Monterey sand, a medium beach sand with rounded and only slightly elongate particles, to investigate the effects of sample preparation techniques on the soil fabric and compressibility. They prepared the samples at an in-situ density by methods of pluviation (dry pouring) and vibration. They found dry poured samples to have increased compressibility compared to vibrated specimens of the same density. Figure 2.2 summarizes their results.

Various investigators have attempted to study the effects of soil fabric on liquefaction potential of a soil. The resistance to liquefaction depends on the method of sample preparation in addition to several other aspects. Ladd (1974) indicates that soil fabric influences the volume change tendencies of a soil and hence affects

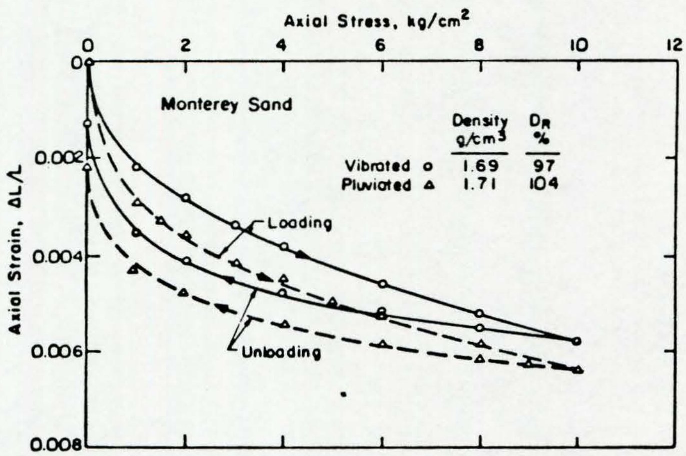

Figure 2.2. Effects of method of specimen preparation on the axial compression of dense sand. (After Mahmood et al., 1975).

its liquefaction potential. Figure 2.3 illustrates the relationship between cyclic stress level and number of cycles to cause liquefaction for samples of three sands prepared to the same density by two different methods, "wet tapping" and "dry vibration". The applied stress ratio, $\pm \sigma_d/2\bar{\sigma}_c$, is plotted versus log N, number of cycles required to obtain peak-to-peak axial strain of 10%. In each case, the specimens prepared by the dry-vibration method appear to be weaker and hence more easily liquefiable than the specimens prepared using the wet-tamping method.

Shear strength characteristics of cohesionless materials under cyclic loading have been characterized by a large number of investigations in terms of soil fabric resulting from different methods of soil deposition. Pyke (1973) noted that vibrated samples of Monterey sand exhibited less settlement during drained cyclic loading in simple shear tests than specimens formed by dry pouring. Ladd (1974) found the undrained cyclic strength of triaxial samples prepared by wet tamping to be approximately double the strength of that of specimens prepared using the dry vibration method for three medium to dense SP-SM sands (Figure 2.3). Similar trends with smaller differences were reported by Silver and Park (1976) for a unifom angular quartz sand with relative density, D_r, of 60%. Wet tapping also increased the undrained cyclic modulus by 20 to 40%. Mulilis et al. (1975) performed a comprehensive study of the influence of sample formation on the undrained cyclic behavior of

σ_d = deviation stress

$\bar{\sigma}_{3c}$ = minor principal stress (effective)

$\bar{\sigma}_d$ = preconsolidation stress (effective)

D_r = relative density

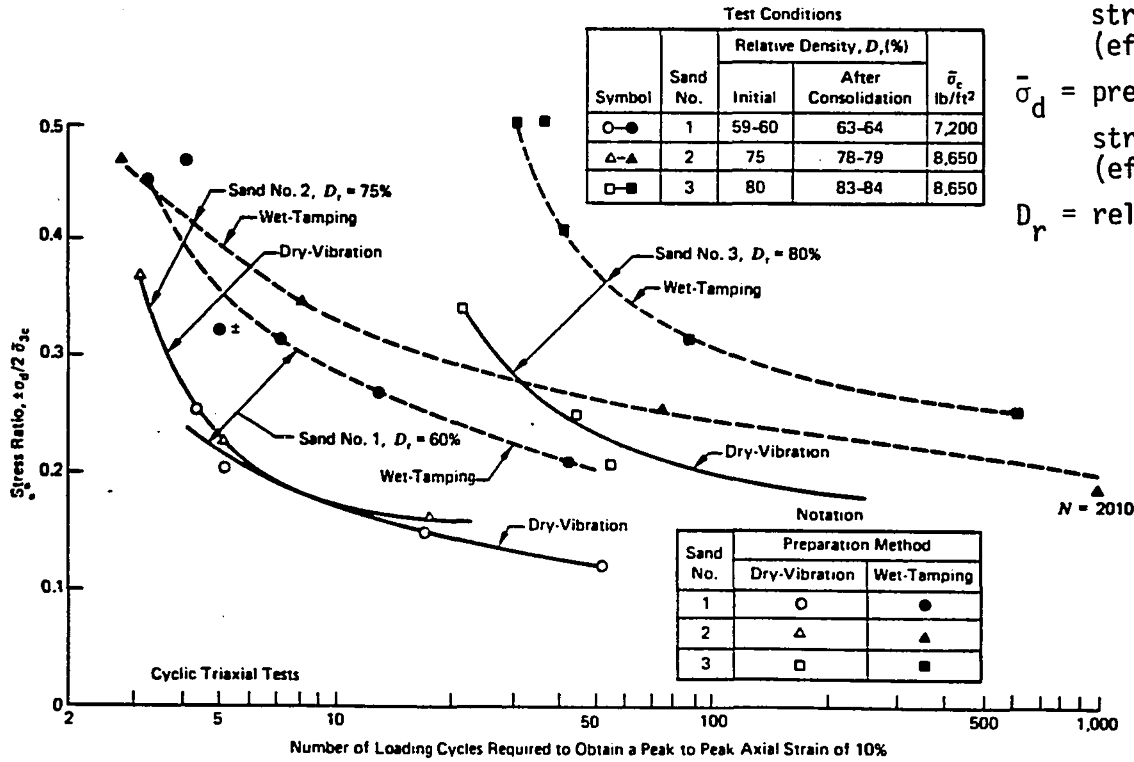

Figure 2.3. The influence of sand specimen preparation method on liquification behavior. (After Ladd, 1979.)

Monterey sand. Triaxial test specimens were prepared using a wide range of techniques, including pouring through air and water, dry and moist tamping, and vibration of dry and moist sand. Each procedure produced different behavior with moist samples compacted by high frequency vibration exhibiting the highest strength and air poured samples the lowest.

The above mentioned considerations also apply to the deposition of in-situ granular soils. The deposits formed by different processes tend to demonstrate varying physical behaviors. Japanese Society of Soil Mechanics and Foundation Engineering (1976) cites a very interesting case history of a reclamation project at Ohgishima Island near Tokyo. Sand was first deposited from barges onto the sea bed at a water depth of some 10 to 20m. When the sand surface reached about 3m below sea level, a sand-water slurry was pumped through a pipe and discharged into the fill area. The operation continued until the fill reached 5m above original sea level. In situ densities were measured using a Bishop type sampler. A distinction in relative density was observed when the sand-water slurry was discharged into air rather than water. To see the effect of driving piles on changes in D_r, compaction piles (0.4m diameter with 1.73m spacing) were installed. Because the relative density was determined indirectly through N values, compaction and vibration induced by the piles did not affect the relative density significantly. Yet the SPT resistance increased beyond 10m. The only possible explanation is the variation in soil fabric. The higher N-values may have resulted from increased horizontal stresses after installation of the compaction piles. Also, according to Toki and Kitago (1974), small amplitude vibrations can cause large increases in the modulus of sands and thus presumably the N-values.

In summary, the literature pertaining to the effects of sample preparation on various geotechnical parameters appear to indicate two things:

1) The majority of the geotechnical properties both measured in the laboratory and in the field depend on soil fabric which in turn depends on the method of sample preprations.
2) For cohesionless soils, the relative density should not be completely relied upon for soil characterization. It should be supplemented with information on the soil fabric as well.

Partially Saturated Soils

In the 1930's, Karl Tarzahgi, who is recognized as founder of modern soil mechanics, produced convincing evidence to support the concept of effective stress. The concept indicated that for a saturated soil, the difference between the total stress and the pore water stress formed an independent stress variable which controlled the deformation and strength characteristics of a soil. This concept has been proven to be the key in formulating and analyzing soil mechanics problems. Consequently, soil mechanics has been primarily concerned with the stress deformation and strength behavior of saturated soils.

But in reality, the situation encountered in practice involves non-saturated or partially saturated soils. The partially saturated soils are in the form of either man made structures such as earth-fill dams, roadway embankments, etc. or natural deposits. In arid and semi-arid areas, the land surface is exposed to desiccating influences that leave the land cracked and unsaturated. Hamilton (1969) points out that volume changes are result of environmental changes over a period of time. The soils that are highly susceptible to volume change due to change in the environment are either expansive soils, having considerable fraction of swelling and shrinking clay minerals, or residual, desiccated soils. Bishop and Blight (1963) suggest that the degree of partial saturation of a soil deposit depends on the atmospheric humidities. The soil deposits situated in relatively dry climates tend to have higher negative pore water pressures than a deposit exposed to humid atmosphere.

The literature review reveals that numerous efforts have been and are being made to establish a parallel of the effective stress concept for the partially saturated soils. A number of equations that consider an air phase in the traditional effective stress relationship have been developed in the past. However, none of these equations has been proven to be completely successful in practice. Lambe and Whitman (1969) expressed a similar opinion. Fredlund

(1981) has made a recent contribution towards advancing our understanding of the partially saturated soils. Parts of his works and his critique of others' works shall be quoted, where appropriate, in this report with regard to partially saturated soils.

Definition of a Partially Saturated Soil

Generally soils whose pore volume is only partially filled with water are called partially saturated soils. Lambe and Whitman (1969) state that a partially saturated soil contains "three distinct phases: solid (mineral particles), gas, and liquid (usually water)". Sisler et al. (1953) postulate that "many mixtures are heterogeneous, consisting of two or more physically distinct portions, differing in properties and composition and separated from each other by definite bounding surfaces". Fredlund (1981) has deduced from this statement that an unsaturated soil can be visualized as a four-phase medium; the fourth phase would be the air-water interface. In surface chemistry (Matyeveev, 1967), this interface is referred to as the "contractible skin". Figure 2.4 illustrates the four phases as they exist in an element of an unsaturated soil. The thinness of the contractile skin makes its physical subdivision unfeasible and unnecessary from the viewpoint of volume-weight relationships. The four phase system therefore reverts to a three phase system.

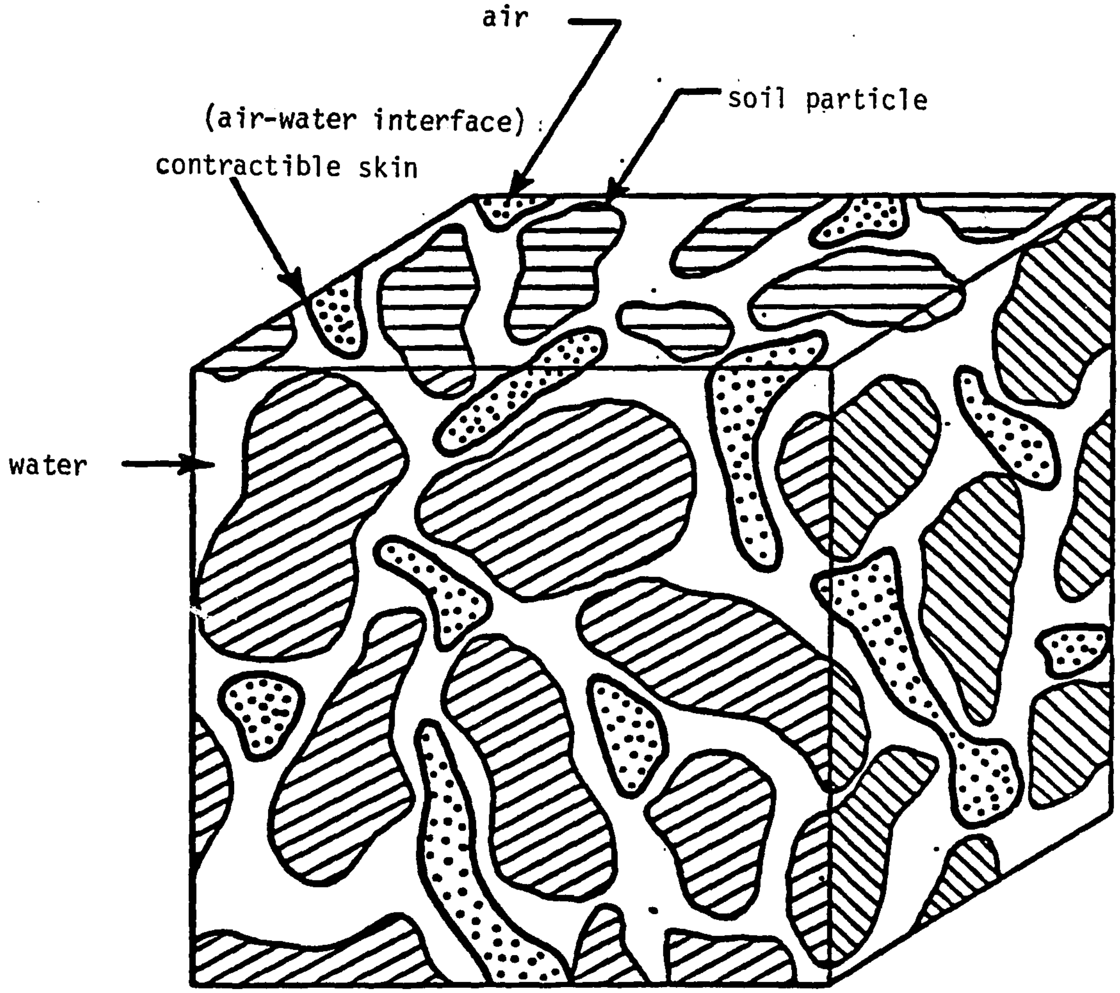

Figure 2.4. An element of an unsaturated soil, continuous air phase. (After Fredlund, 1981.)

Development of a Partially Saturated Soil Deposit

Most of the natural soil deposits are originally saturated. Lacustrine deposits, for example, are deposited at water contents above the liquid limit and then consolidated by the weight of the overlying sediments. The drying up of the lake and subsequent evaporation commence desiccation of the sediments. The pore water pressure becomes negative with respect to the atmospheric pressure above the water table giving rise to consolidation and eventually desaturation of the sediments. As plant life begins to grow on the surface, evapotranspiration further dries out the soil. The tension in the water phase acts in all directions and can readily exceed the lateral confining pressure in the soil mass thus causing cracking. The situation of a water retaining embankment, subjected to draw-down, and a tailings dam, whose constituents transform from nearly saturated to unsaturated form, is analogous to the foregoing.

With this brief introduction, one may be better able to understand the physical behavior of an unsaturated particulate medium. Deformation and strength are the two basic soil parameters that need be considered in the formulaton of a number of geotechnical problems. As stated earlier, both of these are founded on the concept of effective stress. It will be therefore useful to present a review of the efforts done so far to state the concept of effective stress for the partially saturated soils. This will be followed by a brief discussion on the deformation and strength behavior of the partially saturated soils.

Effective Stress Concept

Terzaghi (1936) asserts that the behaviorial considerations of a saturated soil are governed exclusively by the difference between the total and pore-water pressure. That is:

$$\sigma_1' = \sigma_1 - u \tag{2.1}$$

$$\sigma_2' = \sigma_2 - u \tag{2.2}$$

$$\sigma_3' = \sigma_3 - u \tag{2.3}$$

where

σ_i = total stress
σ_i' = effective stress
u = pore water stress.

where subscript i denotes the principal stress directions. He suggests that "all the measurable effects of a change in stress, such as compression, distortion and a change in shearing resistance are exclusively due to changes in the effective stresses σ_1', σ_2', and σ_3'.

Terzaghi's expression was refined by other researchers like Skempton (1961). He argued to include the grain-to-grain "contact area", a, in Terzaghi's effective stress equation for the saturated soils. His equation is:

$$\sigma' = \sigma - (1 - a)u_w. \tag{2.4}$$

However, Fung (1969) disagreed with Skempton and advocated that the description of stress state must be independent of the soil properties such as area. Fredlund (1981) indicated that inclusion of contact area renders the effective stress equation a constitutive relation rather than a description of the stress state.

Refinements or modifications to Terzaghi's equation were not suggested for unsaturated soils until the late 1950's. Bishop (1955) suggested a "tentative" effective stress equation for unsaturated soils as

$$\sigma' = \sigma \; u_a + \chi(u_a - u_w) \tag{2.5}$$

where

σ' = effective stress

σ = total stress

u_a = pore air pressure

u_w = pore water pressure

χ = a parameter, $0 < \chi < 1$.

Lambe and Whitman (1969) call for a further investigation of this expression. They argue that only a thorough testing of the equation for field applications might attest to the validity of the expression.

Croney et al. (1958) presented another modified effective stress equation:

$$\sigma' = \sigma - \beta' u_w \tag{2.6}$$

where

β' = the holding and bonding factor which is a measure of the number of bonds under tension effective in contributing to the shear strength of the soils.

Atchison (1961) suggested the following modification to Terzaghi's equation

$$\sigma' = \sigma + \psi p'' \tag{2.7}$$

where

p'' = pore water pressure deficiency

ψ = parameter with values $0 < \psi < 1$.

Jennings (1961) proposed a new effective stress expression:

$$\sigma' = \sigma + \beta p'' \tag{2.8}$$

where

p'' = pore water pressure deficiency

β = a statistical factor proposed to be measured experimentally.

All of the above equations for unsaturated soils are equivalent when the pore air pressure is atmospheric, $\chi = \beta' = \psi = \beta$. All of these equations were proposed on the assumption that only one stress variable is required to describe the behavior of an unsaturated soil. Jennings and Burland (1962) argued, however, that effective stress given by Bishop's equation was not uniquely related to void ratio below a critical degree of saturation. Bishop and Blight (1963) re-evaluated their equation and noted that variation in the $(u_a - u_w)$ term did not result in a direct change in effective stress. They pointed out that the path of the two compnents, $(\sigma - u_a)$ and $(u_a - u_w)$, must be considered in stress-volume change predictions. Burland (1964, 1965) suggested that in Bishops' equation, stress systems be separated into internally applied water stresses and internally applied loads.

Fredlund et al. (1973) states that an approach similar to that used for the saturated soils can be used to analyze the equilibrium of an unsaturated soil element. He agrees that linear equations can be written for each of the four phases of an unsaturated soil from which a total equilibrium equation can be formulated. In 1977, he showed from a stress field analysis that any two of three possible stress variables can be used to define the stress state in an unsaturated soil. Possible combinations are: 1. $(\sigma - u_a)$, 2. $(\sigma - u_w)$, and 3. $(u_a - u_w)$.

Compression of Partially Saturated Soils

Lambe and Whitman (1969) presented a schematic of how a partially saturated soil passes through various stages during an oedometer test. Figure 2.5 depicts what they call a qualitative picture of compression of a partially saturated soil.

As they indicated in the figure, initally the soil is quite compressible since pore air-water offers little resistance until the degree of saturation exceeds 85% (compare total stress, void ratio, and pore pressure between times t_1 and t_2). During the initial phase of loading the effective stress increases while the pore pressure remains almost unchanged. Furthermore, it does not matter whether drainage is permitted or not because very little water will be squeezed out during this phase.

If the load is increased to such an extent that all of the air in the pores is released or compressed and dissolved, the soil will become fully saturated. Now any further increase in load will

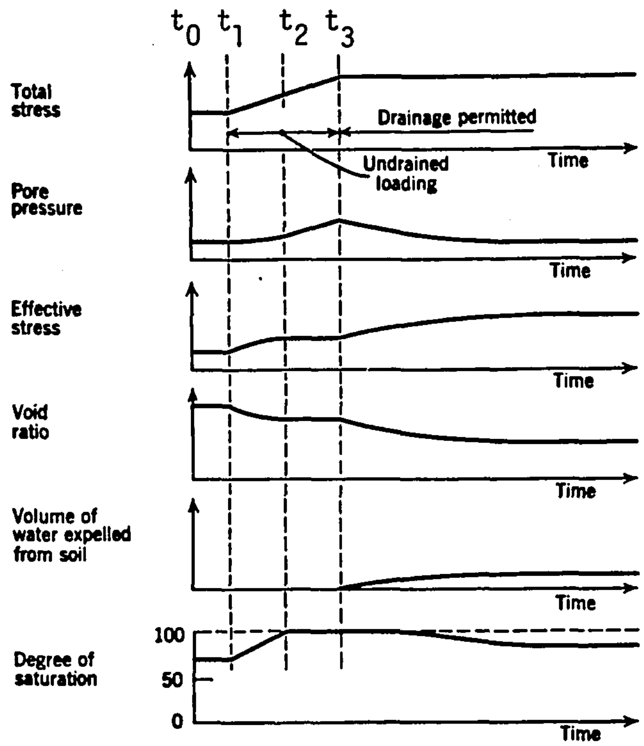

Figure 2.5. Compression of partially saturated soil in an odeometer. (After Lambe and Whitman, 1969.)

be carried by the pore fluid. If drainage is permitted, water will flow from the specimen as in a consolidation test upon an initially saturated soil. Once the load reaches its maximum value and becomes constant (at t_3), a drained soil may again become only partially saturated due to the squeezing of the air-water mixture out of the soil pores.

As pointed out by Lambe and Whitman and many other researchers, a detailed quantitative characterization of volume-change of a partially saturated soils is difficult. The difficulty stems from the poorly understood interactions among the various phases in the soil. This poor understanding renders the conventional effective stress concept virtually inapplicable for the partially saturated soils. In a general way, the principle of effective stress must apply. The volume of soil at any time is related to the effective stress at that time. Both the pore air pressure and the pore water pressure must be considered when evaluating effective stress by Bishop's equation. However, the difficulty faced in the evaluation of the parameter χ makes the effective stress concept very complex. To avoid an erroneous estimation of the compression of a partially saturated soil, the best approach is to apply, both initially and finally, the total stress, pore air pressure, and pore water pressure existing or expected in-situ.

Strength of Partially Saturated Soils

As noted earlier, the shear strength of partially saturated soils is related to effective stress. Fredlund (1981) has developed a theory for shear strength of partially saturated soils using two combinations of the stress variables $(\sigma - u_a)$ and $(u_a - u_w)$, and $(\sigma - u_w)$ and $(u_a - u_w)$. He notes that regardless of the combination of stress variables used to define the shear strength, the value of a shear strength obtained for a particular soil with certain values of σ, u_a, and u_w must be the same. Furthermore, he shows that the cohesion is dependent on the matric suction, matric suction increases the cohesion of a partially saturated soil. Matric suction is the component of the total suction prevailing in a partially saturated soil. It is defined as the mechanism of water retention by surface tension and water fixation by polar adsorption and is related to the negative pore water pressure in soils. It is expressed as:

$$\tau_m = u_a - u_w$$

where

τ_m = matric suction

u_a = pore air pressure

u_w = pore water pressure

Cohesion, c, has two components:

$$c = c' + [u_a + u_w] \tan \phi^b \tag{2.9}$$

where

c' = intercept of τ vs. $(\sigma - u_a)$ plot when two stress variables are zero,

ϕ^b = friction angle with respect to changes in $(u_a - u_w)$ when $(\sigma - u_a)$ is held constant.

Based on the above, he suggested the following equation for shear strength:

$$\tau = c' + [\sigma - u_s] \tan \phi' \tag{2.10}$$

Fredlunds' approach appears to be reasonable. Apparently, it seems to be paralled to the conventional shear strength theory as used for saturated soils. According to him, his theory reverts to the saturated case when matric suction is zero (u_a becomes equal to u_w). The argument is well founded in that matric suction is a phenomenon attached only to partially saturated soils.

Lambe and Whitman (1969) indicate that $\phi = 0$ concept does not in general apply to partially saturated soils. While quoting a typical relation between confining stress and strength during $\overline{uu}$ tests, they state that for the lower range of confining stresses, the soil remains partially saturated and effective stress increases as the confining stress is increased. Once the confining stress becomes large enough to cause full saturation, further increase in confining stress does not cause the effective stress to increase and from this point $\phi = 0$ concept does apply. They have quoted the following relationship between undrained strength and total stress:

$$\tau_{ff} = c_u + \sigma_{ff} \tan \phi_u$$

where

c_u, ϕ_u = total stress strength parameters. They depend on range of σ_{ff} which is of interest.

They warn to exercise caution in duplicating the expected field conditions (σ, u_a, u_w) when evaluating the undrained shear strength of partially saturated soils.

Summary of Main Points

The discussion presented in the preceding sections may be summarized as follows:

1. Drying or drying and wetting cycles tend to change the physical behavior of a soil. The drying under natural temperature causes

partial saturation, negative pore water pressure, and surface tensions that affect shrinkage in soil and hence a reduction in the volume. The alternate cycles of wetting and drying have been found to increase the stiffness of a soil. Both drying and wetting and drying have been reported to increase the preconsolidation pressure, ρ_c, of the soils.

2. Sample disturbance has a considerable effect on the laboratory testing of soils in general. It has been observed to increase or decrease the magnitude of deformation and/or strength of the soils. Its evaluation with respect to the soil property being considered may be made using a number of field and laboratory tests such as density measurements and oedometer tests. Omitting links of the chain, "sampling to testing", will result in testing of a perfect sample, the one having the stress-release related component of disturbance only.

3. Method of sample preparation affects the soil structure and hence the physical behavior of the soils. It has been successfully demonstrated that different methods of sample preparation produce different soil structures for the same density. It is therefore necessary to consider the soil structure, with respect to the method of sample preparation, together with relative density when characterizing the physical behavior of a cohesionless soil.

4. The behavior of partially saturated soils does not appear to be well understood though they are encountered very frequently. When a partially saturated soil is analyzed for deformation or strength characteristics, the various components of the pore fluid pressure need be evaluated independently. The best approach would be to test undisturbed samples under field conditions.

CHAPTER 3

LABORATORY TESTING

Table 3.1 illustrates the laboratory testing program implemented to investigate the copper mine tailings for effects of drying, sample disturbance, and sample preparation on soil structure and hence on the deformation characteristics. The testing program consisted of two phases. Phase-I included the tests that were conducted to classify the tailings based on their physical properties such as particle size distribution, Atterberg limits, etc. It also included the standard and modified compaction tests which were to establish the moisture density relationship for the tailings. The moisture density relationship, as obtained from the impact compaction test, is shown in Figure 3.1. It was used to select the moisture contents (0%, 6%, and 14%) that corresponded to 95% of Maximum Dry Density, $\gamma_{d(max)}$. Later, the consolidation samples, both disturbed and undisturbed, were prepared and tested at these moisture contents. This was done to duplicate Chen's (1984) procedure for preparation of laboratory samples.

Phase-II consisted of sample preparation and oedometer testing. Undisturbed samples for consolidation tests were prepared from the "block" samples retrieved from the field. The distributed samples were prepared at the aforementioned moisture contents using impact and static compaction. Subsequently, all three types of

Table 3.1. Laboratory testing program.

Phase-I

- **Classification Test**
  - Sieve analysis
  - Hydrometer analysis
  - Atterberg limits
  - Specific gravity
  - x-rays diffraction Analysis
- **Sample Preperation Control Test**
  - Moisture density relation (modified proctor
  - Moisture density relation (standard (proctor)
  - Selection of moisture range from from modified proctor curve for consolidation tests

Phase-II

- **Sample Preperation**
  - Undisturbed specimens from "blocks"
  - Impact compacted to 0.95 $\gamma_{d(max)}$, and $\gamma_{d(field)}$
  - Static compacted to to 0.95 $\gamma_{d(max)}$, and $\gamma_{d(field)}$
- **Consolidation (Oedometer) and Swell Testing**
  - **Unsaturated Specimens**
    - Undisturbed triplicate specimens tested at: MC = 23%, field MC
      = 14%
      = 6%
      = 0%
      = 0%
    - Impact/static compacted triplicate specimens tested at MC = 14%
      = 6%
      = 0%
  - **Saturated Samples**
    - Testing conditions were the same as those of unsaturated specimens except that specimens were saturated and allowed to free swell before loading them in the consolidometer.

Note: *Testing carried out according to ASTM specifications and procedures.* Any modifications or departures are reported in the text of this report.

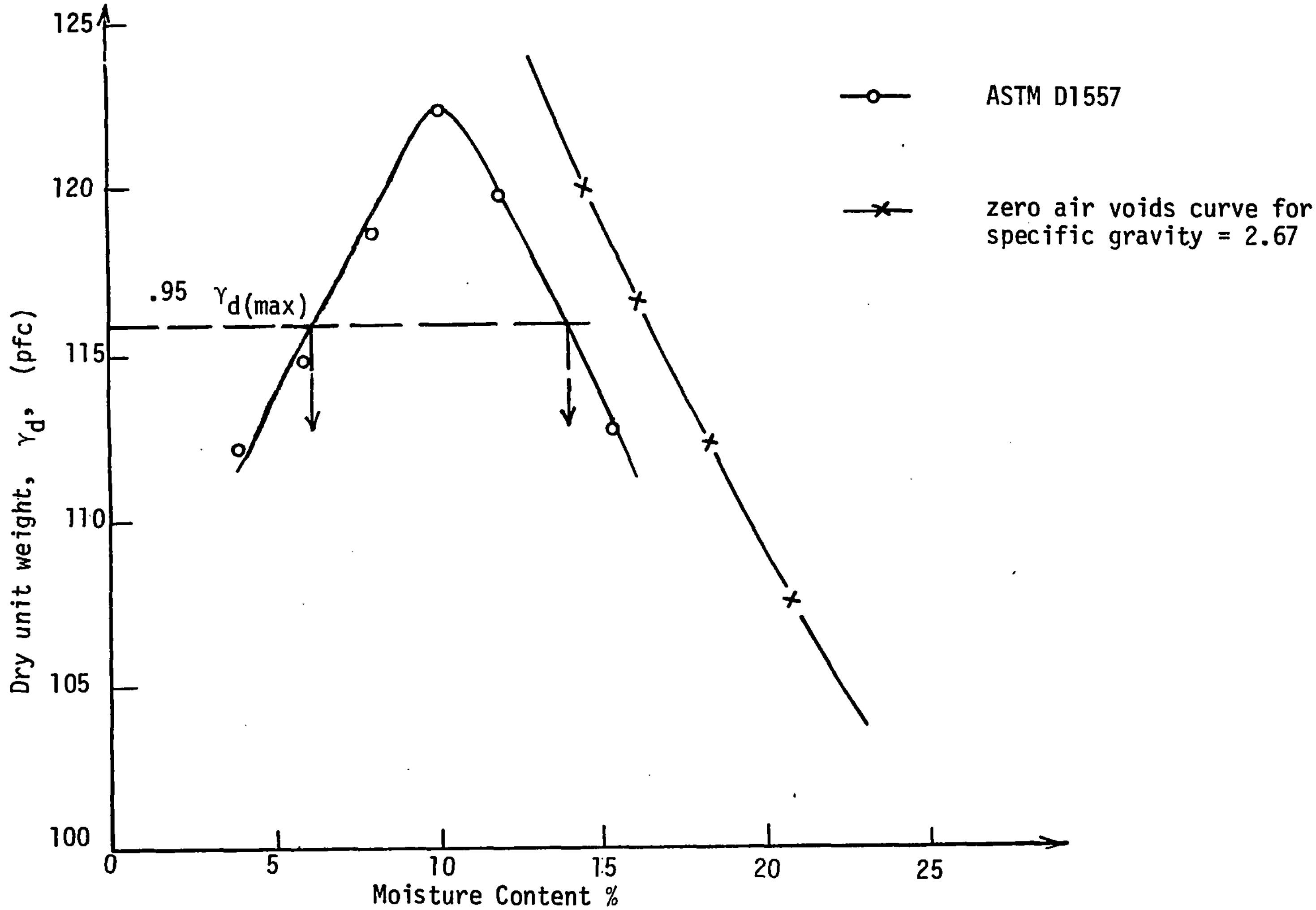

Figure 3.1. Compaction curve for Duval Mine tailings.

specimens were loaded in the consolidometer. They were tested both partially saturated as well as fully saturated. The fully saturated specimens were allowed to free swell prior to loading. The oedometer tests in this phase were conducted to evaluate the effects of sample disturbance due to sampling and handling on the soil structure in general and deformation properties in particular.

Further details of the tests conducted in each phase will be presented in the following sections of this chapter.

Samples and Sampling

The tailings investigated in this study were sampled from the tailings impoundment of Duval Corporation's copper mine situated about 35 miles southwest of Tucson, Arizona. Figure 3.2 is a location map for the impoundment. The samples were collected from the beach surface about 200 yards from the dam embankment and towards the pond. All the samples showed the presence of plastic "slimes".

The undisturbed samples were retrieved in the shape of large "blocks" approximately 1 ft. in diameter and 1.5 ft in height. They were secured in polythene bags to preserve the natural moisture conditions. Despite a relatively high moisture content, about 23%, the samples held together probably due to capillary action. These samples were handled with great care and caution was taken to ensure a minimum of disturbance. They were placed in specially constructed wooden boxes in order to protect them against any disturbance arising from handling, transportation etc. Disturbed samples were also

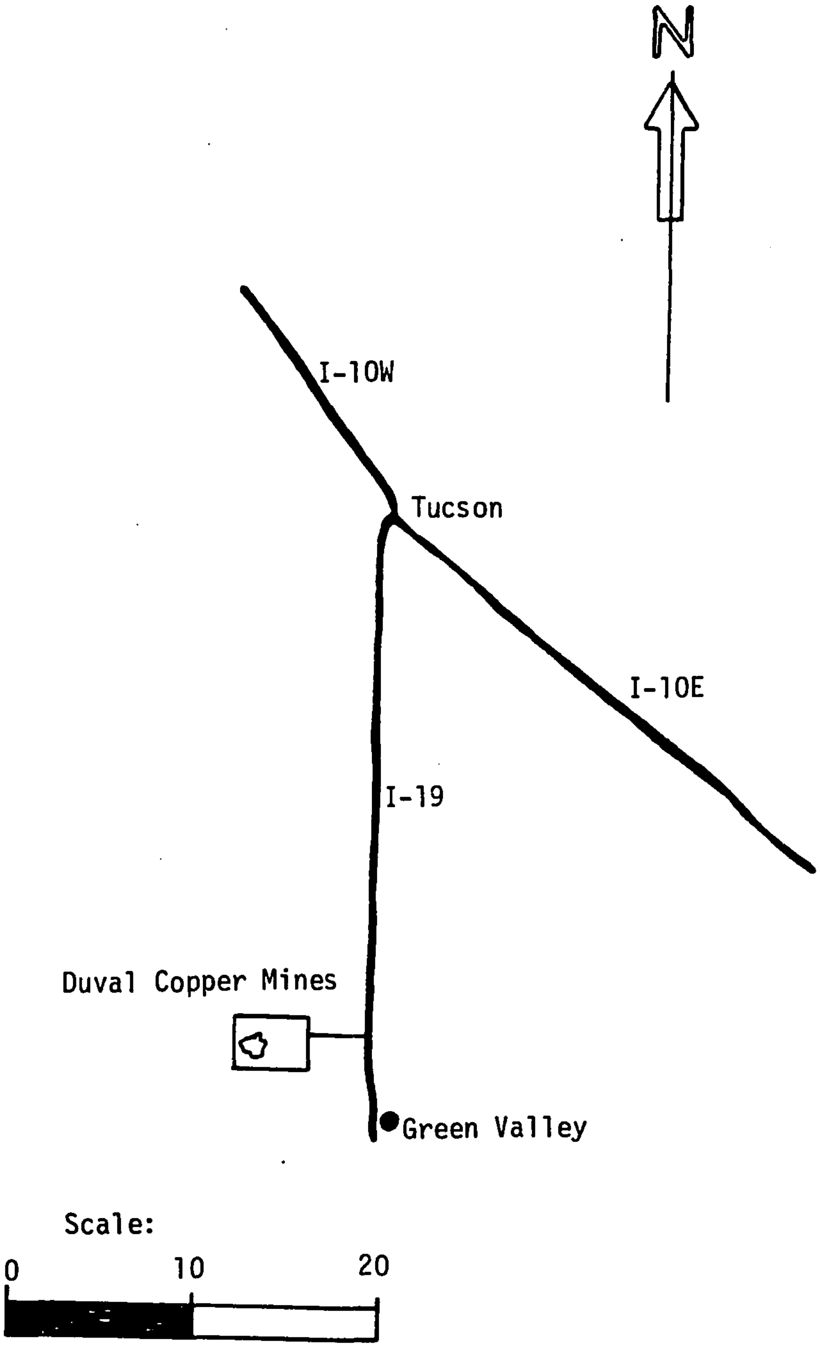

Figure 3.2. Location map for Duval Mine tailings.

collected in the shape of bulk samples. These were picked up from open pits excavated to shallow depths. They were stored in cloth bags.

To ensure uniformity, all the samples were acquired from the same preselected location in the tailings dam. During the sampling operation, it was observed that a fairly intact and undisturbed sample could be obtained at a moisture content of 23%. An attempt to secure a block sample with a higher moisture content met with failure. The reason may be the presence of montmorillonite which is known to reduce the shear strength of soils when their moisture content is well above the optimum moisture content thus making it virtually impossible to obtain an undisturbed block sample.

Following the field sampling, samples were transported to the Graduate Geotechnical Laboratory of the Department of Civil Engineering, University of Arizona, for further processing and analyses.

Description, Classification and Properties of Duval Mine Tailings

General

For physical characterization, classification tests (particle size distribution, Atterberg limits, and specific gravity) were performed on bulk samples of Duval Mine tailings. Besides, a series of x-ray diffraction analysis tests were also performed to determine the mineralogic composition of the tailings.

The tailings material can be described as an inorganic, well graded, slightly plastic silty sand with trace of clay and classified as an "ML" soil according to the Unified Soil Classification System. Figure 3.3 presents the grain size distribution curve. The effective particle size, D_{10}, is 0.0064 mm and the coefficient of uniformity, C_u, is 19. The specific gravity of the soil solids is 2.67. Chen (1984), who studied a similar copper mine tailings material, reports that the particles of both sand and silt have an angular shape. The in-situ dry density, γ_d, as obtained in the laboratory from the undisturbed block samples ranged from 84 pcf to 105 pcf.

Samples tested for Atterberg limits did not exhibit any plasticity, however they were observed to swell when brought in contact with water. To acertain the cause of swelling, a series of x-ray diffraction tests were performed. Table 3.2 illustrates the results of x-rays diffraction analyses. The clay mineral montmorillonite was detected.

In summary, the Duval Mine tailings can be described as a gray, well graded, slightly plastic, low swelling, medium dense, fine silty sand with a trace of clay.

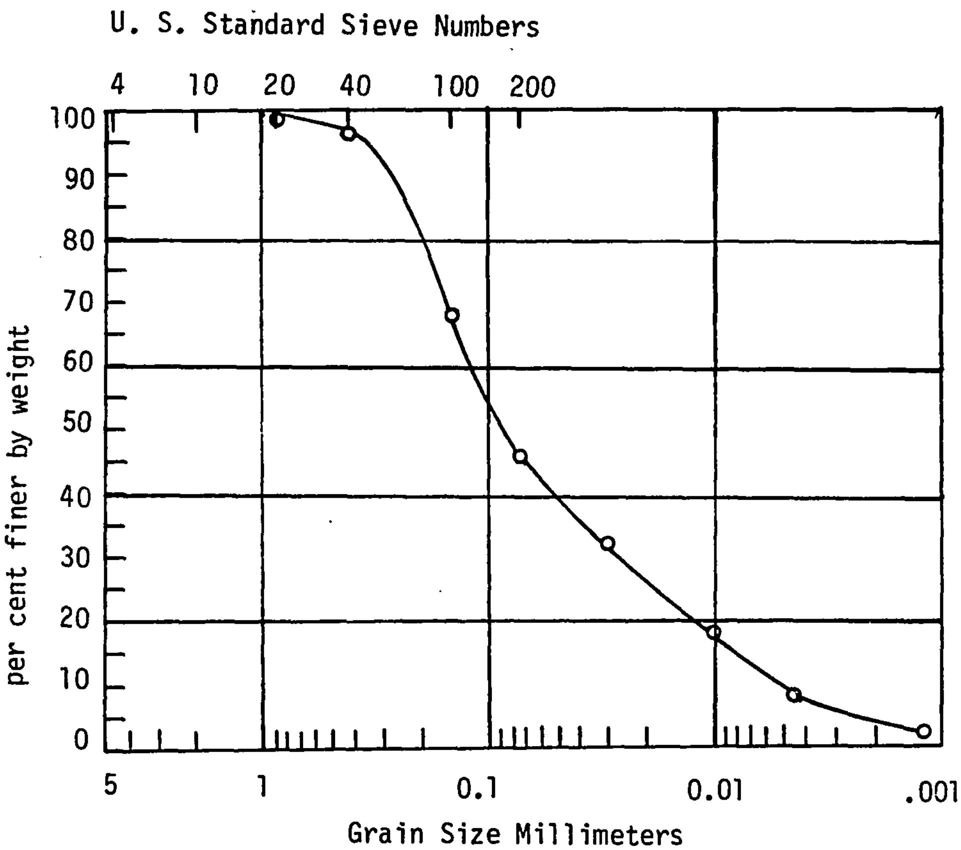

Figure 3.3. Grain size distribution curve for Duval Mine tailings.

Table 3.2. X-ray diffraction test results for Duval tailings (slurry specimen).

	Untreated slurry slide		
2 θ	d(Å)		Mineral
6.50	13.586		Montmorillonite
9.00	9.8168		Micas
12.50	7.0751		Chlorite
18.00	4.9238	out of limit	
21.00	4.2267	out of limit	
23.75	3.7509		Plagioclase
25.25	3.5309		Chlorite
26.75	3.3359		Orthoclase
2752	3.2406		Microcline
28.00	3.1839		Plagioclase
35.30	2.5404		Feldspars
3.660	2.4531		Quartz
50.50	1.8057		Feldspars
60.30	1.5336		Serpentine
61.80	1.4999		Muscovite & Illite
	Treated with Ethylene Glycol		
5.50	16.0540	out of range	
9.00	9.8168		Mica
12.50	7.0751		Chlorite

Compaction Tests

The relative density is not recommended for a material with more than 12% passing the No. 200 sieve. Impact compaction usually results in greater density than vibratory compaction for such materials. Impact compaction tests were therefore performed on the tailings. The maximum dry density obtained from the modified Proctor compaction test (ASTM D-1557) was 122 pcf and the optimum moisture content was 10.10%. The standard Proctor compaction test (ASTM D-698) produced a maximum dry density of 117 pcf and an optimum moisture content of 11.7%. The compaction curves are given in Figure 3.4.

One-Dimensional Consolidation Tests

A series of oedometer tests was performed on a number of disturbed and undisturbed samples with varying densities, and degrees of saturation ranging from 0% to 100%. The purpose of these tests was to evaluate the relative effects of drying and sample preparation dsturbance on the soil fabric and compressibility characteristics of a copper mine tailing.

As demonstrated by this study and others (Chen, 1984), the deformation vs log time curves did not exhibit a well defined break to indicate completion of primary consolidation for a given loading. Rather a linear curve marking a continuous deformation emerged. Loads left on for more than one day did not reach equilibrium. Rather a continuing volume change was observed. Lee et

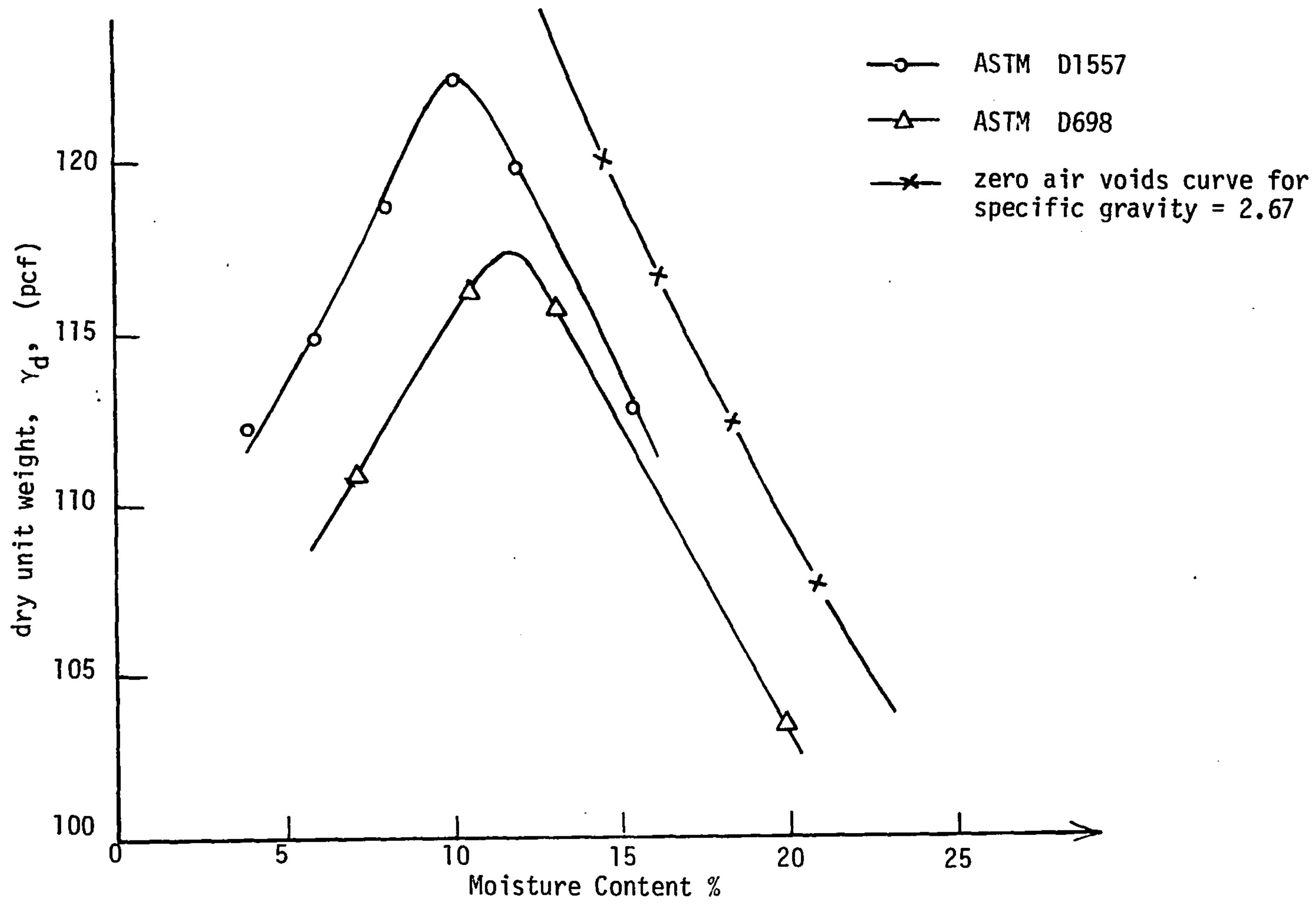

Figure 3.4. Moisture density relationship for Duval Mine tailings.

al. (1967) explain this behaviour by suggesting that continuing fracture propagation at grain-to-grain contacts promoted by the presence of water may be responsible for such linear continuation of secondary consolidation in an angular particulate soil medium such as tailings. Chen (1984) reports that the primary consolidation of a tailings material can not be detected with a conventional consolidometer since it occurs very rapidly. This point of view seems to be reasonable in the context of present experience and the experience of other researchers (Vick, 1983). The results of consolidation tests may be analyzed in terms of partially as well as fully saturated consolidation tests. Figures 3.5 presents e-log (p) curves for an undisturbed sample. They present a behavior that is typical of a partially saturated tailings soil.

Sample Preparation

Undisturbed Samples

Traditionally, undisturbed samples are obtained either by taking tubes from boreholes or by hand cutting blocks from test pits. Current state-of-the-art discourages tube samples from boreholes; depending on thickness to diameter ratio (t/d), the tubes are considered to disturb the soil. For a carefully hand cut block there need be no soil distortion as soil is cut away from around the block. A hand cut block may be regarded as the same as a tube sample taken with a tube with infinitely thin walls (i.e., t/d = 0).

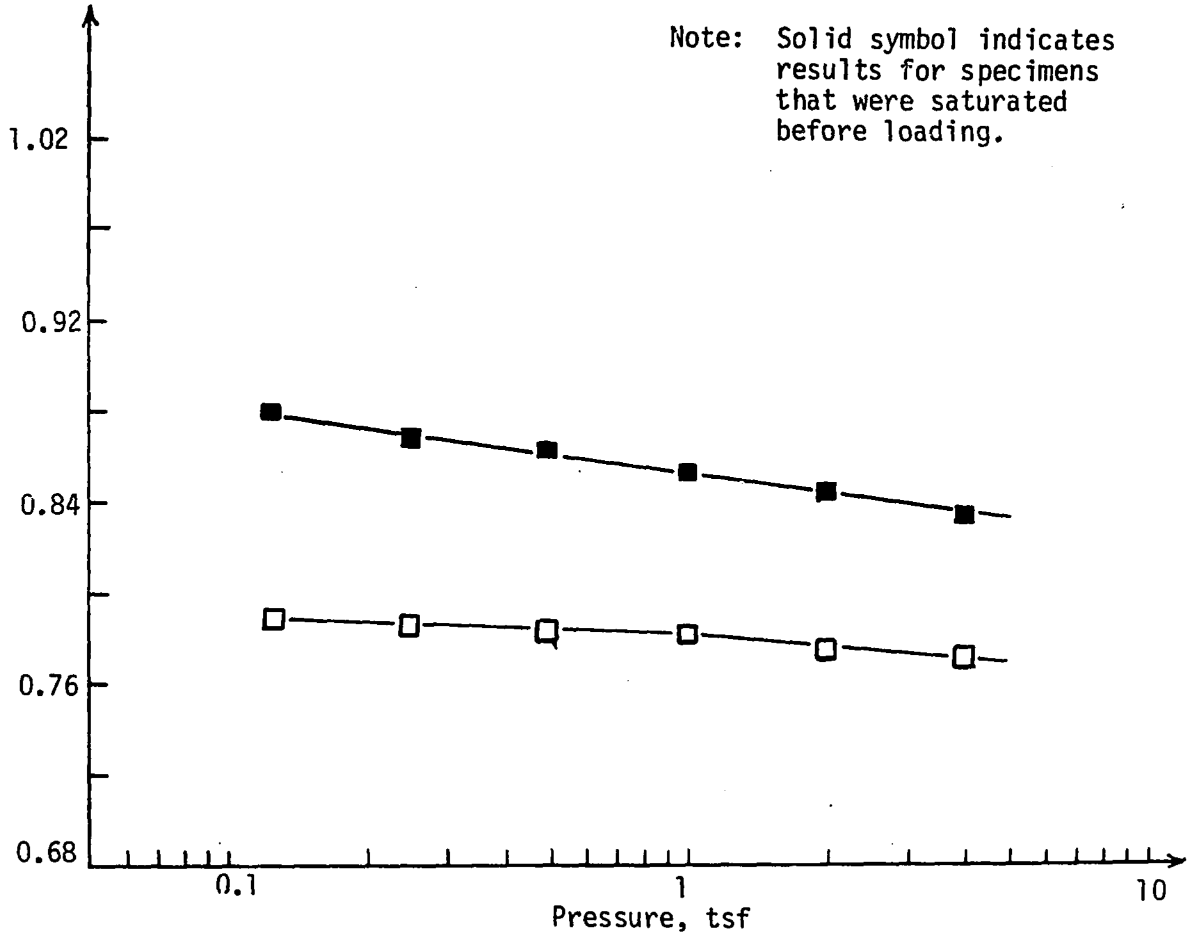

Figure 3.5. Stress-void ratio relationship for partially saturated, undisturbed tailings specimen at 0% initial moisture content.

Therefore, the undisturbed samples were retrieved from the tailings pond in the shape of relatively large blocks 1 foot in diameter and $1\frac{1}{2}$ foot in height. Consolidation specimens with varying moisture contents were carved from the block samples. To achieve the moisture contents, as selected from the moisture density curve, the blocks were stored in a humidity room where they were surveiled and checked for moisture contents on a daily basis. The moisture contents were determined by the gravity method; that is, small moisture samples were trimmed away from the blocks and dried in the oven.

Conventional tools such as a spatula and a wire-saw were used to carve and trim each specimen such that it fitted into a 2.5 inch diameter by 1.0 inch high consolidation ring. These specimens were carved from within the block and away from the edges so that as much mechanical disturbance could be avoided as possible.

Disturbed Specimens

The disturbed specimens were prepared in the laboratory using impact and static compaction. They were prepared at the moisture contents and dry density corresponding to 95% of the ASTM D-1557 maximum dry density (refer to Figure 3.1).

Impact Compacted Specimens. These specimens were prepared in the compaction mold following the ASTM D-1557 procedures. The compacted specimens were extruded from the mold with the aid of a mechanical extruder. Finally, consolidation specimens were cut by hand from the compacted blocks directly into the consolidation rings. Six consolidation specimens were prepared from each block. Three specimens were tested as molded whereas the other three were saturated before loading. The test data for the specimens so prepared appears to be in a fairly good agreement.

For the specimens to be tested dry, material was prepared at a moisture content close to that of field, compacted in the mold and dried in the oven. The consolidation specimens were then carved from the dried block. Figure 3.6 shows a consolidation specimen being prepared from the block.

For comparison with the undisturbed samples, a set of triplicate samples was also prepared at the field density of 88 pcf.

Statically Compacted Specimens. The statically compacted specimens were prepared directly in the consolidation ring. The amount of soil and water required to accomplish 95% of $\gamma_{d(max)}$ was determined based on the total unit weight (i.e., for each moisture content) and the known volume of the ring. The premixed soil and water were statically compacted in the ring in five layers. An aluminum pedestal was used to compact the material. For almost all of the samples, it was not possible to compact all the material into the ring. About 10 to 20 grams of soil was generally left out. This happened despite the fact that each layer was subjected to an axial stress of about 3,960 lb/ft^2. Probably, a much greater amount of energy was needed to compact all the material in the ring. However,

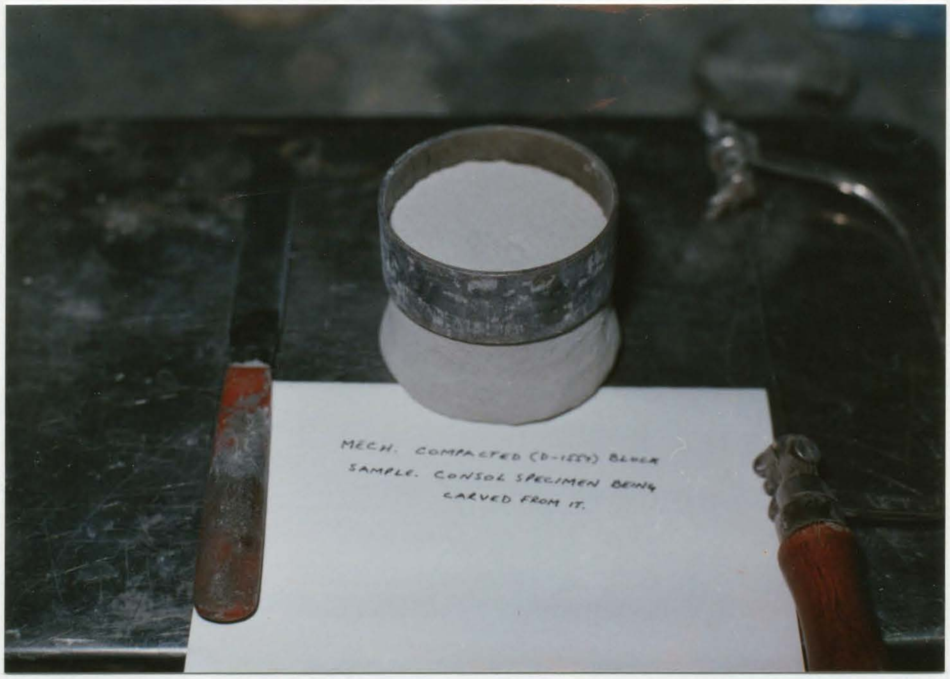

Figure 3.6. Consolidation specimen being carved from an impact compacted dry block sample.

it was suspected that such a higher energy might create an identical soil fabric for the specimens compacted wet and dry of optimum moisture content. Lambe and Whitman (1973) indicate that the structure of the soils, initially having different structures, becomes identical under large loads. The left out material was weighed and the dry density of the specimens was adjusted accordingly.

The consolidation specimens to have zero moisture content initially were prepared from an oven-dried, well-pulverized material. However, the specimens to be tested at the field density were statically compacted in the compaction mold at a moisture content as close to the field as practicable (i.e., MC = 20%). The purpose was to duplicate the field conditions. Later on, the mold with the compacted specimen intact was placed in the oven and allowed to dry to a point where the soil had gained some shear strength. The specimen was then extruded and placed back in the oven to dry completely. Finally, the consolidation specimen were cut from the dried block.

Consolidation Tests

Table 3.3 shows the details of testing performed in phase-II. Consolidation tests were run on undisturbed as well as disturbed specimen. The tests were performed on the specimen as prepared and fully saturated. The tests were performed according to the

Table 3.3. Phase II testing program.

	0%[a]	6%[a1]	14%[b]	20%[c]	23%[d]
Undistrubed Samples ($\gamma_{d(field)}$)	3[e](3)	3(3)	3(3)	-	3(3)
Impact Compacted					
95% $\gamma_{d(max)}$ ASTM D1557 = 115.9 pcf					
impact	3[f](3)	3(3)	3(3)	-	-
static	3[g](3)	3(3)	3(3)	-	-
$\gamma_{d(field)}$ = 88 pcf.	3[f](3)	-	-	-	-
static	3[f](3)	-	-	-	-

(a) Dry of optimum moisture content corresponding to 95% $\gamma_{d\ max}$ ASTM D1557.
(a$_1$) Dry of optimum moisture content corresponding to 95% $\gamma_{d(max)}$ ASTM D1557.
(b) Wet of optimum moisture content corresponding to 95% $\gamma_{d(field)}$ ASTM D1557.
(c) Wet of optimum moisture content corresponding to $\gamma_{d(field)}$ or ASTM D1557 Compaction curve
(d) Field moisture content
(e) Dried to 0% from natural moisture content
(f) Dried to 0% from compaction moisture content of 14%
(g) Compacted at 0% moisture content

NOTE: The numbers in parentheses represent saturated samples.

procedures and specifications as outlined in ASTM D-2435. The only departure was in case of the partially saturated specimens; they were tested at the moisture content they were prepared at. The intent in doing so was to simulate the field conditons as closely as possible.

The specimens were subjected to loads ranged from 1/8 tsf to 4 tsf. The applied stress was doubled at one hour intervals. The material deformation for each load increment was recorded at a preselected time sequence. The unloading was followed only at the beginning of the testing program. The rebound was observed to be negligible. When a similar pattern was noticed in several specimens, the unloading part of the test was viewed to be useless and hence discontinued.

The Soil Test Levermatic Apparatus, Model C-220, shown in Figure 3.7 was used for consolidation tests.

Swell Tests

As described earlier, the tailings material was observed to swell when brought in contact with water. The x-ray diffraction tests showed that the swelling was probably due to a small amount of montmorillonite in the tailings. This prompted a full fledged study into the swelling characteristics and the swell pressure of the tailings.

Free-swell type tests were performed on the samples to be tested fully saturated. Prior to initiating the consolidation tests, the specimens were placed in the consolidometer with the deformation

Figure 3.7. Soil test lavermatic consolidation test apparatus, model C-220.

swell gauge installed and allowed to free swell to equilibrium. The swell tests lasted for several hours. Following the completion of swelling, these specimens were loaded in the consolidometer. The amount of pressure required to compress the swelled specimens back to its initial height was taken to be the swell pressure. The dial reading at this point was considered to be the initial reading for the consolidation test. In certain cases, especially dry specimens the specimens were found to return to their initial height even before the dial gauge went back to the intial reading. It happened because some of the material had squeezed out under the applied load. The contention that the specimens had resumed the initial height and void ratio was merely an assumption. To determine an exact height would require knowledge of the pore water pressures which were not possible to determine in the laboratory.

The swell pressure, however, turned out to be relatively low. It ranged from 1/16 tsf for a specimen with initial moisture content wet of optimum to 1 tsf for the initially dry specimens. This confirms and agrees with the rebound test data suggesting that the tailings were a low swelling material. Figure 3.8 shows a swell test assembly.

Data Evaluation

Initially, the preconsolidation pressure (p_c) was selected as the parameter to study the effects of drying, sample disturbance, and a sample preparation disturbance on soil fabric and

Figure 3.8. Swell test assembly.

compressibility. Since this could not be achieved due to primary consolidation's taking place very rapidly, no further attempt was made to determine p_c. Instead it was chosen to make a comparison of the shapes and slopes of e-log(p) curves produced during the consolidation tests. Therefore, the consolidation test data would be plotted on the e-log(p) coordinates and, based on the shape and slope of the curves, the evaluation of sample distrubance would be made. The slopes of the curves for the disturbed specimens would be compared with those of the undisturbed specimens. If the slopes for the disturbed specimens are greater than those of the undisturbed, the disturbed specimens would tend to show higher compressibility and hence a pronounced variation in the soil structure. This would imply that the sample preparation technique has more adverse effects on the soil structure than the in-situ drying.

CHAPTER 4

ANALYSIS AND DISCUSSION

This chapter presents the results of the laboratory testing program conducted on copper mine tailings. The program consisted of two phases: Phase-1 testing was to identify the general physical characteristics of the tailings such as particle size distribution and etc. while Phase-2 comprised a series of one dimensional consolidation tests on a number of specimens representing different initial water contents and a variety of preparation modes. Tables 3.1 and 3.2 present a detailed outline of the laboratory testing program.

General

Based on the indicator tests, the mine tailings could be described as a well-graded, fine silty sand with a trace of clay. Further, it could be classified as a low swelling soil since it exhibited a maximum swell of over 6.62% when statically compacted to 95% of $\gamma_{d(max)}$ and allowed to free swell under a surcharge load of 1/16 tsf. The swelling took place because of the presence of montmorillonite, a highly expansive clay mineral.

Consolidation specimens were carved from undisturbed block spamples obtained from the field. These specimens were tested at water contents of 23% (field water content) 14%, 6%, and 0%. Disturbed specimens were prepared by impact and static compaction to

95% of $\gamma_{d(max)}$ and $\gamma_{d(field)}$ and tested at moisture contents of 20%, 14%, 6%, and 0%. The specimens compacted to $\gamma_{d(field)}$ and tested at 0% moisture content were molded at the field water content before they were dried in oven to obtain a condition of 0% moisture content. All of the specimens were prepared and tested partially as well as fully saturated in triplicate.

Based on the experience with the material investigated for this study, the following aspects are considered to be useful for the interpretation and analysis of the test results.

1. Initial Water Content
2. Clay Content
3. Soil Fabric
4. Density
5. Swelling and Saturation

The above effects are considered to influence the shape and slope of the e-log(p) curves. A comparison of the e-log(p) curves for all the soil specimens under the various test conditions shall be made to judge which of the conditions have the most pronounced effect on the material properties of the mine tailings.

First of all, test results of each specimen group (undisturbed, and disturbed) shall be described separately. This shall be followed by a discussion involving the comparison and interpretation of all of the test results.

Laboratory Test Results

Undisturbed Specimens

To study the effects of dessication on the preconsolidation pressure in general and the shape of the e-log(p) curves in particular, a set of tailings specimens was allowed to dry naturally in the laboratory. This was done to maintain constant environmental conditions. The specimens were dried to predetermined moisture contents and tested both partially and fully saturated in consolidometer. Triplicate specimens were tested as prepared, while another triplicate set was saturated before the load was applied. The saturated specimens were first allowed to free-swell under a surcharge pressure of 1/16 tsf. The consolidation test results are presented in Figure 4.1, and Table 4.1.

As illustrated by Figure 4.1, there was no marked break in the e-log(p) curves. Hence, a determination of the preconsolidation pressure was not possible. Vick (1973) and Chen (1984) report that the primary consolidation of a copper mine tailings takes place in a very short time and it is not possible to detect it with conventional consolidometers. For a comparative study, the slopes of the e-log(p) curves could be used in lieu of the preconsolidation pressure. The curves are almost linear for all the tested specimens. The arithmetic plot of the void ratio vs. consolidation pressure (Figure 4.2) shows the effect of linearity

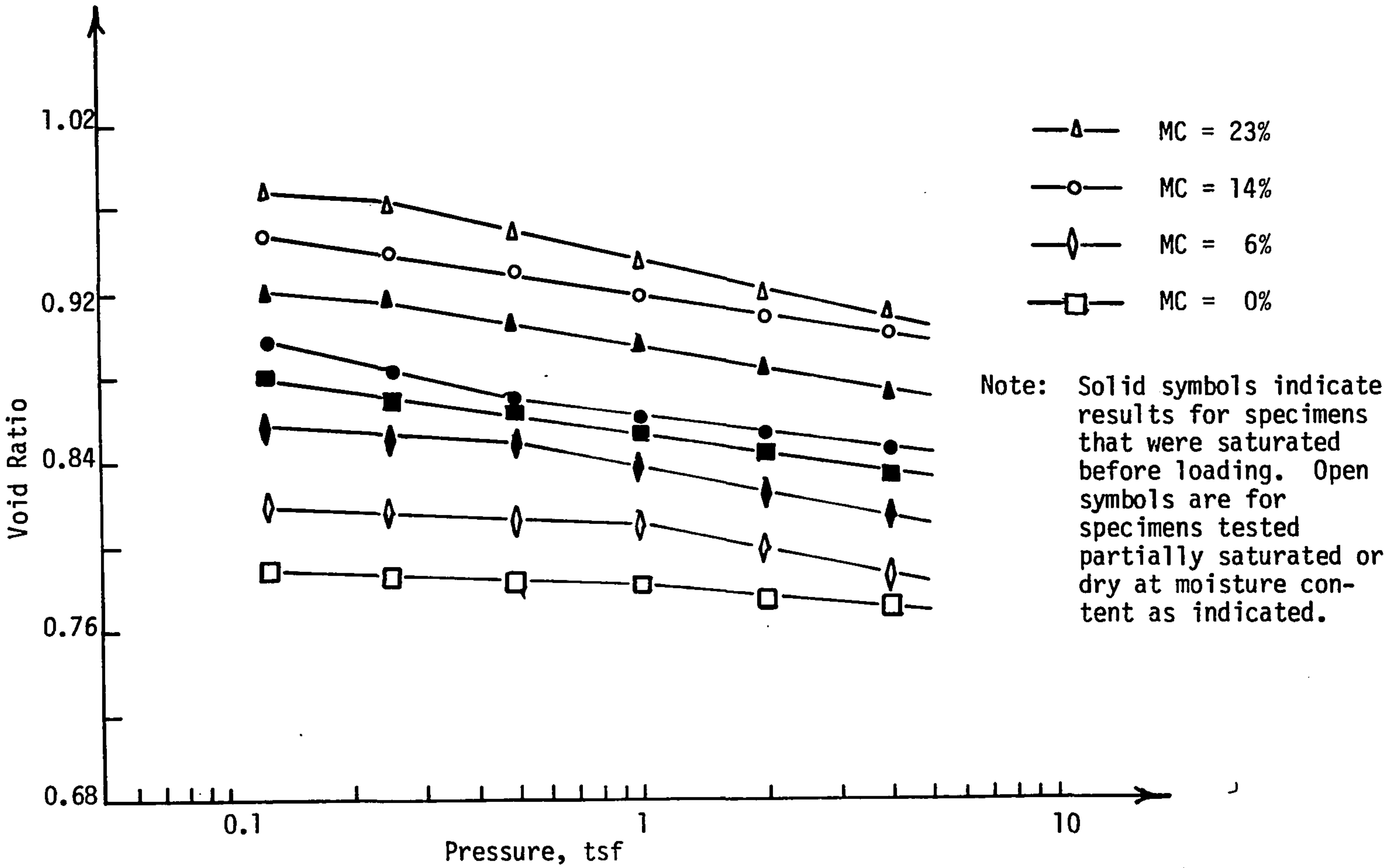

Figure 4.1. e-log(p) curves for undistributed specimens at 0%, 6%, 14%, 23% initial moisture contents.

Table 4.1. Summary of consolidation test results for undisturbed specimens at initial moisture contents 0%, 6%, 14%, and 23%

Specimen Condition	Void Ratio at pc = .125 tsf	Slope of Consolidation Curves	Consolidation Stress (p_c) Range
At 23%			
As prepared	0.966	-0.09	0.125 - 0.25
		-0.25	0.25 - 4.00
Saturated	0.920	-0.09	0.125 - 0.25
		-0.19	0.25 - 4.00
At 14%			
As prepared	0.945	-0.19	0.125 - 4.00
Saturated	0.895	-0.25	0.125 - 0.50
		-0.14	0.50 - 4.00
At 6%			
As prepared	0.821	-0.05	0.125 - 1.00
		-0.20	1.00 - 4.00
Saturated	0.855	-0.09	0.125 - 0.50
		-0.20	0.50 - 4.00
At 0%			
As prepared	0.789	-0.04	0.125 - 1.00
		-0.11	1.00 - 4.00
Saturated	0.879	-0.16	0.125 - 4.00

Pc = consolidation pressure (stress)

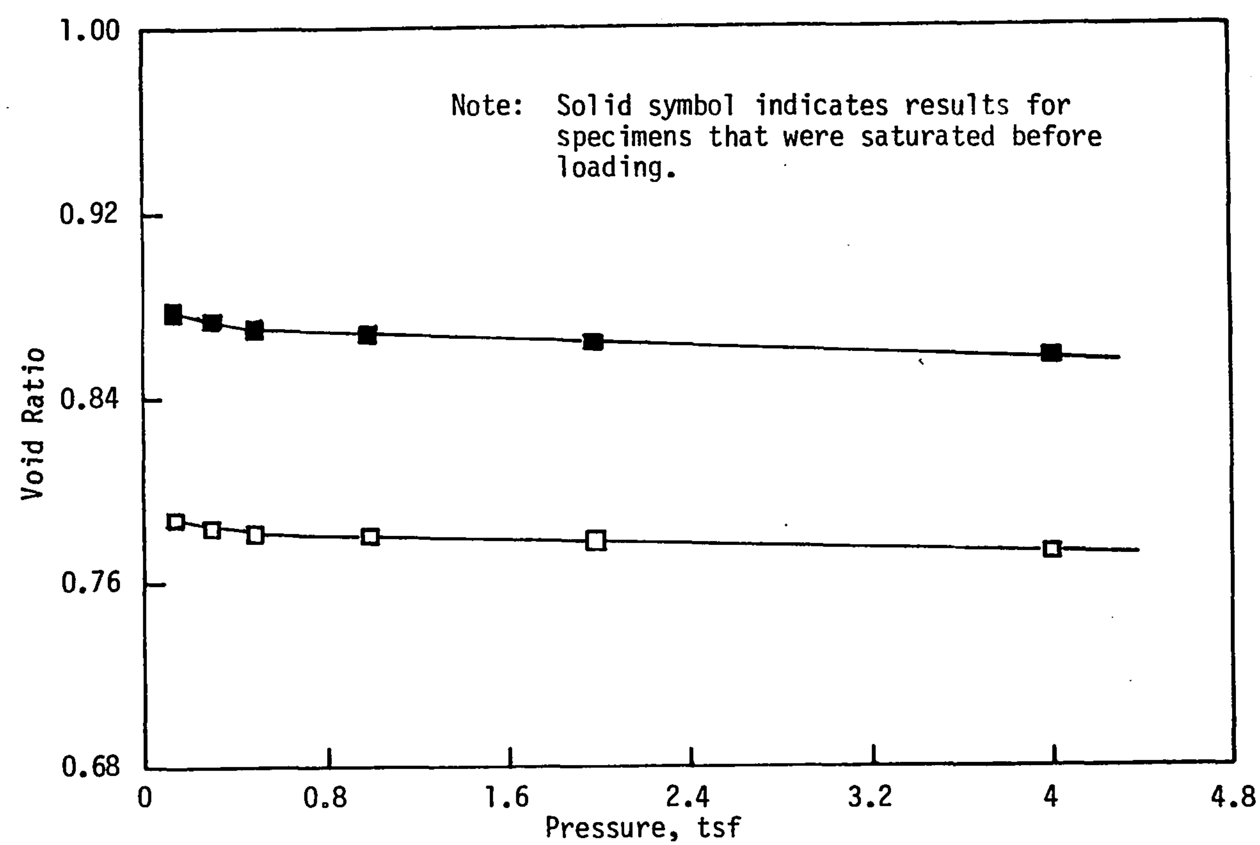

Figure 4.2. Mathematical plot of stress-void ratio relationship for undisturbed specimens at an initial moisture content of 0%.

more clearly. The curves become linear after about 1 tsf and continue in this fashion for higher consolidation pressures.

As is indicated by the e-log(p) curves, the effects of drying and swelling are quite pronounced. The specimens with initial moisture content of 0% have shown the minimum amount of compression under the entire loading range. The specimens with initial moisture content dry of optimum (6%, 0%) tend to be less compressible when tested as molded. They indicate a relatively higher compression when tested saturated. However, they are still less compressible than those tested at original moisture contents wet of optimum (14%). The specimens tested at the field moisture content of 23% tend to demonstrate maximum amount of compression under the full range of loading. This soil behavior could be attributed to the variation in soil fabric. The specimens at moisture contents wet of optimum have a deflocculated or dispersed structure which is more compressible even under light loads. The slope of the e-log(p) curve for a specimen at an initial moisture content of 23% is -.25 for a loading range of 1/4 to 1 tsf whereas the curve for a specimen at an initial moisture content of 0% is only -0.04 for the same range of loading.

The specimens saturated in a relatively short time, one hour in most cases. The swell percent ranges between 1.95% and 0.5% for specimens with initil moisture contents of 0% and 23% respectively. Implying that the specimens at higher initial moisture content swelled less than the one at the lower initial water content. Obviously, specimens at higher initial water content have less water deficiency than the ones at lower initial moisture content.

Disturbed Specimens

Impact Compacted. A series of specimens impact compacted to 95% of $\gamma_{d(max)}$ were tested at both as molded moisture contents as well as at saturation moisture contents. The plot of e-log(p) curves and a summary of consolidation test results are presented in Figure 4.3, and Table 4.2, respectively. Comapcted tailings appear to show a somewhat different behavior than the one reported in the preceding section. All of the e-log(p) curves are nearly flat with very minor slopes ranging between -0.3 and -0.08 for the entire range of loading except the one for the specimen at 14% initial moisture content when tested saturated. This curve exhibits a relatively steep slope of -0.31 over the loading range of 2 to 4 tsf. The specimens with initial water contents either dry of optimum or wet of optimum produced almost similar e-log(p) curves for both partial and full saturation conditions. The saturation does not appear to have any profound effect on the compressibility of the tailings under the given range of loading of 1/8 to 4 tsf. Nor does the initial water content.

Unlike the undisturbed specimens, the impact compacted specimens have reflected higher swell ranging between 1.02% to 5.82% for initial moisture contents of 0% and 14%, respectively. This

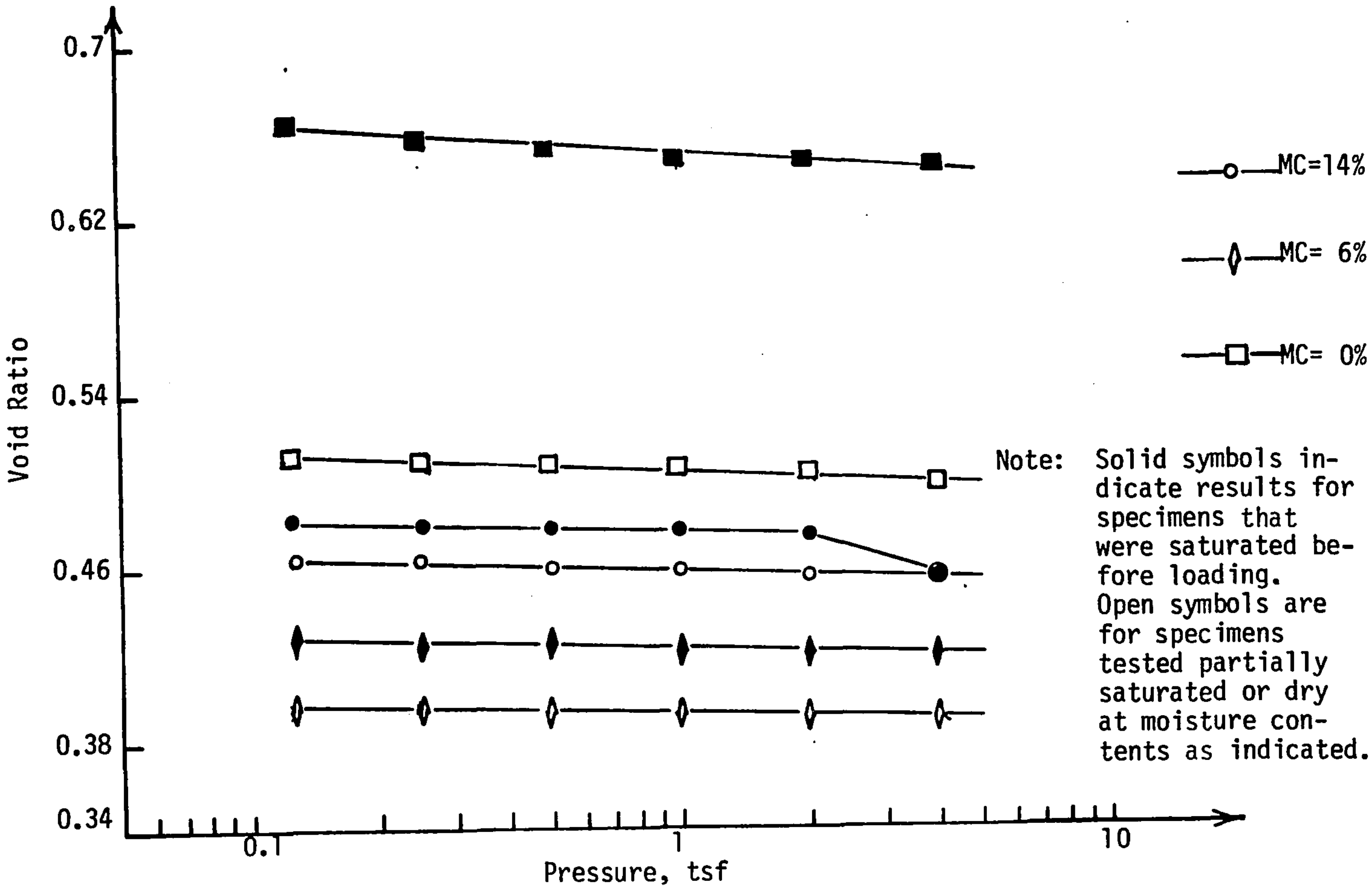

Figure 4.3. e-log(p) curves for disturbed specimens, impact/statically compacted to field density.

Table 4.2. Summary of consolidation test results for distributed specimens, impact compacted to 95% of $\gamma_{d(max)}$ at initial moisture contents 0%, 6%, and 14%.

Specimen Condition	Void Ratio at pc = .125 tsf	Slope of Consolidation Curves	Consolidation Stress (p_c) Range
<u>At 14%</u>			
As molded	0.463	-0.04	0.125 - 4.00
Saturated	0.481	-0.05	0.125 - 2.00
		-0.31	2.00 -4.00
<u>At 6%</u>			
As molded	0.395	-0.03	0.125 - 4.00
Saturated	0.428	-0.03	0.125 - 4.00
<u>At 0%</u>			
As molded	0.51	-0.06	0.125 - 4.00
Saturated	0.662	-0.08	0.125 - 4.00

could be due to the variation in soil structure. Dry of optimum, the particle orientation as induced by the impact compaction appears to be more random, thus causing more swell than the undisturbed specimens.

Statically compacted. A set of specimens statically compacted to 95% of $\gamma_{d(max)}$ at moisture contents of 14%, 6%, and 0% was subjected to one-dimensional consolidation. Test results are presented in Figure 4.4 and Table 4.3. As denoted by the e-log(p) curves, the soil behavior is somewhat similar to that of the undisturbed specimens. The specimens with initial moisture content wet of optimum (14%) appear to show more compression than those with initial water content dry of optimum. Generally, both partially saturated and fully saturated specimens have same slope for a loading range of 1/8 to 1 tsf. For a loading range of 1 to 4 tsf, the fully saturated specimens have a greater slope indicating a larger reduction in the void ratio than the ones tested partially saturated. Moreover, the curves for the saturated specimens plot below the curves for the specimens tested as molded. This behavior is opposite to that of the impact compacted specimens. It indicates a collapse of the soil structure.

The swell percent for these specimens is as high as 6.62% at initial water content of 0% and as low as 0.04% at 14% initial water content.

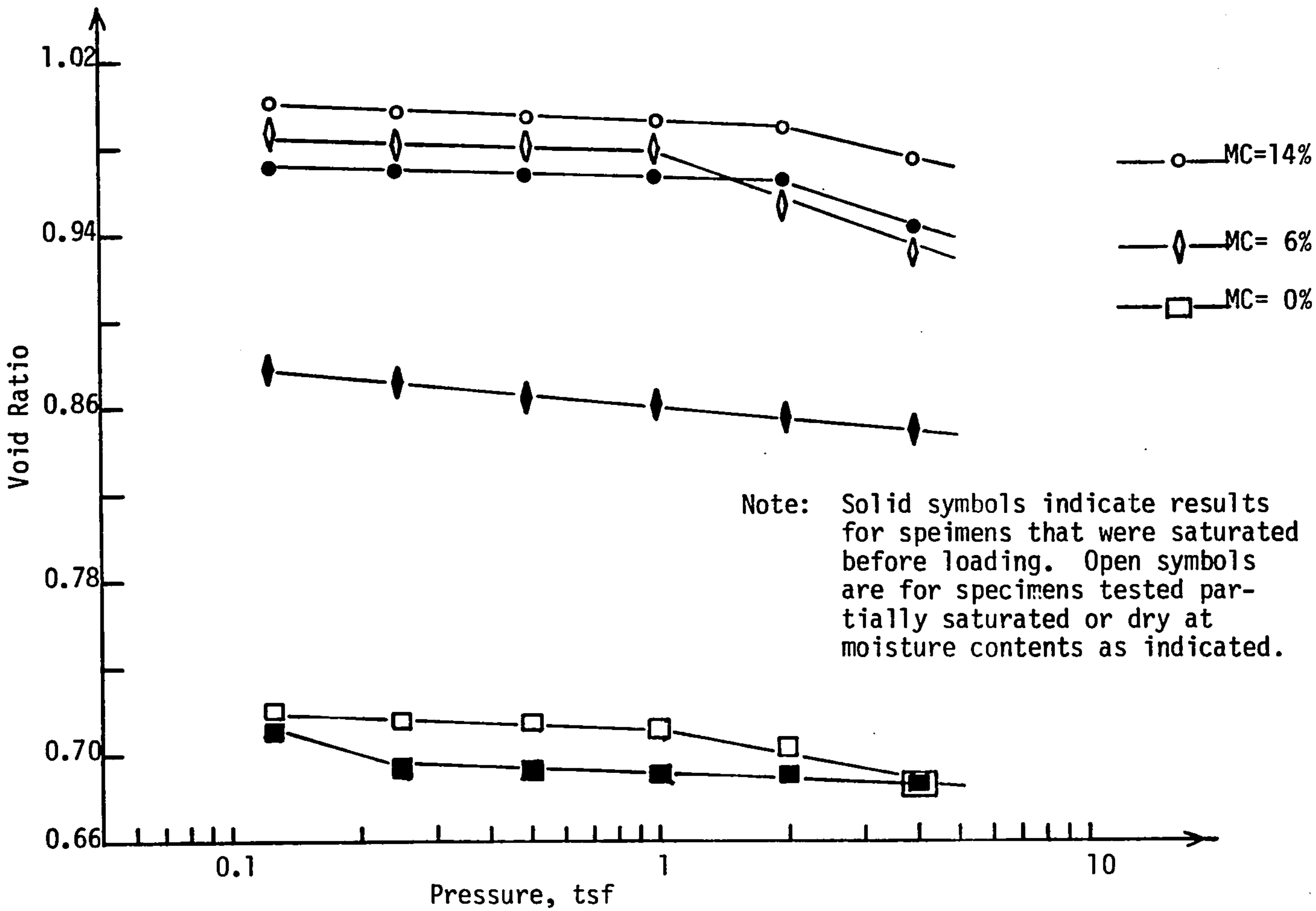

Figure 4.4. e-log(p) curves for disturbed specimens, statically compacted to 95% of $\gamma_{d(max)}$ at 0%, 6%, 14% initial moisture contents.

Table 4.3. Summary of consolidation test results for distributed specimens, statically compacted to 95% of $\gamma_{d(max)}$ at initial moisture contents 0%, 6%, 14% .

Specimen Condition	Void Ratio at pc = .125 tsf	Slope of Consolidation Curves	Consolidation Stress (p_c) Range
At 14%			
As molded	2.00	-0.05	0.125 - 2.00
		-0.23	2.00 - 4.00
Saturated	0.972	-0.31	0.125 - 2.00
		-0.31	2.00 - 4.00
At 6%			
As molded	0.879	-0.11	0.125 - 1.00
Saturated	0.984	-0.04	0.125 - 1.00
		-0.35	1.00 - 4.00
At 0%			
As molded	0.721	-0.03	0.125 - 1.00
		-0.19	1.00 - 4.00
Saturated	0.715	-0.31	0.125 - 0.25
		-0.05	0.25 - 4.00

<u>Compacted to Field Density</u>. To simulate the field conditions, a set of specimens with initial moisture content being 0% were prepared, using both impact and static compacton to the field density, $\gamma_{d(field)}$, of 88 pcf. A plot and summary of one-dimensional consolidation test results are presented in Figure 4.5 and Table 4.4. The impact compacted specimens show a smaller slope range, -0.02 to -.14, than the statically compacted specimens. The impact compacted specimens with initial moisture content of 0% produced an almost linear curve with a minor slope of -0.03 for the entire range of loading. Whereas the fully saturated specimens produced a slope of -0.02 for a loading range of 1/8 to 1/2 tsf and -.14 for 1/2 to 4 tsf.

The statically compacted specimens with initial moisture content of 0% have a minor slope of -0.01 for a loading range of 1/8 to 1tsf and -.18 for 1 to 4 tsf. The fully saturated statically compacted specimens yielded a behavior somewhat similar to that of the fully saturated impact compacted specimens in that they show a higher slope than the specimens tested as molded. The impact compacted specimens tested at full saturation have revealed a similar tendency. The slope of the curve for these specimens' curve is -0.09 for a loading range of 1/8 to 1/2 tsf and then a relatively steep slope of -0.29 for the loading range of 1/2 to 4 tsf.

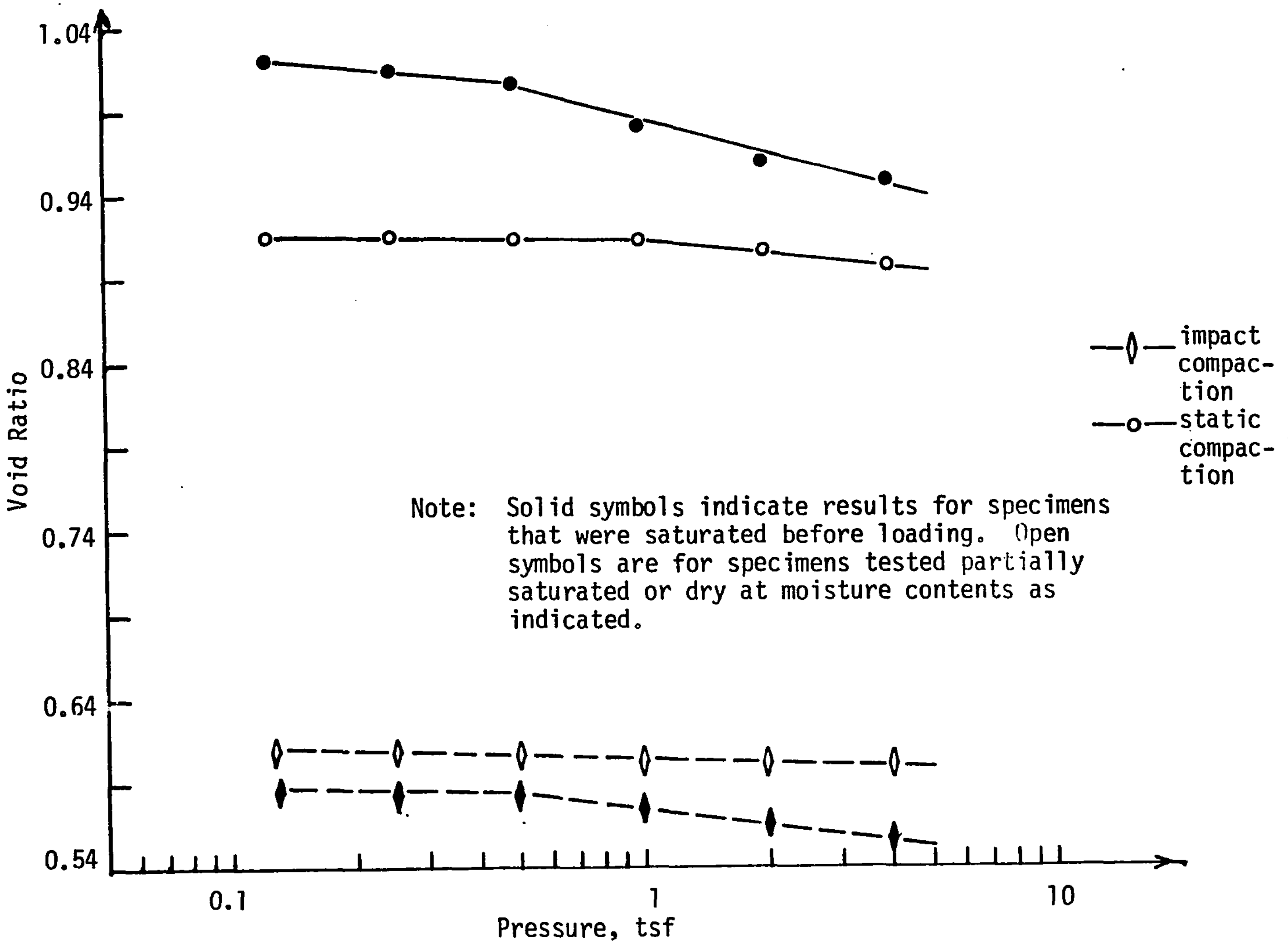

Figure 4.5. e-log(p) curves for disturbed specimens, impact compacted to 95% $\gamma_{d(max)}$ at 0%, 6%, 14% initial moisture contents.

Table 4.4. Summary of consolidation test results for disturbed specimens, impact and statically compacted to $\gamma_{d(field)}$ at initial moisture content 0%.

Specimen Condition	Void Ratio at pc = .125 tsf	Slope of Consolidation Curves	Consolidation Stress (p_c) Range
Impact compacted			
As molded	0.611	-0.03	0.125 - 4.00
Saturated	0.583	-0.02	0.125 - 1.00
		-0.14	1.00 - 3.00
Statically compacted			
As molded	0.918	-0.01	0.125 - 1.00
			1.00 - 4.00
Saturated	1.03	-0.9	0.125 - 0.50
		-0.29	0.50 - 4.00

A comparison of the e-log(p) curves of these two groups reveals that the saturated specimens have a greater tendency to compress than the ones tested at 0% initial moisture content.

Analysis of Test Results

Before touching on the analysis of the consolidation test results, it would be appropriate to offer a brief description of the factors, as mentioned earlier, to be considered in the interpretation of the test results.

1. Initial Water Content. The initial water content, whether natural or molding, has a profound effect on the material properties of any soil in general and a swelling or collapsing soil in particular. Depending upon the method of specimen preparation and other related reasons, the soil fabric of any soil greatly depends on the amount of the water the soil initially has. Fine-grained soils, for instance, when impact compacted at a moisture content dry of optimum tend to have a flocculated structure which would make them relatively less compressible under small loads. Fine-grained soils with low initial moisture content have greater water deficiency and hence tend to swell more when brought into contact with free water. On the other hand, fine-grained soils at moisture contents wet of optimum tend to have a dispersed structure, less water deficiency, and consequently are less likely to swell. However, they show a relatively greater compression under small loads.

2. Clay Content. Soil behavior due to moisture variation depends on the composition of the clay content of a specimen. Even a small amount of clay can govern the overall behavior of a soil. For instance, a soil containing only a small fraction of montmorillonite will behave as an expansive soil. The amount of montmorillonite in the tailings tested is estimated to be low. Yet it has caused enough swelling to classify the tailings as a low swelling soil.

3. Soil Fabric. Soil fabric has a definite effect on almost all of the engineering properties of a soil. The soil fabrics generally fall into two major classes namely flocculated soil fabric and deflocculated soil fabric. Figure 4.6 shows the types of soil fabrics (after Mitchel 1976). For compacted soils, the type of fabric depends on two chief factors: 1) the method of specimen preparation, and 2) the initial molding water content. Specimens compacted dry of optimum are known to have flocculated structure while the ones compacted wet of optimum have a deflocculated or dispersed structure.

In the past, relative density was the only factor considered in the characterization of the behavior of a noncohesive granular soil. Recent research shows that compacted specimens prepared at the same dry density but with different methods of preparation show completely different stress-strain behavior. This phenomenon was noticed to influence the results of this study too.

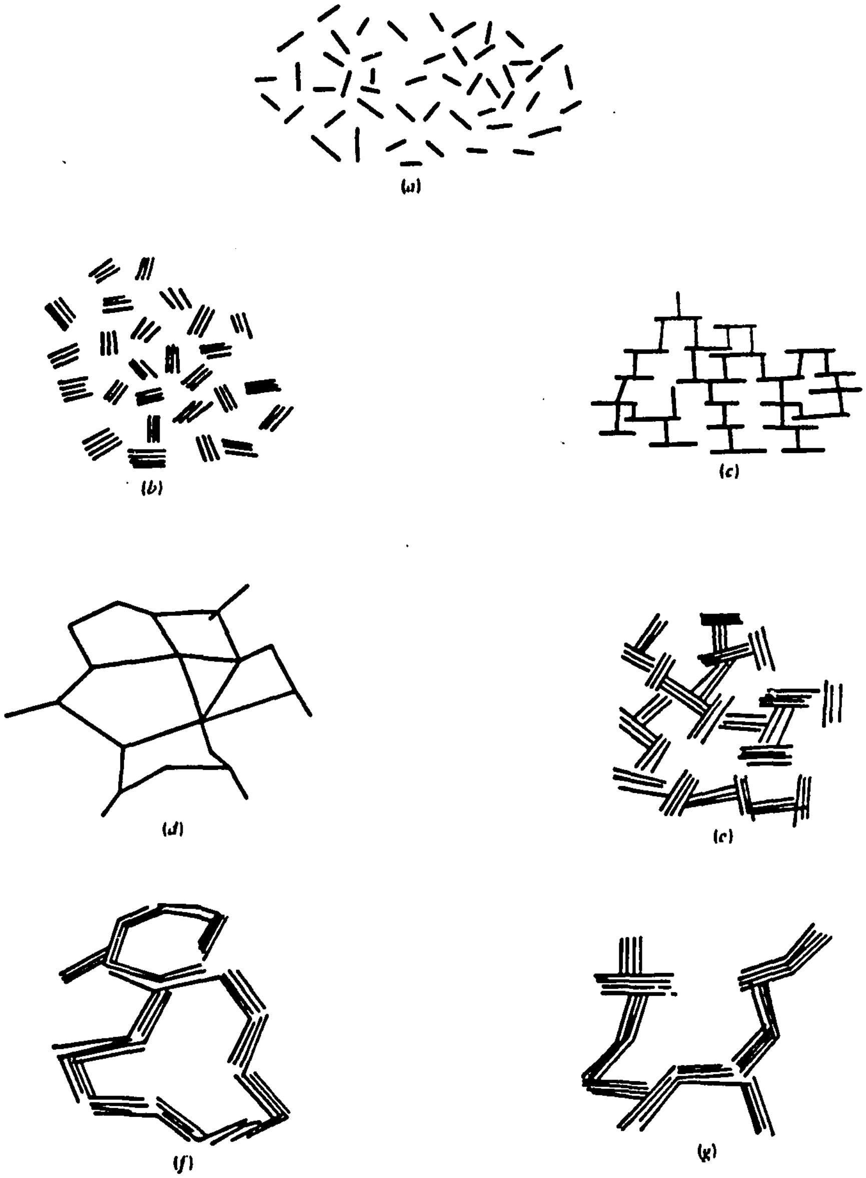

Figure 4.6. Modes of partical association in clay suspensions. (After Van Olphen, 1963. From Mitchell, 1976.)

(a) Dispersed and deflocculated
(b) Aggregated but deflocculated
(c) Edge-to-face flocculated but dispersed
(d) Edge-to-edge flocculated but dispersed
(e) Edge-to-face and edge-to-edge flocculated and aggregated
(f) Edge-to-edge flocculated and aggregated
(g) Edge-to-face and edge-to-edge flocculated and aggregated

5. Density. The initial density of a soil deposit has a significant effect on its stress-deformation characteristics. Soil specimens in their loose state have a higher initial void ratio and consequently experience a more drastic vertical compression than those in a dense state. Loose soils also experience a change in the fabric under a very small loading. Initially, loose soils experience less volume change due to swelling than dense soils; this behavior was observed during this investigation. Soil strength parameters C and ν are greatly influenced by the density of the soil.

6. Swelling and Saturation. Saturation of certain types of fine-graind soils results in volume change. These volume changes can be either positive (dilation or swell) or negative (compression or collapse). The swelling of a soil affects the pore geometry as a consequence of which the soil fabric changes. Swelling icnreases soil volume. Therefore, a substantial swell can result in the failure of a structure founded on a swelling soil. A swell percent of as low as 5% can cause some damage to light structures.

As indicated previously, saturation also tends to cause collapse in certain types of soils. Even non-collapsible, non-cohesive soils compress to one degree or another when saturated. For instance, in this study specimens statically compacted to field denisty exhibited less compression when tested fully saturated than the dry specimens, refer to Figure 4.7 This happened because the structure of these specimens might have already appreciably collapsed due to saturation and hence resulted in relatively less compression upon loading.

Comparison of Test Results

For convenience, the test results for all of the specimens at one moisture content, say for instance 0%, are plotted together on the same e-log(p) coordinate. The test results are presented in the relevant figures. These figures present the results of consolidation tests performed on undisturbed specimens as well as the results for impact and statically compacted specimens. The compacted specimens were prepared at 0%, 6% and 14% moisture contents to densities of 0.95 $\gamma_{d(max)}$ and $\gamma_{d(field)}$. The results for the specimen in each moisture group will be discussed separately.

Specimens at 0% Initial Moisture Content. Figure 4.7 and Table 4.5 present results of specimens with initial moisture content of 0%. The effects of drying, saturation, and soil fabric are obvious. Specimens prepared by impact compaction, both at 95% of the mximum dry density and the field dry density, exhibted the least amount of compression of all the specimens. Dry specimens have almost the same slope but have different initial void ratios which is due to a difference in the dry densities. The saturated specimens displayed relatively more compression for the given range of loading. Specimens compacted to the field density tend to be more compressible for the loading range of 1/2 to 4 tsf with a slope of -.14.

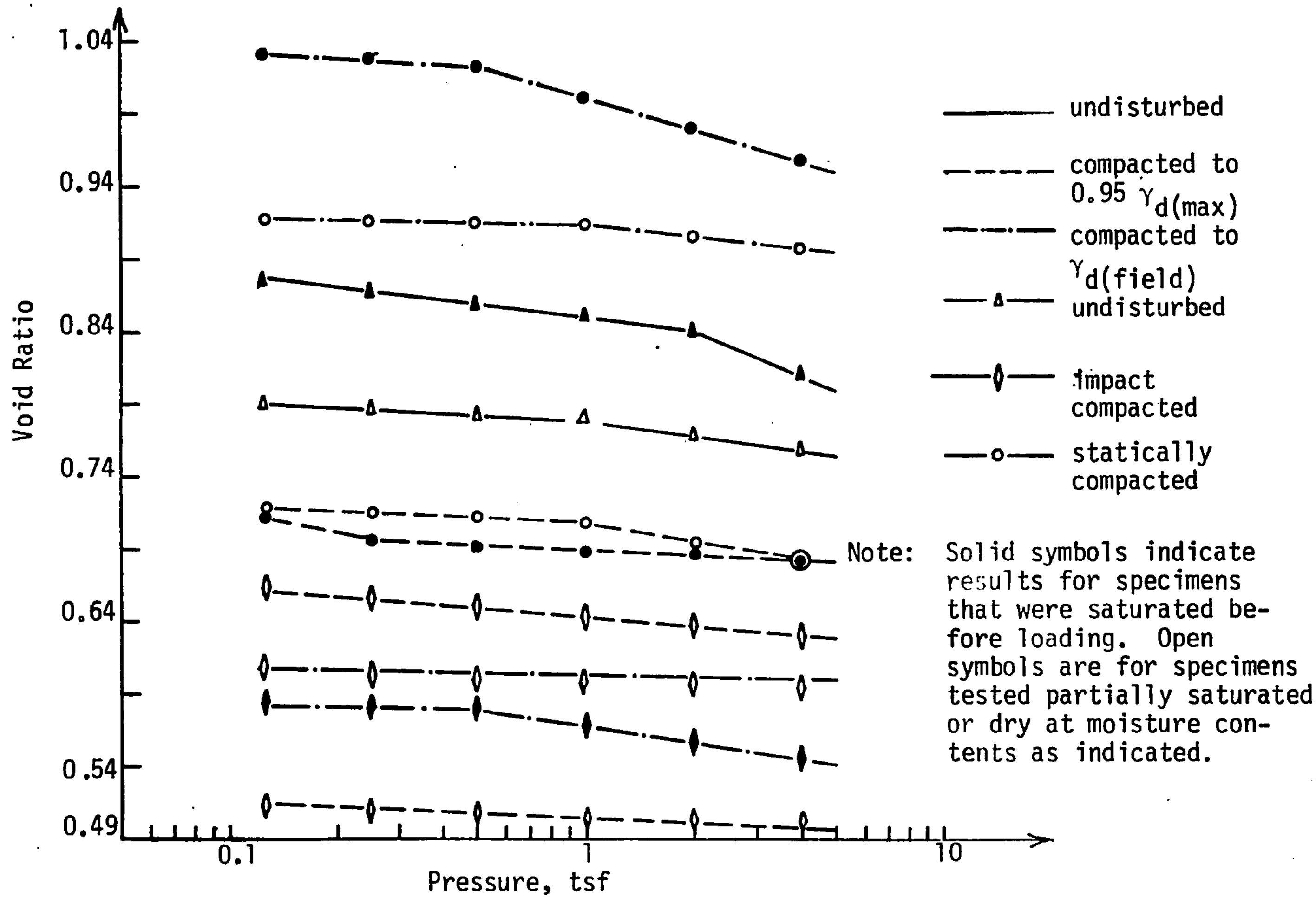

Figure 4.7. e-log(p) curves for undisturbed and disturbed, impact/statically compacted, specimens at 0% initial moisture content.

Table 4.5. Comparison of consolidation test results for undisturbed and disturbed, impact, statically compacted to 95% of $\gamma_{d(max)}$ and $\gamma_{d(field)}$ specimens at initial moisture content 0%.

Specimen Condition	Void Ratio at pc = .125 tsf	Slope of Consolidation Curves	Consolidation Stress (p_c) Range
Undisturbed			
Dry	0.789	-0.04	0.125 - 1.0
		-0.11	1.0 - 4.0
Saturated	0.879	-0.16	0.125 - 2.0
Impact compacted	$\gamma_{d(field)}$		
Dry	0.609	-0.03	0.125 - 4.0
Saturated	0.582	-0.02	0.125 - 0.5
		-0.14	0.50 - 4.0
0.95 $\gamma_{d(max)}$			
Dry	0.510	-0.06	0.125 -4.0
Saturated	0.663	-0.08	0.125 - 4.0
Statically compacted	$\gamma_{d(field)}$		
Dry	0.919	-0.01	0.125 - 1.00
		-0.18	1.00 - 4.00
Saturated	1.03	-0.09	0.125 - 0.50
		-0.29	0.50 - 4.00
0.95 $\gamma_{d(max)}$			
Dry	0.719	-0.03	0.125 - 1.00
		-0.19	1.00 - 4.00
Saturated	0.71	-0.31	0.125 - 0.25
		-0.03	0.25 - 4.00

The results of statically compacted specimens reveal that the saturated specimens experienced more compression than the non-saturated. Furthermore, the saturated specimens prepared to field density exhibited more compression for a loading range of 1/2 to 4 tsf than the ones prepared to 95% of $\gamma d(max)$. The results for non-saturated specimens indicate an almost identical soil behavior over the entire loading range.

The results for the undisturbed specimens indicate a behavior not very different from that of the statically compacted specimens. The saturated specimens showed a continuous compression under the full range of loading.

Specimens at 6% Initial Moisture Content. Figure 4.8 and Table 4.6 show the results of specimens having an initial moisture content of 6%. A pattrern similar to the case where the initial moisture content was 0% is visible here. Impact compacted specimens, both partially saturated and fully saturated, show the least amount of compression with the slopes of their e-log(p) curves being identical, -0.03.

The undisturbed and statically compacted specimens show a somewhat similar behavior. The unsaturated specimens have the same slopes for a loading range of 1/8 to 1 tsf beyond which the statically compacted specimens compresses more with its slope being -.35 as compared with a slope of only -.11 for the undisturbed specimen. The undisturbed saturated specimen shows more

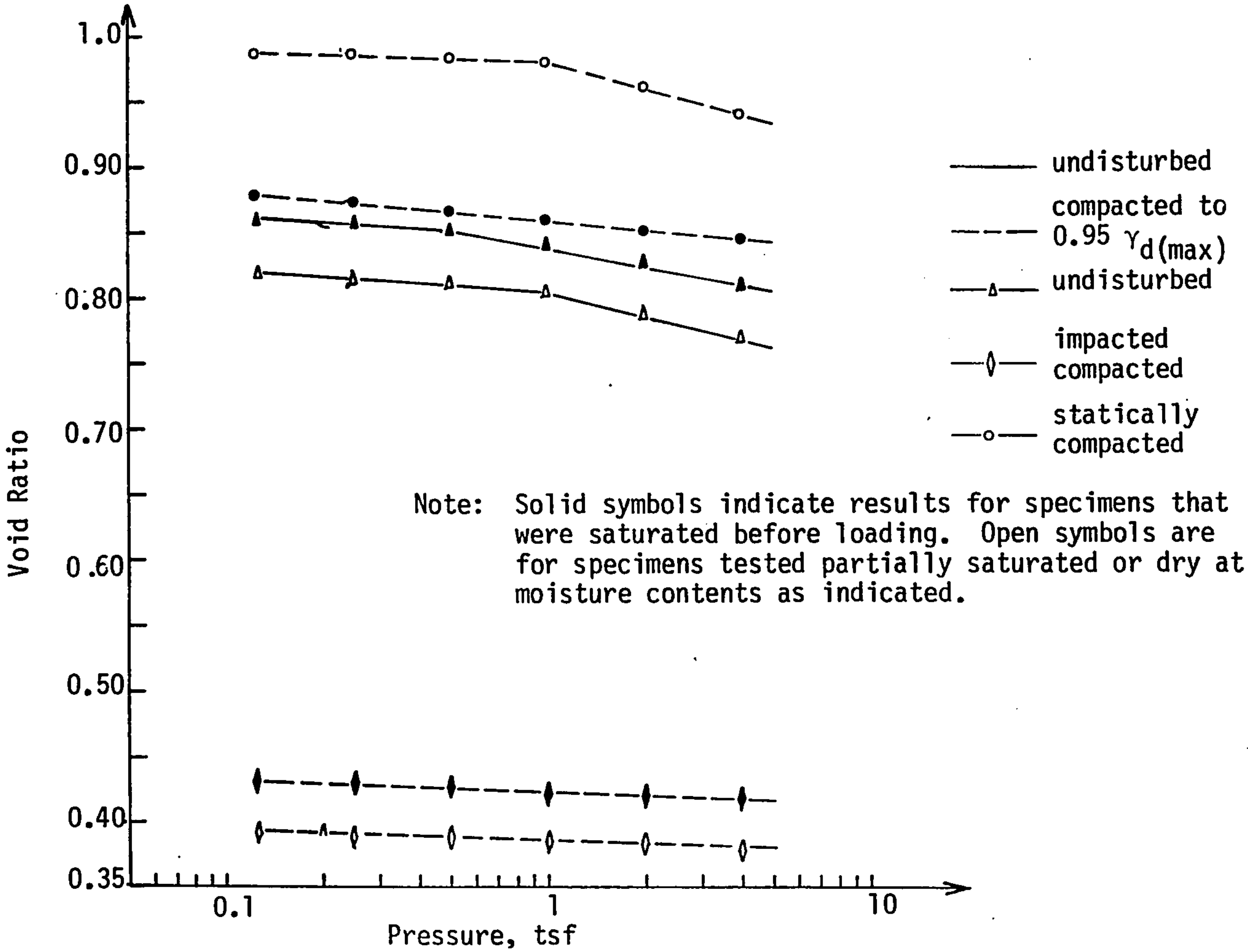

Figure 4.8. e-log(p) curves for undisturbed and disturbed, impact/statically compacted, specimens at 6% initial moisture content.

Table 4.6. Comparison of consolidation test results for undisturbed and disturbed, impact, statically compacted to 95% of $\gamma_{d(max)}$ at initial moisture content 6%.

Specimen Condition	Void Ratio at pc = .125 tsf	Slope of Consolidation Curves	Consolidation Stress (p_c) Range
Undisturbed			
Dry	0.821	-0.05	0.125 - 1.00
		-0.11	1.00 - 4.00
Saturated	0.858	-0.09	0.125 - 0.5
		-0.20	0.50 - 4.0
Impact compacted			
Dry	0.396	-0.03	0.125 - 4.00
Saturated	0.429	-0.03	0.125 - 4.00
Statically compacted			
Dry	0.985	-0.04	0.125 - 1.00
		-0.35	1.00 - 4.00
Saturated	0.878	-0.11	0.125 - 4.00

compression, slope -0.12, for the loading range of 1/2 to 4 tsf than the statically compacted specimen whose slope for the entire range of loading is -.11.

Specimens at 14% Initial Moisture Content. Figure 4.9 and Table 4.7 show the results of tests for specimens with an initial moisture content of 14%. The impact compacted saturated sepcimen has a minor slope of -.05 over the entire range of loading while the statically compacted specimen has the same slope for the loading range of 1/8 to 2 tsf beyond which the slope becomes relatively steeper, -.23. The undisturbed specimens, tested both as molded and fully saturated, shows a compression larger than both statically and impact compacted specimens.

Discussion of Test Results

The apparent variation in the inital void ratios of the specimens tested during this study is due to a slight inconsistency in the dimensions of the specimens. It was not possible to adhere to precise dimensions for all the specimens due to some difficulty faced in carving the consolidation specimens from the large blocks. Especially in case of specimens to be tested dry, the specimen preparation became relatively more difficult on account of the rigidness and brittleness of the tailings material. Under those conditions, the tailings took on a consistency of a sand-cement-mixture. Had the inital void ratios been identical, the e-log(p)

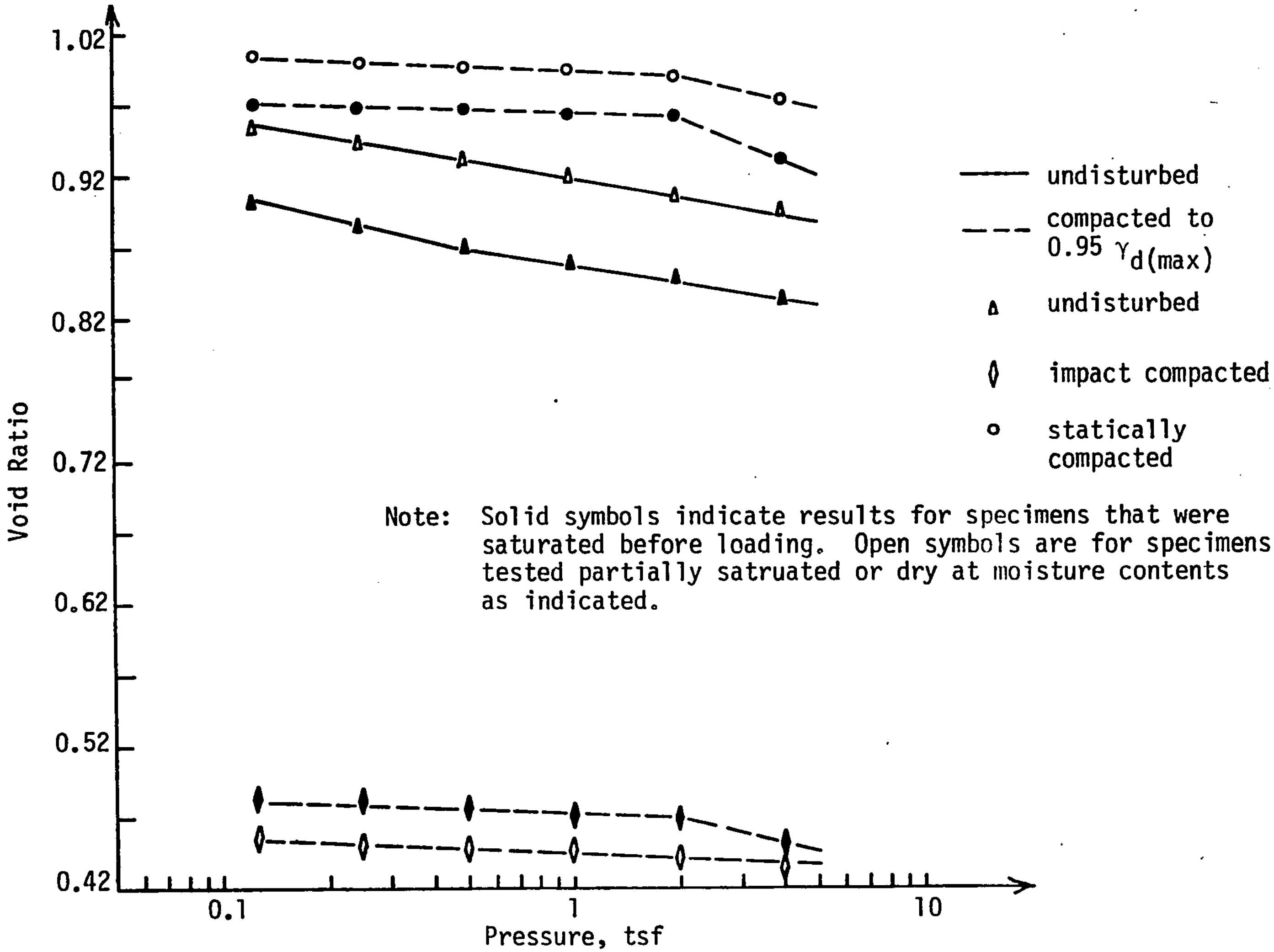

Figure 4.9. e-log(p) curves for undisturbed and disturbed, impact/statically compacted, specimens at 14% initial moisture content.

Table 4.7. Comparison of consolidation test results for undisturbed and disturbed, specimens impact, statically compacted to 95% of $\gamma_{d(max)}$ at 14% initial moisture content.

Specimen Condition	Void Ratio at pc = .125 tsf	Slope of Consolidation Curves	Consolidation Stress (p_c) Range
Undisturbed			
Dry	0.955	-0.19	0.125 - 4.00
Saturated	0.902	-0.25	0.125 - 0.50
		-0.14	0.50 - 4.00
Impact compacted			
Dry	0.463	-0.04	0.125 - 4.00
Saturated	0.481	-0.05	0.125 - 2.00
		-0.31	2.00 - 4.00
Statically compacted			
Dry	1.00	-0.05	0.125 - 2.00
		-0.23	2.00 - 4.00
Saturated	0.973	-0.02	0.125 - 2.00
		-0.31	2.00 - 4.00

curves for a specific initial moisture condition would probably have plotted more closely than they appear on the figures presented herein. But, this does not seem to have affected the material behavior to any great extent. It is not the initial void ratios that need to be compared. The analysis, however, is to be based on a comparison of the slopes of the specimens representing various test conditions. Therefore, the scatter of the curves does not matter here.

The test results suggest that soil fabric exerts the major influence on the material behavior. The specimens prepared to a same density but with different methods of preparation showed a different material behavior. Refer to Figure 4.5 and Table 4.7. The second major factor to influence the test results was the initial moisture content. The initial density and swelling of the material do not appear to have affected the test results; the effect of the initial density is apparently overshadowed by the soil fabric. Generally, the undisturbed specimens underwent more compression under a given range of loading than the impact or statically compacted specimens. The impact compacted specimens underwent the least amount of compression at all the initial water contents (0%, 6%, 14%). The statically compacted specimens displayed compression trend similar to those of the undisturbed specimens. This trend is more obvious in case of the specimens tested at full saturation. Another general tendency exhibited by all the specimens is that the specimens prepared at an initial moisture content dry of optimum are less compressible than those prepared wet of optimum. The same was found to be true with respect to swelling. The specimens prepared dry of optimum showed more swelling than the ones prepared wet of optimum. Also, the specimens tested fully saturated displayed more compression than the unsaturated ones. These trends could be easily seen in Figures 4.5, 4.8, and 4.9 and Tables 4.5 and 4.6.

A possible explanation for the above behavior could be furnished with the help of the concept of soil fabric. It appears that the undisturbed specimens have a soil structure which is more dispersed than random thus making them more compressible. The impact and statically compacted specimens prepared to a moisture content dry of optimum have a deflocculated structure which is more resistent to deformation over a loading range such as the one used during this study. The impact compacted specimens showed the least amount of compression even when fully saturated as compared to the other specimens. The reason for this is that the flocculated structure does not transform to a dispersed structure under the effect of saturation alone. The structure of a fully saturated specimen still might be between dispersed and flocculated states thus making the specimen less compressible than the one with the dispersed structure. Sets of specimens prepared at a similar density but different methods of preparation, static and impact compaction, showed different amounts of compression under a similar range of loading. This behavior is undoubtedly due to a variation in the soil fabric.

Generally, specimens impact compacted dry of optimum moisture content (0% and 6%) tend to indicate higher swell than the statically compacted specimens. The opposite was observed in this study. The specimen which was statically compacted at the field moisture content, dried to zero moisture content, and then saturated exhibited a maximum swell of 6.62% which was greater than all the rest of the specimens. A possible explanation is that the soil structure might have been affected appreciably during the drying process. The desiccation tends to consolidate the soil and hence change the soil fabric.

CHAPTER 5

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary

A series of consolidation tests was conducted on a Southern Arizona copper mine tailings. The purpose was to investigate the effects of in-situ drying versus the effects of sample preparation disturbance on compressibility. The tailings material was a silty sand with highly angular grain shapes, rough grain surface texture and 45% by weight passing #200 sieve. The minus No. 200 fraction contains a small amount of an expansive clay mineral, montmorillonite, which caused the tailings to swell slightly when saturated in water.

The consolidation tests were performed on undisturbed as well as disturbed specimens. The undisturbed specimens were retrieved in the form of large size blocks from a preselected location in the tailings dam. Later, test specimens were carved into the consolidation rings from these undisturbed blocks. The disturbed specimens were prepared to a dry density of 95% of the maximum dry density and to the field dry density using impact and static compaction. All specimens were prepared and tested in triplicate. To study the effects of drying and wetting on the tailings (tailings undergo alternate cycles of drying and wetting) the consolidation tests were run on both partially and fully saturated specimens.

The consolidation test results indicate that the tailings do not have any past stress history because the e-log(p) curves did not reveal any preconsolidation pressure. However, the preconsolidation pressure expected from the specimens during this study was not supposed to be due to a past stress history. Rather, it was to develop due to the effect of desiccation.

For a comparative analysis, the slopes of all the e-logp curves were measured. Generally, the slopes of the curves for the undisturbed and statically compacted specimens were greater than those for the impact compacted specimens. This suggests that the former had a relatively higher reduction in the void ratio and hence a greater compression than the later. Further, all of the saturated specimens underwent more consolidation compression than the unsaturated specimens over the same range of loading. All of the saturated specimens were allowed to free swell under a surcharge pressure of 1/16 tsf prior to applying an additional load increment. The statically compacted specimens tested at an initial water content of 0% produced maximum swell of 6.62%.

Conclusions

The conclusions presented herein were either directly reached from the analysis of data gathered throughout this investigation or were indirectly inferred by comparing the test results of all the specimens with one another. The conclusions are as follows:

1. The one-dimensional consolidation (oedometer) test was found to

be less than adequate in modelling the problem. The effects of desiccation on preconsolidation pressure could not be tracked by this test.

2. The characteristics of the tailings are very much dependent on its location in the impoundment. The tailings in the pond contain more "slimes" than the beach material.

3. The material behavior is governed by the soil fabric. The major classification of fabrics used to interpret the results of this investigation are floocculated and defflocculated or dispersed soil fabrics. The specimens prepared dry of optimum showed less compression under the given range of loading due to their flocculated structure than the ones prepared wet of optimum which had dispersed structure. The effect of dry density was, therefore, overshadowed by the effect of the soil fabrics. The specimens prepared to the same density but with different methods of sample preparation showed different material behavior suggesting that the test results were influenced by the soil fabric. The initial water content showed an effect in that the specimens prepared dry of optimum exhibited more swell than the ones wet of optimum moisture content.

4. The amount of swelling as exhibited by this material would not affect the material behavior remarkably if the in-situ water content remains equal to or above 6%. In other words, the swelling would influence the tailings' physical behavior only if

the tailings underwent complete cycles of wetting and drying.

5. The effect of specimen preparation techniques tended to be more dominent than the in-situ drying. The impact compacted specimens were found to have undergone the least amount of compression of all the specimens.
6. There was a slight discrepancy in the preparation of the specimens. The dimensions of all the specimens could not be kept consistent as approximate manual techniques were employed in the specimen preparation. A slight variation, of a few grams, in the weight of the solids led to a relatively large error in the computation of the void ratio. This, however, did not influence the physical behavior of the tailings significantly except that the incorrect initial value of void ratio caused e-log(p) curves to plot scattered.

Recommendations

The following recommendations are made on the basis of the results of this research:

1. The effects of the in-situ drying versus the effects of methods of specimen preparation on the compressibility characteristics of mine tailings should be investigated further by using some other methods of specimen preparation which would model the field deposition of the tailings more closely. The methods of specimen preparation by wet pouring and sedimentation appear to be promising.

2. In addition to the effects of drying versus the effects of specimen preparation techniques, the effects of specimen disturbance caused by factors like stress-release, transportation, and handling should also be incorporated in a future sltudy. The effects of transportation, for instance, could be simulated by dropping the "tube" specimens to a hard surface from a known height.
3. The effects of dry density and soil fabric need further study. The future investigation should also include some microscopic study.
4. The test results for tailings from a specific location in the dam should not be genrealized. They shoud be used only for the area from which the tailings specimens were obtained. For instance, the material from the pond contains more slimes (fines) than the material at the beach. Hence, it could be expected to have an altogether different material behavior for specimens collected from different locations in the impoundment.
5. The effects of drying and specimen preparation techniques could not be determined very efficiently by the testing method used in this study as the parameter (p_c) chosen for this purpose could not be recorded. Consequently, the analyses had to rely on the stress-deformation behavior of the material. In the future, a comparative study such as this could be better conducted with the help of triaxial testing if the comparison of the effects of

drying versus specimen preparation techniques is to be made in the context of the stress-deformation behavior of the tailings. In other words, in the light of this experience, it might be better to study the effects of drying and sample preparation techniques on th strength characteristics of the tailings.

APPENDIX A

SUMMARY OF CONSOLIDATION TEST RESULTS

Table A.1. Consolidation test data for undisturbed specimens.

Tested as Molded $S_i(\%) = 0$				Saturated Before Loading $S_i(\%) = 96$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
0	0	1	0.789	0	0	1	0.888
	1/8		0.787		1/8		0.876
	1/4		0.785		1/4		0.867
	1/2		0.781		1/2		0.865
	1		0.780		1		0.852
	2		0.776		2		0.841
	4		0.769		4		0.822
0	0	2	0.779	0	0	2	0.901
	1/8		0.777		1/8		0.891
	1/4		0.775		1/4		0.875
	1/2		0.771		1/2		0.872
	1		0.768		1		0.860
	2		0.766		2		0.853
	4		0.763		4		0.840
0	0	3	0.811	0	0	3	0.891
	1/8		0.799		1/8		0.885
	1/4		0.796		1/4		0.874
	1/2		0.794		1/2		0.871
	1		0.791		1		0.863
	2		0.785		2		0.855
	4		0.771		4		0.840

MC_i = initial moisture content   tsf = tons per square foot

S_i = initial degree of saturation

Table A.1. Consolidation test data for undisturbed specimens.

Tested as Molded $S_i(\%) = 20$				Saturated Before Loading $S_i(\%) = 98$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
6	0	1	0.821	6	0	1	0.867
	1/8		0.819		1/8		0.859
	1/4		0.817		1/4		0.851
	1/2		0.816		1/2		0.849
	1		0.809		1		0.838
	2		0.796		2		0.829
	4		0.782		4		0.810
6	0	2	0.833	6	0	2	0.841
	1/8		0.825		1/8		0.837
	1/4		0.823		1/4		0.832
	1/2		0.820		1/2		0.826
	1		0.817		1		0.817
	2		0.801		2		0.811
	4		0.788		4		0.807
6	0	3	0.816	6	0	3	0.860
	1/8		0.808		1/8		0.854
	1/4		0.806		1/4		0.850
	1/2		0.802		1/2		0.846
	1		0.797		1		0.835
	2		0.781		2		0.828
	4		0.775		4		0.813

Table A.1. Consolidation test data for undisturbed specimens.

Tested as Molded $S_i(\%) = 41$				Saturated Before Loading $S_i(\%) = 94$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
14	0	1	0.958	14	0	1	0.900
	1/8		0.951		1/8		0.898
	1/4		0.949		1/4		0.881
	1/2		0.942		1/2		0.869
	1		0.931		1		0.861
	2		0.918		2		0.852
	4		0.908		4		0.845
14	0	2	0.961	14	0	2	0.921
	1/8		0.959		1/8		0.908
	1/4		0.951		1/4		0.898
	1/2		0.946		1/2		0.889
	1		0.932		1		0.875
	2		0.921		2		0.861
	4		0.909		4		0.854
14	0	3	0.965	14	0	3	0.908
	1/8		0.963		1/8		0.897
	1/4		0.957		1/4		0.885
	1/2		0.951		1/2		0.866
	1		0.941		1		0.860
	2		0.920		2		0.855
	4		0.911		4		0.843

Table A.1. Consolidation test data for undisturbed specimens.

Tested as Molded $S_i(\%) = 62$				Saturated Before Loading $S_i(\%) = 98$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
23	0	1	0.985	23	0	1	0.949
	1/8		0.969		1/8		0.920
	1/4		0.963		1/4		0.917
	1/2		0.949		1/2		0.907
	1		0.934		1		0.894
	2		0.919		2		0.882
	4		0.910		4		0.873
23	0	2	0.978	23	0	2	0.951
	1/8		0.961		1/8		0.924
	1/4		0.959		1/4		0.916
	1/2		0.947		1/2		0.911
	1		0.931		1		0.895
	2		0.920		2		0.880
	4		0.914		4		0.875
23	0	3	0.980	23	0	3	0.962
	1/8		0.971		1/8		0.931
	1/4		0.965		1/4		0.925
	1/2		0.950		1/2		0.914
	1		0.939		1		0.891
	2		0.918		2		0.883
	4		0.912		4		0.871

Table A.2. Consolidation test data for disturbed specimens, impact compacted to 95% of maximum dry density.

Tested as Molded $S_i(\%) = 0$				Saturated Before Loading $S_i(\%) = 98$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
0	0	1	0.514	0	0	1	0.678
	1/8		0.511		1/8		0.667
	1/4		0.510		1/4		0.659
	1/2		0.509		1/2		0.651
	1		0.507		1		0.646
	2		0.503		2		0.645
	4		0.498		4		0.644
0	0	2	0.509	0	0	2	0.681
	1/8		0.507		1/8		0.678
	1/4		0.504		1/4		0.661
	1/2		0.501		1/2		0.653
	1		0.498		1		0.648
	2		0.497		2		0.644
	4		0.493		4		0.690
0	0	3	0.510	0	0	3	0.710
	1/8		0.508		1/8		0.692
	1/4		0.505		1/4		0.685
	1/2		0.502		1/2		0.679
	1		0.500		1		0.661
	2		0.497		2		0.656
	4		0.95		4		0.648

Table A.2. Consolidation test data for disturbed specimens, impact compacted to 95% of maximum dry density.

Tested as Molded $S_i(\%) = 40$				Saturated Before Loading $S_i(\%) = 97$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
6	0	1	0.402	6	0	1	0.440
	1/8		0.397		1/8		0.428
	1/4		0.397		1/4		0.427
	1/2		0.396		1/2		0.425
	1		0.395		1		0.425
	2		0.393		2		0.424
	4		0.391		4		0.423
6	0	2	0.399	6	0	2	0.451
	1/8		0.397		1/8		0.436
	1/4		0.396		1/4		0.425
	1/2		0.394		1/2		0.423
	1		0.392		1		0.422
	2		0.391		2		0.421
	4		0.390		4		0.420
6	0	3	0.413	6	0	3	0.447
	1/8		0.398		1/8		0.424
	1/4		0.397		1/4		0.423
	1/2		0.394		1/2		0.421
	1		0.392		1		0.419
	2		0.390		2		0.418
	4		0.388		4		0.417

Table A.2. Consolidation test data for disturbed specimens, impact compacted to 95% of maximum dry density.

Tested as Molded $S_i(\%) = 80$				Saturated Before Loading $S_i(\%) = 100$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
14	0	1	0.479	14	0	1	0.495
	1/8		0.465		1/8		0.480
	1/4		0.464		1/4		0.480
	1/2		0.463		1/2		0.478
	1		0.459		1		0.476
	2		0.456		2		0.475
	4		0.455		4		0.457
14	0	2	0.481	14	0	2	0.503
	1/8		0.471		1/8		0.494
	1/4		0.466		1/4		0.492
	1/2		0.462		1/2		0.488
	1		0.457		1		0.471
	2		0.455		2		0.468
	4		0.451		4		0.457
14	0	3	0.470	14	0	3	0.510
	1/8		0.458		1/8		0.495
	1/4		0.457		1/4		0.489
	1/2		0.456		1/2		0.487
	1		0.454		1		0.477
	2		0.452		2		0.475
	4		0.450		4		0.448

Table A.3. Consolidation test data for disturbed specimens, statically compacted to 95% of maximum dry density.

Tested as Molded $S_i(\%) = 0$				Saturated Before Loading $S_i(\%) = 96$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
0	0	1	0.721	1	0	1	0.738
	1/8		0.718		1/8		0.712
	1/4		0.716		1/4		0.696
	1/2		0.712		1/2		0.695
	1		0.710		1		0.693
	2		0.700		2		0.692
	4		0.686		4		0.688
0	0	2	0.718	2	0	2	0.731
	1/8		0.712		1/8		0.718
	1/4		0.709		1/4		0.701
	1/2		0.707		1/2		0.698
	1		0.705		1		0.695
	2		0.701		2		0.691
	4		0.693		4		0.686
0	0	3	0.728	3	0	3	0.718
	1/8		0.719		1/8		0.698
	1/4		0.717		1/4		0.689
	1/2		0.713		1/2		0.678
	1		0.711		1		0.676
	2		0.708		2		0.673
	4		0.680		4		0.665

Table A.3. Consolidation test data for disturbed specimens, statically compacted to 95% of maximum dry density.

Tested as Molded $S_i(\%) = 16$				Saturated Before Loading $S_i(\%) = 94$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
6	0	1	0.992	6	0	1	0.908
	1/8		0.984		1/8		0.878
	1/4		0.983		1/4		0.872
	1/2		0.982		1/2		0.860
	1		0.978		1		0.859
	2		0.968		2		0.855
	4		0.936		4		0.850
6	0	2	1.03	6	0	2	0.921
	1/8		0.995		1/8		0.889
	1/4		0.993		1.4		0.875
	1/2		0.992		1/2		0.866
	1		0.976		1		0.855
	2		0.959		2		0.850
	4		0.929		4		0.846
6	0	3	0.988	6	0	3	0.910
	1/8		0.982		1/8		0.879
	1/4		0.980		1/4		0.873
	1/2		0.979		1/2		0.861
	1		0.975		1		0.857
	2		0.963		2		0.849
	4		0.931		4		0.841

Table A.3. Conslidation test data for disturbed specimens, statically compacted to 95% of maximum dry density.

	Tested as Molded $S_i(\%) = 38$				Saturated Before Loading $S_i(\%) = 99$		
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
14	0	1	1.00	14	0	1	0.979
	1/8		0.999		1/8		0.971
	1/4		0.997		1/4		0.970
	1/2		0.996		1/2		0.968
	1		0.993		1		0.966
	2		0.989		2		0.962
	4		0.974		4		0.942
14	0	2	1.05	14	0	2	0.981
	1/8		1.02		1/8		0.979
	1/4		0.995		1/4		0.971
	1/2		0.994		1/2		0.968
	1		0.993		1		0.965
	2		0.985		2		0.960
	4		0.971		4		0.939
14	0	3	0.999	14	0	3	0.970
	1/8		0.997		1/8		0.969
	1/4		0.994		1/4		0.967
	1/2		0.993		1/2		0.964
	1		0.990		1		0.961
	2		0.985		2		0.958
	4		0.971		4		0.936

Table A.4. Consolidation test data for disturbed specimens, impact compacted to field density.

Tested as Molded $S_i(\%) = 0$				Saturated Before Loading $S_i(\%) = 96$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
0	0	1	0.613	0	0	1	0.585
	1/8		0.610		1/8		0.583
	1/4		0.608		1/4		0.583
	1/2		0.606		1/2		0.580
	1		0.605		1		0.573
	2		0.604		2		0.565
	4		0.601		4		0.555
0	0	2	0.630	0	0	2	0.603
	1/8		0.619		1/8		0.602
	1/4		0.614		1/4		0.598
	1/2		0.612		1/2		0.596
	1		0.611		1		0.587
	2		0.609		2		0.585
	4		0.607		4		0.578
0	0	3	0.614	0	0	3	0.580
	1/8		0.611		1/8		0.578
	1/4		0.609		1/4		0.576
	1/2		0.607		1/2		0.573
	1		0.604		1		0.568
	2		0.601		2		0.557
	4		0.600		4		0.549

Table A.5. Consolidation test data for disturbed specimens, statically compacted to field density.

Tested as Molded $S_i(\%) = 0$				Saturated Before Loading $S_i(\%) = 95$			
MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)	MC_i (%)	Load (tsf)	Test No.	Final Void Ratio (e_f)
0	0	1	0.923	0	0	1	1.040
	1/8		0.918		1/8		1.034
	1/4		0.918		1/4		1.028
	1/2		0.917		1/2		1.015
	1		0.916		1		0.994
	2		0.911		2		0.984
	4		0.902		4		0.980
0	0	2	0.953	0	0	2	1.038
	1/8		0.931		1/8		1.035
	1/4		0.909		1/4		1.026
	1/2		0.897		1/2		1.016
	1		0.894		1		0.997
	2		0.890		2		0.981
	4		0.882		4		0.979
0	0	3	0.931	0	0	3	1.042
	1/8		0.924		1/8		1.033
	1/4		0.917		1/4		1.030
	1/2		0.913		1/2		1.013
	1		0.910		1		0.990
	2		0.908		2		0.984
	4		0.904		4		0.980

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