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FINAL REPORT

COMPACTION AND SETTLEMENT OF EXISTING EMBANKMENTS

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16 Abstract

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Eight embankments constructed between 1994 and 2000 were selected for undisturbed field sampling. Two borings were drilled in each embankment and shelby tube samples were collected for testing at regular intervals. Samples of the cuttings were also collected for testing.

A telephone survey of all state DOT's was conducted to assess current practice with regard to specifications for compaction of fills. Thirty-two states, including Kansas, responded to the survey. It was determined that a number of compaction and moisture specifications are currently in use, however there were common themes among the specifications.

Based on the results of this research it is recommended that KDOT specify a relative compaction standard and a moisture content range based on the optimum moisture content for compaction of embankments. It is recommended that the compaction standard be at least 95 percent of maximum density as determined by KT-12 (AASHTO T 99), or an equivalent relative compaction based on modified effort (AASHTO T 180).

It is also recommended that the specified moisture range be centered about optimum or a point slightly above optimum, as the average moisture content of the existing embankments is slightly above optimum.

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PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

Unanticipated settlement of compacted earth fill has been a continuing problem for embankments managed by the Kansas Department of Transportation (KDOT). This report contains the results of an investigation of current compaction specifications, with particular emphasis on the Type B compaction specification that relies on visual verification of compaction by sheepsfoot roller walkout. This investigation consisted of two parts: a field investigation of existing embankments with varying levels of performance and a telephone survey of other Departments of Transportation to determine the status of compaction specifications.

Eight embankments constructed between 1994 and 2000 were selected for undisturbed field sampling. Two borings were drilled in each embankment and shelby tube samples were collected for testing at regular intervals. Samples of the cuttings were also collected for testing.

At the time of collection, 97 percent of the shelby tube samples had a relative compaction exceeding 90 percent, based on standard effort "standard Proctor" as defined by KT-12 (AASHTO T 99). Those embankments with a lower average relative compaction did not perform as well as those with a higher relative compaction. However, even the embankments that performed poorly had an average relative compaction significantly above 90 percent. Type B compaction is defined by sheepsfoot roller walkout rather than a specific relative compaction standard, however it can be presumed that the equivalent relative compaction standard is not greater than that of Type A compaction, which specifies that 90 percent relative compaction be achieved. This suggests that compaction of fill in accordance with the existing standard is insufficient to assure satisfactory performance.

A telephone survey of all state Departments of Transportation was conducted to assess current practice with regard to specifications for compaction of fills. Thirty-two states, including Kansas, responded to the survey. It was determined that a number of compaction and moisture specifications are currently in use, however there were common themes among the specifications. Twenty-seven of 32 states require that a specific relative compaction be achieved. The most common compaction specification (19 of 32 states) was 95 percent of standard effort as defined by AASHTO T 99 (KT-12). Twenty-five states use the nuclear gauge as the primary method for verifying compaction. Twenty-five states specify a numerical range or target value based on the optimum moisture content. The most common individual moisture specification was optimum ±2 percent (7 of 32 states). Iowa is the only other state still using a visual specification for compaction of fill. Iowa is currently engaged in a detailed review of its compaction specifications.

Based on the results of this research, it is recommended that KDOT specify a relative compaction standard and a moisture content range based on the optimum moisture content for compaction of embankments. It is recommended that the compaction standard be at least 95 percent of maximum density as determined by KT-12 (AASHTO T 99), or an equivalent relative compaction based on modified effort (AASHTO T 180). An additional option that may be considered is scheduling a delay between placement of fill and finish grading to give the fill time to settle prior to compaction.

Adoption of a relative compaction standard will require the use of verification equipment in the field. A number of technologies exist to verify compaction, including the nuclear density gauge, sand cone, balloon, and drive tubes. Use of the dynamic cone penetrometer (DCP) may also provide valuable information about the condition of compacted fill.

It is also recommended that the specified moisture range be centered about optimum or a point slightly above optimum, as the average moisture content of the existing embankments is slightly above optimum.

Further research is recommended to more thoroughly evaluate the current KDOT compaction specifications. Iowa has been investigating its compaction specifications for several years and it is recommended that the results of their study be monitored and that a joint study be considered.

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1.1 Introduction

Soil is the most abundant construction material and is an integral part of most construction projects. Soil available for use may or may not have desirable engineering properties. When the engineering properties of the soil on site do not meet requirements of the project the engineer has five options:

1) Abandon the project,
2) Move the project to an area with "better" soil,
3) Exchange the on site soil with more desirable soil,
4) Redesign the project to better handle the poor conditions, or
5) Modify the existing soil to improve the engineering properties.

Abandoning or moving a project are not usually viable options. Removal and replacement of the existing soil and redesigning the project are expensive options. This leaves modification of the soil as the most feasible option for improving poor soil conditions for most projects.

1.2 Background

There are four major groups of soil modification techniques used in construction today: mechanical, hydraulic, physical/chemical, and inclusion or confinement. The most common technique is mechanical modification of the soil by increasing its unit weight with mechanical force applied using compaction equipment.

Recently the KDOT has been experiencing unanticipated settlement at some of their earth embankments. Improper compaction of the embankments during construction is a possible cause of this substandard performance.

The current KDOT compaction specification has multiple levels of moisture content and density control. These include: Moisture Range 0-5 (MR-0-5), MR-3-3, MR-5-5, and MR-90 for moisture content control and Type AAA, Type AA, Type A, Type B, and Type C for density control. Each of the moisture content ranges allows a different level of deviation from optimum moisture content; i.e.MR-3-3 specifies the compaction be within ±3 percent of optimum moisture content. The three types of "A" density specification each specify a different minimum density, while types "B" and "C" are visual verification specifications.

Of these five types of density and moisture control, Type B/MR-90 Compaction is commonly specified for embankment construction. This specification places a great deal of responsibility on the field personnel's judgment because it requires visual inspection of the soil for moisture content control and sheepsfoot roller walkout for density control. Embankments with unanticipated settlement problems prompted this review of compaction and quality assurance procedures.

The project described in this report was part of a review process that KDOT is currently conducting on their compaction specifications and quality assurance procedures. For this project, eight embankments were selected for investigation. At each site two holes were drilled and shelby tubes were pushed at regular intervals. The shelby tube samples were then used for compaction verification, consolidation testing and strength testing. Trimmings were collected off the auger flights to perform compaction tests, grain size distribution, Atterberg Limits, specific gravity and collapse tests. Dynamic cone penetrometer (DCP) tests were performed at six of the embankments investigated in this project. A review of other state's compaction specifications was conducted by a short phone survey. An attempt to contact all 50 states was made. Thirty-two states, including Kansas, responded to the survey.

The information presented in this paper is organized into seven chapters. Chapter One is an introduction to the project. In Chapter Two, the selected literature pertaining to the project was reviewed. The various testing procedures can be found in Chapter Three. The results of the testing followed by discussion of the results can be found in Chapters Four and Five respectively. Finally, the conclusions and recommendations for improving embankment construction can be found in Chapters Six and Seven.

Chapter 2

Literature Review

The available literature on compacted earth fill was reviewed and summaries of the important findings are presented in this chapter. The history of compaction and why compaction improves the engineering properties of soil is presented. The available literature on the settlement mechanisms of compacted earth fill was also reviewed.

2.1 Compaction

Naturally occurring soils often do not have desirable engineering properties. More often than not soils must be improved in order to perform as required by an engineer. The most common type of soil improvement is soil compaction, also referred to as mechanical manipulation or densification.

2.1.1 Types of Compaction

Coduto describes the four major types of compaction processes currently in use by modern construction equipment (10). Compaction equipment may incorporate one or more of the following methods to compact a soil:

1) Impact
2) Manipulation
3) Pressure
4) Vibration

Impact compaction involves dropping a weight on the soil during compaction. This compaction equipment, usually a tamping foot-roller type, subjects the soil to a series of blows until the desired density is reached (10). In order to effectively compact a soil with an impact-type compaction process; the soil must be placed in multiple lifts so that the stress of the blow is distributed through the entire lift. Another form of impact compaction is known as deep dynamic compaction. This type of compaction uses a crane and a very large mass to compact a soil to a significant depth below the surface.

R. R. Proctor first studied the process of soil compaction in the 1930s and presented his findings in a series of four articles in the Engineering News-Record (28). The principles he is credited with discovering apply to many different types of soil compaction. During his research he developed the procedure for determining moisture-density relationships for cohesive soil, often referred to as the Proctor test.

The Proctor test (ASTM D-698/AASHTO T 99) uses an impact compaction procedure by dropping a mass onto a soil sample in a mold. The height of drop, size of the mass, size of the mold, number of blows, and number of layers is described in various testing procedures developed over the last 70 years. The test is repeated on multiple samples at different moisture contents in order to develop a moisture-density curve similar to the one shown in Figure 2.1. Moisture-density curves are a very common way to present compaction information. The main points on a typical moisture-density curve are:

1) Optimum Moisture Content (OMC)
2) Maximum Dry Density (MDD)

Both pieces of information are gathered from the peak of the curve as shown in Figure 2.1. Compaction dry of OMC typically results in a soil that has higher strength, lower compressibility, and increased propensity for shrink/swell behavior. Compaction wet of OMC usually results in a soil that has lower strength, higher compressibility, and less shrink/swell behavior (15). A comparison between compaction of soil dry of OMC versus wet of OMC is presented in Table 2.1.

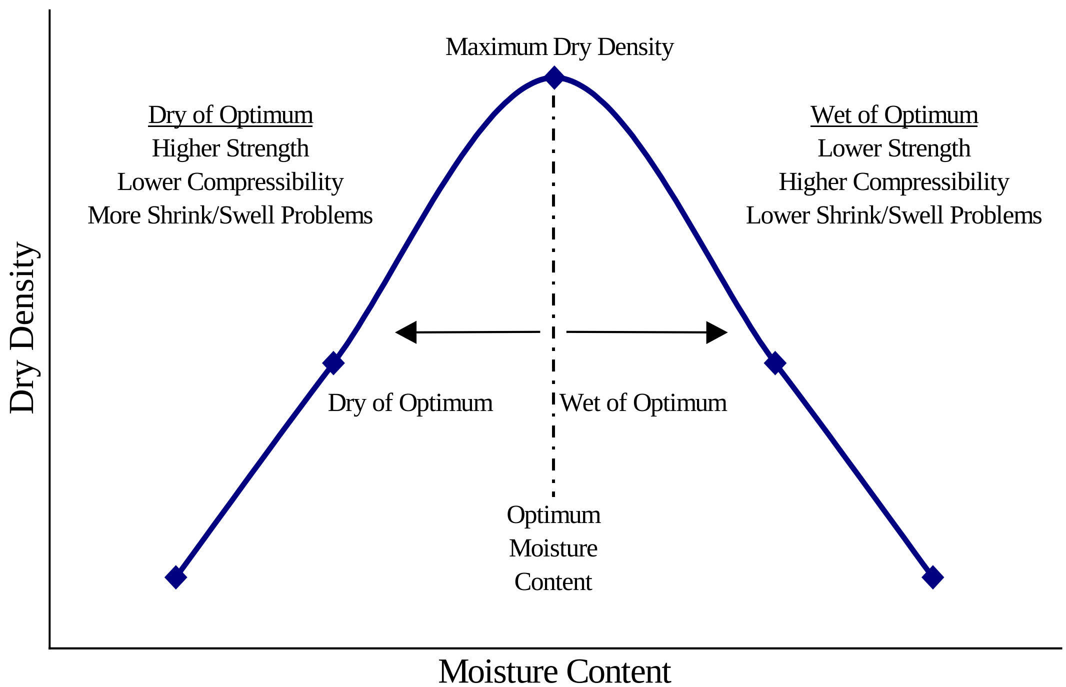


FIGURE 2.1 Typical Proctor Curve and Engineering Properties of Soils (7)

TABLE 2.1 Dry of Optimum vs. Wet of Optimum Compaction Characteristics (15)

	Property	Comparison
1.	**Structure**	
	A. Particle Arrangement	Dry side more random.
	B. Water Deficiency	Dry side more deficiency therefore imbibe more water, swell more, have lower pore pressure.
	C. Performance	Dry side structure sensitive to change.
2.	**Permeability**	
	A. Magnitude	Dry side more permeable.
	B. Performance	Dry side permeability reduced much more by permeation.
3.	**Compressibility**	
	A. Magnitude	Wet side more compressible in low-pressure range, dry side in high-pressure range.
	B. Rate	Dry side consolidates more rapidly.
	C. Rebound	Wet side rebound per compression greater.
4.	**Strength**	
	A. As Molded	
	a. Undrained	Dry side much higher.
	b. Drained	Dry side somewhat higher.
	B. After Saturation	
	a. Undrained	Dry side somewhat higher if swelling prevented; wet side can be higher if swelling permitted. Dry side about the same or slightly greater.
	b. Drained	
	C. Pore Water Pressure at Failure	Wet side higher.
	D. Stress-Strain Modulus	Dry side much greater.
	E. Sensitivity	Dry side more apt to be sensitive.

Compaction performed by manipulation or kneading is accomplished by introducing a shearing force to the soil during compaction (10). The construction equipment kneads the soil over a series of passes until the desired level of compaction is achieved.

The Proctor test does not accurately model the manipulation mechanisms of this type of compaction. A test was developed to more accurately model the mechanisms of compaction done by modern construction equipment such as sheepsfoot and tamping rollers. The most common type of kneading compaction test is typically referred to as the Miniature Harvard Compaction Test. The test produces a 25.3 mm diameter sample by tamping a sample of soil with a calibrated spring-loaded piston (11). While neither ASTM nor AASHTO currently has a recommended procedure for use of the Harvard Miniature Mold apparatus for compaction testing it is commonly used for research purposes.

Pressure is probably the most important type of compaction mechanism for compaction of soils (10). Historically, compaction was first accomplished by foot and animal traffic when ancient civilizations compacted soils for levees and dams. Today, pressure construction equipment incorporates drum, pneumatic, or sheepsfoot rollers to compact soils.

Using pressure to compact a laboratory sample is typically called static compaction. This method of research allows the researcher to compact a specimen into a sample at a specific density. Usually a consolidation apparatus is used to compress the soil into a ring of known volume. Hausmann notes that static compaction is a useful research method but the researcher must realize that in the field the contractor will not have the same level of control (11).

Vibratory compaction is used to shake the soil into a more dense state. The compaction equipment induces strong vibrations in the soil by eccentric weights, usually in the range of 1000 to 3500 cycles per minute.

Modern compaction equipment typically incorporates more than one type of compaction mechanism to accomplish compaction of the soil. Selection of the proper compaction method depends on the type of soil, the size of the project, final compaction requirements, rate of production, and economic factors (10). The most typical application and the compaction mechanism used by compaction construction equipment are summarized in Figure 2.2.

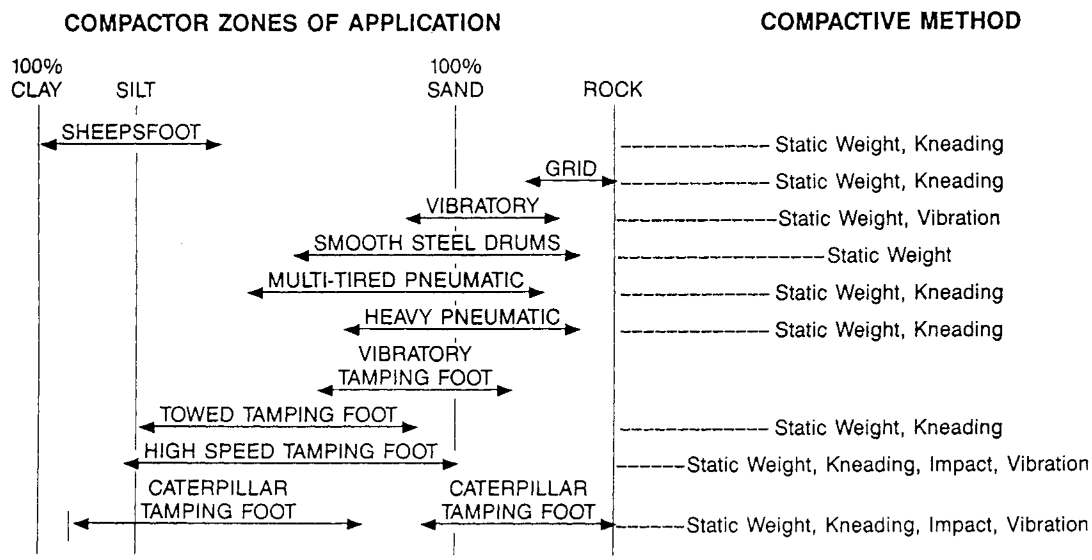


FIGURE 2.2 Compactor Applications and Methods (9)

It is important to understand that different types of compaction tests have been developed to more accurately model field compaction. The engineers and contractors need to understand that soil properties change rapidly and multiple compaction tests may be required to remain abreast of the changing material properties during construction. Terzaghi makes a valid point about compaction methods and testing:

> "Because of the influence of the method of compaction on the moisture-density curve, no standard test of any kind, including the Proctor test, should be expected to produce results of general validity. Conclusive information on the optimum moisture content can be obtained only by making large-scale field tests with the compacting equipment to be used on the job. (30)

While full scale testing may be impractical, it is important to realize that it is the best way to determine material properties for almost all cases.

2.1.2 Fundamentals of Compaction

Fundamentally, the compaction process causes the expulsion of air from the soil resulting in a denser material. The theory of why compaction results in a denser material and why there is a limit to the obtainable maximum density has been researched since Proctor first introduced his findings. The compaction process has significant differences for cohesive soil versus cohesionless soils. The major difference is cohesive soils are typically very moisture dependent and cohesionless soils are not (35). Most soils used for embankment construction in Kansas are cohesive and therefore they are the focus of this literature review.

Proctor recognized that moisture content affects the compaction process. He believed the reason why a moisture-density curve "breaks over" at optimum moisture content was related to capillarity and frictional forces (28). He also believed that the force of the compactive effort was applied to overcoming the inter-particle friction of the clay particles. As the water content increased from dry of optimum to wet of optimum he believed that the water acted as a lubricant between the soil particles. The addition of more water continued to aid the compaction process until the water began replacing the air voids. At this point the compaction process was complete and the addition of more compactive energy would not result in a denser soil.

Hogentogler, summarized by Hausmann, introduced the next major compaction theory. He also thought that water was a lubricant in the compaction process (11). He described compaction along the moisture density curve from dry to wet as a four-step process. First, the soil particles become hydrated as water is absorbed. Second, the water begins to act as a lubricant helping to rearrange the soil particles into a denser and denser state until optimum moisture content is reached. Third, the addition of water causes the soil to swell because the soil now has excess water. Finally, the soil approaches saturation as more water is added. His theory was disproven when research showed that the compaction process does not result in complete saturation and compaction wet of optimum moisture content will tend to parallel the zero-air-voids curve rather than intersect it.

Hilf (1956), as summarized by Winterkorn and Fang, presented another theory about the compaction phenomenon (35). He based his theory on pore water pressures in unsaturated soils. He developed a curve based on void ratio and water-void ratio. The curve looks similar to a typical moisture-density curve because the minimum void ratio is also the maximum density-optimum moisture content point on a moisture density curve. Capillary pressure and pore pressure explain the shape of the curve. The menisci of the water in a drier soil have a high curvature and are stronger than a wetter soil with flatter menisci. The decrease in density after optimum moisture content was attributed to the trapping of air bubbles and a build up of pore pressure lowering the effectiveness of compaction. The build up of negative pore pressure in the moisture films surrounding the soil particles were assumed to be interconnected and resulted in an effective compressive stress on the soil skeleton equal to the negative pressure.

Subsequent research has shown that capillary pressure may not act as an effective stress in unsaturated soils. An X factor that varies from 0 to 100 with saturation relates how much the capillary pressure acts as an effective stress. This X factor is very difficult to measure and therefore Hilf’s theory is still controversial (35).

Lambe studied the microstructure of clays and developed a theory based on physicochemical properties (15). He found that compaction of a soil dry of optimum moisture content results in a flocculated soil structure that has high shear strength and permeability. Compaction of a soil wet of optimum moisture content results in a soil with a dispersed soil structure that has low shear strength and permeability. While this information does not directly explain the shape of the compaction curve, it does help explain the strength and volume characteristics of compacted soils (15).

According to Winterkorn and Fang, the compaction theories based on effective stresses explain the shape of the compaction curve better than the theories based on lubrication and viscous water (35). The facts that soils are anisotropic and heterogeneous complicate the research process and add validity to Terzaghi's remarks about large-scale field tests. Burmister stated that the natural variability of soil is too great to effectively classify soils for engineering works (8). He believed that while identification of the soil is important it should not take the place of actual testing of the soil being used in the project.

2.1.3 Compaction Control

"Although compaction is an important part of all earthwork projects, it is often treated casually" (29). Good compaction control begins with soil preparation prior to compaction. The engineer must demand that the soil be mixed as evenly as possible so that the moisture and particle distribution are uniform. Agrawal and Altschaeffl concluded from their research on water content variability in clay embankments that the water content range in each lift was the most important aspect to good embankment construction (1). They also suggest efforts to control the water content at the time of compaction in a more stringent manner may offer a significant cost savings for the project.

Once the soil is as homogenous as possible, the soil must be placed in lifts of specified thickness. If the engineer does not validate the lift thickness, soft layers may develop during the compaction process. Sheepsfoot rollers and lift thickness are fundamentally related. Sheepsfoot rollers begin compaction by compacting the soil below the bottom of the foot and after a number of passes the roller gradually "walks out" of the lift (12). Some densification of lower lifts continues during the compaction of subsequent lifts. If the engineer allows the lift thickness to vary significantly, soft spots can develop and may eventually cause problems in the finished product. A soft layer sandwiched in between two denser layers can be thought of as an "Oreo cookie" effect (34).

The control of moisture during and after compaction is often the most difficult to control. Natural variation in moisture content during excavation and rainy days during the construction season makes moisture control very difficult. Soil that is too dry must have water added. This water is usually only added to the surface and not mixed evenly throughout the lift. This may cause uneven compaction depending on the type of roller used. If the soil is too wet this slows construction because the soil must be given a chance to dry either naturally or by disking. Iowa DOT found in a review of their embankment specification that disking of a wet soil was very effective in improving compaction but contractors rarely disked and field engineers rarely demanded disking (34).

2.1.4 Compaction Specifications

The variability of compacted fill can make it very hard to control due to natural material variability as well as diurnal variations in moisture content. Noorany studied the variability of compaction control and concluded that better fill-placement techniques are urgently required to improve structural fills (24). Collection of knowledge about improving compaction is of little value without an effective way to control compaction in the field. Compaction specifications are the most common way to ensure that soils are compacted to a more useful state.

When writing a compaction specification it is important to remember why compaction of soil is necessary and what will be the final result.

> "Essentially, compaction of soils is a planned and organized method for stabilizing soils and for improving their performances by incorporating definite density and strength properties by the processes of compaction. Compaction tests should establish practical and definite bases: first, for the selection of the more suitable workable soils, and second, for the determination of practical criteria for effective field control of compaction of soils" (8).

The most common form of compaction specification is the density specification. Selig pointed out that this is true even though density is only an indirect measure of the properties desired from compaction (29). There are multiple reasons why Selig suggests the density specification has remained the most common approach:

1) The density test is simple and inexpensive;
2) A lower bound specification without moisture control can be written;
3) It can be applied to most soil conditions; and
4) An established procedure that is widely accepted.

The density specification has limitations that must be understood by the people writing the specification as well as the people applying it to a construction project because the desired properties can be competing and therefore a compromise must be accepted.

The density specification was first used to try to account for the natural variability in soils. According to Selig when writing a specification one must know the answers to these questions (29):

1) How much compaction is needed to meet the desired engineering requirements?
2) How should compaction be specified?
3) How should the results be verified in the field?

Field verification during construction is the most critical step. Without verification that the specified level of compaction was achieved during construction, there is no assurance the specification has been met. Field density measurements are often made during and after construction. The most common forms of field density verification tests are; nuclear density gauge (ASTM D-2922/AASHTO T-238), sand cone (ASTM D-1556/AASHTO T-191), and the rubber balloon method (ASTM D-2167/AASHTO T-205). The drive cylinder (ASTM D-2937/AASHTO T-204), shelby tube sample collection, dynamic cone penetration, and visual verification by sheepsfoot roller walkout are also used. Field density tests must be fast, inexpensive, reliable, and repeatable so testing does not slow the rate of production. The field density measurements obtained during construction are compared with the densities obtained during laboratory testing for evaluation of how well the soil was compacted.

A good specification includes a written action procedure for when a test fails to meet the specification. Selig suggests the following six-step procedure (29):

1) Repeat the field test, if the second test passes accept the compaction. If the second test does not pass go to step two.
2) Require more compaction from the contractor and retest. If the material still fails go to step three.
3) Consider another type of compactor. Try different compactor and retest. If the test still fails go to step four.
4) Rerun the laboratory test. Compare the previous compaction tests to the new compaction curve. If the test still fails go to step five.
5) Redo compaction by scarifying or removal and replacement of the soil and retest. If the material fails go to step six.

6) Consider lowering the minimum density required.

The specification needs to take in account that compaction of soil will have some natural variability while understanding that the end product must perform as required by the engineer. If the material cannot perform as required, the design engineer must be informed so that appropriate changes can be made before construction continues.

Benson and Daniel have done extensive research on the compaction of clay and concluded that a compaction specification that does not include the line of optimums may result in poor compaction (6). Their research has primarily focused on the compaction of clay for landfill liners and therefore may not apply to all type of compaction. The compaction of clay for landfill liner applications requires that the compaction stay wet of optimum to lower permeability. Their research has shown that compaction specifications based on percent compaction and intended to require compaction wet of optimum moisture content can result in compaction dry of optimum moisture content because optimum moisture content varies with compactive effort.

2.2 Embankment Settlement

Construction of an embankment introduces a significant load on the existing soil. If the foundation material is not properly prepared it may settle excessively under the increased load. Also, if the embankment is not constructed properly with care given to material selection and placement it may settle excessively.

2.2.1 Introduction to Consolidation

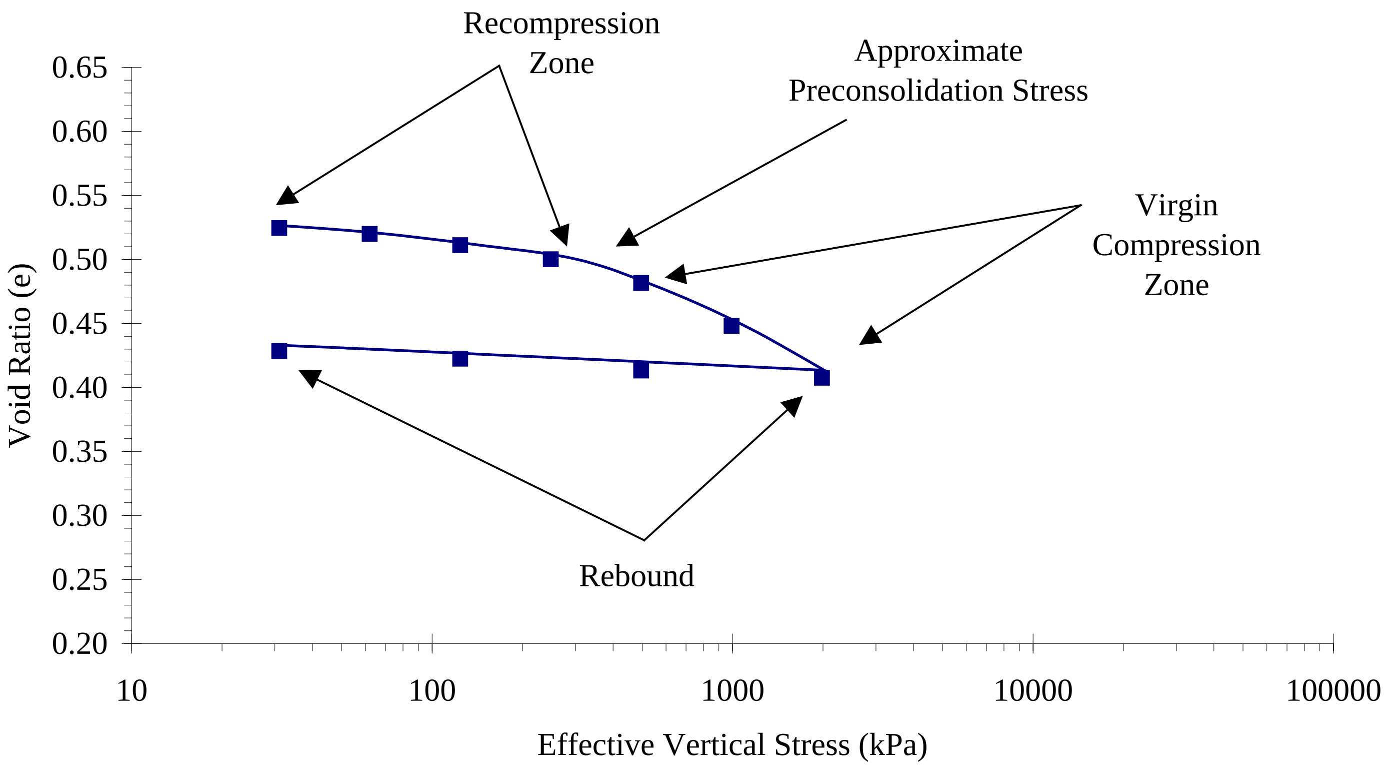
Consolidation is another form of volume reduction for cohesive soils. When a cohesive soil with low permeability is loaded, the water is squeezed out of the pores over a period of time ranging from minutes to years. The volumetric strain produced is formally known as consolidation. Karl Terzaghi first introduced his consolidation theory in 1929 in a paper titled, "Settlement Analysis – the Backbone of Foundation Research" (27). The terms compaction and consolidation are often erroneously used interchangeably because both involve volume reduction. The major difference between compaction and consolidation is that compaction involves the reduction of volume through the removal of air voids, and consolidation is the reduction of volume through the dissipation of pore pressure in saturated soils by the removal of water.

2.2.2 Consolidation Testing

Terzaghi developed equations that estimate the settlement of a cohesive soil based on the assumption of one-dimensional consolidation (35). The assumption is valid for large soil areas and for vertical loading. Terzaghi also assumed that the soil is located below the water table and therefore the soil is 100 percent saturated.

Laboratory consolidation testing is performed in a device called an oedometer or a consolidometer. A sample is carefully trimmed and placed into a confining ring and porous stones are placed on the top and bottom of the sample in order to allow free drainage from the sample (ASTM D-2435).

Once the sample is prepared, it is loaded and unloaded over a number of days with deformation of the sample carefully measured. Each new load is left on the sample long enough for primary consolidation to occur. This cycle is repeated until a maximum load is reached. Once at the peak, the load is incrementally removed until the initial load is reached. The void ratio is calculated at each load and then plotted on a semi-logarithmic graph. An example of the graph is shown in Figure 2.3.



<u>FIGURE 2.3 Example of a Consolidation Graph</u>

Void ratio is usually how consolidation deformation information is presented. Equations have been developed to estimate the settlement at any load using the void ratio. Soil deforms under the increasing stress and therefore a stress-strain relationship can be developed as described by Holtz and Kovacs (12).

2.2.3 Embankment Settlement

When an embankment fails from an unanticipated settlement it can be difficult to know the true source of the failure. Failure may result from unanticipated settlement of the foundation soils, the embankment fill, or a combination of the two.

2.2.3a Foundation Failures

When an embankment fails the first area investigated for the cause of the failure is often the foundation material. There are many reasons to suspect the foundation soil first. It is possible that only a limited site investigation was conducted. If so, little may be known about the material properties and the presence of abnormalities such as soft layers in the subsurface.

If an adequate site investigation with good laboratory testing was performed, the predicted settlement of the embankment should be reasonably well known and properly designed for (30). Because of this, recent research on foundation failures of embankments has focused on embankments built on soft soils and old landfills. This research has shown that the settlement analyses that are most commonly used in practice will produce satisfactory results. Mesri's extensive research on settlement analysis of soft clay soils stressed the need for high quality soil sampling and laboratory testing in order to accurately predict embankment settlement (21). He also noted that with computer analysis it is no longer necessary to idealize the subsurface profile into one or two homogeneous layers. Anderson concluded that a disparity between actual and predicted settlement magnitudes is due to a "gross underestimation of settlement rates," not the application of consolidation theory (5).

Multiple classical and modern mitigation techniques to reduce the settlement of embankment on soft soils were reviewed by Jean-Pierre Magnan. He reviewed techniques such as: preloading, soil replacement, stone columns, lightweight fill, inclusion, jet grouting, and geotextiles. He concluded that no general conclusions could be drawn due to the demands and wide variability of construction projects. He does point out how the evolution of construction and recent experience with various techniques has lead to new techniques that work well to reduce the settlement of embankments. He also noted that there is a need for review of classic techniques to see if modern construction equipment allows them to perform better than previously believed (20).

2.2.3b Embankment Failures

If the failure of the foundation material can be excluded as the cause of the embankment settlement the investigation turns to the embankment itself. There does not appear to be very much research into the general failure of embankments from internal stability problems. There is good justification for the lack of research because the construction of an embankment is usually tightly controlled and well specified. After an embankment has failed it is usually not possible to rule out foundation failure because monitoring equipment has not been in place to measure the settlement of the foundation separate from the embankment.

Embankments constructed in New York State in the early 1990s were analyzed for internal stability after foundation stability questions were ruled out from field investigations. From the information collected in this research they concluded that moisture contents in high-plasticity clays will exceed expected values due to environmental factors and this change in moisture content should be accounted for in the design of embankments (14).

The study of moisture-induced collapse of soils has been under investigation for many years. Collapsible soils are produced when a soil is deposited without sufficient moisture and/or overburden to produce consolidation and results in a soil with a loose “honeycomb” structure (3). The soil remains stable until a change in moisture content leads to a rapid rearrangement of particles when the soil collapses.

Initially it was believed that only naturally occurring soils were susceptible to collapse. Research has shown that compacted fill materials are also susceptible to wetting-induced collapse under certain conditions. Lawton concluded that, “nearly all types of compacted soils are subject to collapse under certain conditions” (17). He goes on to point out that many engineers are ignorant about the collapse potential of compacted fills because they believe that compacted soil is in its densest state after compaction. This is a reasonable assumption to make only if the moisture content is expected to remain constant. This has led to research into the area of wetting-induced collapse of compacted fills. Lawton found four criteria necessary for collapse to occur:

i.) An open, partially unstable, partially saturated fabric,

ii.) A high enough total stress that the structure is metastable,

iii.) A bonding or cementing agent that stabilizes the soils in the partially saturated condition, and

iv.) The addition of water to the soil, which causes the bonding or cementing agent to be reduced and the interaggregate or intergranular contacts to fail in shear, resulting in a reduction in total volume of the soil mass.

The principles of compaction wet of optimum moisture content versus compaction dry of optimum moisture content that Lambe presented are now well-known (15). Usually compaction dry of optimum moisture content will result in a stronger, less compressible soil than soil compacted wet of optimum moisture content. This has led some engineers to believe that compaction dry of optimum is preferable when settlement is a major concern as in embankment construction. However, collapse potential increases as the degree of saturation and dry density decreases (22). The competition between desired engineering properties becomes more delicate when collapsible soils are considered. A soil compacted dry of optimum moisture content will have a higher collapse potential while a soil compacted wet of optimum will be more compressible under the self-weight of the embankment.

In conclusion, it is important to understand that all embankments will settle to some degree. Understanding the engineering properties of the materials used on a project and properly designing for them is the key to successful construction projects. Engineers should realize this and incorporate the anticipated settlement of the embankment into the design.

2.3 Summary

Long-term reliable performance of compacted fill requires an understanding of the principles behind basic compaction as well as an awareness of new principles being discovered. Modern compaction equipment will produce an acceptable finished product if adequate specifications are adhered to during construction. Excessive settlement of embankments occurs from many different sources. Foundation material not properly prepared for the load, environmental changes, and wetting-induced collapses are areas where research is currently focusing in order find ways to reduce settlement.

Chapter 3

Testing Plan and Procedures

This chapter contains a discussion of the testing program used to evaluate the relative compaction and settlement of eight earth embankments selected by KDOT. Tests were performed on both undisturbed and disturbed samples to evaluate the basic material properties of the soil and to investigate the quality of the construction of the embankment. Testing focused on the applicability of one-dimensional consolidation testing of compacted earth fill materials and the relative compaction of each sample for specification compliance. A review of the compaction specifications of other states was conducted via phone interview.

Table 3.1 lists the short hand site name used for referencing throughout the paper and background information for each site.

TABLE 3.1 Site List

Site Name	Location	Construction Date	Sample Collection Date	Performance
Brickyard	North Topeka, Kansas	2000	9/26/01	1
I-70/K-30	Highway Intersection	2000	9/28/01	10
I-70 East of 350	East of Exit 350	2000	10/3/01	10
I-35 & 151st	Olathe, Kansas	1994	11/20/01	4
US-40 & Oakland	Topeka, Kansas	2000	11/22/01	1
US-59 & K-10	Lawrence, Kansas	1996	12/1/01	1
I-35 & Antioch	Kansas City, Kansas	1998	12/20/01	1
US-24 & Hog Creek	Basehor, Kansas	1997	1/2/01 & 1/30/01	7

3.1 Materials Used

For this study eight embankments were selected with consultation from KDOT. At each site KDOT personnel drilled two boreholes with a hollow stem auger. Shelby tube samples were then collected at regular intervals. Soil that traveled up the auger flights during drilling was collected for testing purposes. Cuttings of each different type of soil that was observed were collected for testing.

A total of 20 different soils were encountered at the eight embankments studied. Table 3.2 shows number and types of tests performed on the disturbed samples from each site. The number of tests performed on the undisturbed samples is presented in Table 3.3.

TABLE 3.2 Number and Type of Test Performed on Disturbed Samples

Site Name	Gradation	Atterberg Limits	Specific Gravity	Compaction Tests	Unconfined Compression of Compaction Samples	Collapse Tests	Dynamic Cone Penetrometer
Brickyard	2	2	2	2	8	-	4
I-70 East of 350	2	2	2	2	8	-	2
I-70 & K-30	4	4	4	4	16	-	2
I-35 & 151st	2	2	2	2	8	-	2
US-40 & Oakland	2	2	2	2	8	6	2
US-59 & K-10	4	4	4	4	16	-	2
I-35 & Antioch	2	2	2	2	8	4	-
US-24 & Hog Creek	2	2	2	2	8	-	-

TABLE 3.3 Number and Type of Test Performed on Undisturbed Samples

Site Name	Unit Weight	Consolidation	Unconfined Compression
Brickyard	6	4	4
I-70 East of 350	10	8	6
I-70 & K-30	7	6	2
I-35 & 151st	8	8	8
US-40 & Oakland	8	8	8
US-59 & K-10	8	8	7
I-35 & Antioch	8	8	4
US-24 & Hog Creek	8	8	6

3.2 Disturbed Samples

3.2.1 Sample Collection

Five-gallon buckets of loose soil were collected at each site for testing. The material that traveled up the auger flights was carefully observed for major changes in soil type. Each different type of material that was encountered in each hole was collected separately and transported to University of Kansas' (KU) Geotechnical Laboratory. Once in the laboratory the soil was allowed to air dry. Once sufficiently dried, each soil was mechanically ground over a 9.5-mm sieve. After grinding, the soil was stored in covered plastic containers until needed for testing.

3.2.2 Grain Size Analysis

The distribution of particles for each type of soil was determined according to ASTM D-422, Standard Test Method for Particle-Size Analysis of Soils. This test method incorporates a sieve analysis of material retained on the No. 10 sieve and a hydrometer analysis of the material passing the No. 10 sieve. For all the tests performed a No. 40 sieve was substituted for the No. 10. This simplified the sample preparation procedure because the material not needed for the hydrometer analysis could be used for Atterberg Limits determination.

3.2.3 Atterberg Limits

The liquid limit, plastic limit, and plasticity index for each type of soil tested was determined according to ASTM D-4318, Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils. The standard three-point method for determining the liquid limit was used for all tests.

3.2.4 Specific Gravity

The specific gravity of each type of soil was determined according to ASTM D-854, Standard Test Method for Specific Gravity of Soils. The tests were performed in pairs of samples and tested according to Test Method B- Procedure for Moist Specimens. All the specific gravity values were within the accepted range for clay soils of 2.44 to 2.92 (32).

A few of the soils encountered in this project were near the upper limit of the accepted range for soils typically found in Kansas. The precision and bias of each pair of tests were investigated and all are within the ASTM accepted range of 0.06 for single operator precision expected for cohesive soils. A second operator performed both the moist and dry procedures described by ASTM on one soil type to validate the original testing. The results were well within the ASTM precision and bias standards for both single operator and multi-operator testing.

3.2.5 Moisture-Density Relationships

Compaction curves were developed for each soil according to ASTM D-698, Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort. Each soil was sieved over a 9.5-mm sieve and compacted in a 101.6-mm diameter mold as described in Procedure B of the ASTM. The moisture content of the excess material was taken as verification of the moisture content obtained after strength testing. Each sample was immediately tested according ASTM D-2166 and the moisture content obtained in this procedure was used for generation of a compaction curve.

3.2.6 Unconfined Compressive Strength

Each compaction sample was tested for strength according to ASTM D-2166, Standard Test Method for Unconfined Compressive Strength of Cohesive Soils. A moisture content-strength curve was developed for each material. The samples prepared for these tests do not meet the ASTM requirement of a H:D ratio between 2 and 2.5. These samples have a H:D ratio of 1.15, therefore the strength value reported will be higher than samples that meet the H:D ratio required by ASTM. Moisture contents were obtained from the failure plane of each sample at the end of the compression testing and used for both moisture-density and moisture-strength curves.

3.2.7 Collapse Testing

Collapse tests were performed according to ASTM D-5333, Standard Test Method for Measurement of Collapse Potential of Soils. The two sites selected for analysis of collapse potential were US-40 & Oakland and I-35 & Antioch. Two Proctor samples were prepared so that a collapse sample could be trimmed from them. Both Proctor samples were compacted at 95 percent maximum dry density, one dry of optimum moisture content and the other wet of optimum moisture content. Two collapse samples were trimmed from each Proctor sample and tested for collapse potential.

3.3 Undisturbed Samples

<u>3.3.1 Sample Collection</u>

A CME 45 drill rig and crew was supplied by KDOT for all subsurface investigations during this project. Once drilling began a depth for sample retrieval was given to the crew. Typically three to four shelby tube samples were collected at approximately 1-meter intervals. A shelby tube was prepared and hydraulically pushed into the hole. The shelby tube was left in the hole under load for approximately 10 minutes to aid in the retrieval process. The sample was then removed from the hole and logged. Information about the sample was collected and the sample was sealed and carefully transported back to KDOT's Materials and Research Center. Information collected included the following: length recovered, sample condition, and pocket penetrometer strength.

The shelby tube samples were stored in KDOT's moisture room pending extrusion by KDOT personnel. The extruded samples were then wrapped in plastic wrap and aluminum foil and transported to the KU Geotechnical Laboratory for further testing.

<u>3.3.2 Dry Density Determination</u>

The dry density of each sample was determined within 24 hours of arrival at KU's Geotechnical Laboratory. Each sample was carefully unwrapped and the ends were squared off to facilitate measurement. The moisture content of the trimmed material from the top and bottom of the sample was performed according to ASTM D-2216. The length, diameter and mass of each sample were determined in order to calculate the *in situ* density. From this information the dry density of the sample was calculated.

3.3.3 Storage

After the dry density was determined each sample was wrapped in plastic wrap and aluminum foil and stored in a moisture room in plastic covered containers until needed for testing.

3.3.4 Moisture Content

All moisture contents for this project were determined according to ASTM D-2216, Standard Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.

3.3.5 Consolidation

Consolidation tests were performed according to ASTM D-2435, Standard Test Method for One-Dimensional Consolidation Properties of Soils. Approximately one-half of the tests were performed according to Test Method B using a 12-hour load cycle. Two time-rate of consolidation tests were performed manually on each sample tested, one in the recompression zone and one in the virgin compression zone of the sample. Time to primary consolidation was easily exceeded in all of the tests performed by this procedure.

The remainder of the consolidation tests were performed according to ASTM D-2435 Test Method B. Loadtrac II equipment and software was setup to perform the test according to ASTM standards. The Loadtrac II software automatically calculates time to 100 percent consolidation for each load step and applies the next load after an additional one-hour offset.

3.3.6 Unconfined Compressive Strength

Unconfined compressive strength tests were performed according to ASTM D-2166, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil. The standard requires that samples have a height to diameter ratio between 2 and 2.5. Some of the samples did not have intact segments of this length and therefore no strength value was found. All samples were loaded at a strain rate of 1 percent/minute using Loadtrac II equipment and software.

3.4 Field Testing

Field-testing was performed during sample collection to gather quantitative information about the quality of compaction at each site.

3.4.1 Dynamic Cone Penetrometer

The DCP test was used to check for uniform compaction in the top 1meter of the embankment. The test was performed according to United States Army Corps of Engineers (USACE) standards using a 7.9 kg mass dropped a distance of 585 mm. The data obtained in the field was converted to California Bearing Ratio (CBR) as specified by USACE literature. The correlation:

$$\log CBR = 2.46 - 1.12*(\log(DCP)) \quad \text{(Equation 3.1)}$$

was used where DCP is in units of mm/blow (31).

3.4.2 Pocket Penetrometer

A pocket penetrometer, supplied by KDOT, was used to determine a compressive strength value of each shelby tube sample. An average of three tests was recorded as q_p in tons per square foot for the sample.

3.5 Compaction Survey

A short compaction specification survey was conducted in order to investigate if other states were experiencing any trouble with their compaction specifications. All 50 states were contacted by telephone, with 32 states, including Kansas, responded. The specific questions asked during the phone survey are as follows:

1) Do you specify density and moisture control for: fills and subgrades?
    a.) If so, what are your current unit weight and moisture content specifications (i.e. 95 percent Standard Proctor, optimum ± 3 percent) for compaction of fills and subgrades?
2) What verification methods do you use to check for adequate compaction of fills and subgrades?
3) What is the maximum lift thickness permitted for fills and subgrades?
4) What are the restrictions on the inclusion of rock within fills in your specifications?
5) Have you/Are you experiencing construction or performance problems with your compaction specifications or verification procedures? If so what are they?

3.6 Instructional Video

The feasibility of making a video to aid in the instruction of visual compaction recognition was investigated. It was confirmed in the field that making such a video would be possible. ASCE does not currently list a field compaction video for sale on its web site.

Chapter 4

Results

This chapter contains the results of all the tests performed for this project. Tests were performed in KU's Geotechnical Lab on disturbed soil samples and undisturbed samples. Field tests were also performed on site during collection of samples. Sample collection consisted of drilling two holes and pushing shelby tubes at regular intervals. Loose soil that traveled to the surface during the drilling process was collected in order to perform routine tests for general classification of soil type. These tests included: specific gravity, grain size distribution, Atterberg Limits, compaction testing, strength testing, and collapse testing.

Shelby tube samples were collected in each hole at regular intervals. The first sample was obtained near the surface of the embankment and the last sample was obtained from a depth of approximately 3 meters or the bottom of the fill. Samples were numbered according to which hole they were obtained and in what order the sample was retrieved. For example, sample 2-3 came from Hole #2 and was the third sample retrieved from that hole. The dry density of each sample was determined within 24 hours of extraction from the shelby tube. Unconfined compressive strength testing was performed on samples with a continuous segment that had a height to diameter ratio of at least 2:1 per ASTM specifications. Consolidation tests were also performed on all samples that could be trimmed in accordance with ASTM specifications.

DCP and pocket cone penetrometer testing was also conducted at each site.

A survey of compaction specifications at other state DOT's was conducted with over thirty states responding.

4.1 Disturbed Testing Results

Disturbed soil samples of each different soil type encountered during drilling were collected from the auger flights. When a different soil type was encountered it was collected in a separate bucket and an approximate depth of the material change was noted on the boring logs. This loose soil was transported to the soils laboratory at KU in five-gallon buckets. Once in the lab the soil was allowed to air dry and was then mechanically ground over a 9.5-mm sieve. The soils were then stored in covered plastic containers until needed for testing. The tests performed with these soils were: specific gravity, grain size distribution, Atterberg Limits, compaction, and unconfined compression. The disturbed testing results for all eight sites are presented in Tables 4.1a and 4.1b.

TABLE 4.1a Disturbed Sample Testing Results

Soil Property	Brickyard		I-70 East of 350		I-70/K-30				I-35 & 151st	
					Upper Soil		Lower Soil			
G_S	Hole #1	Hole #2	Hole #1	Hole #2	Hole #1	Hole #2	Hole #1	Hole #2	Hole #1	Hole #2
	2.77	2.74	2.73	2.69	2.81	2.80	2.81	2.78	2.79	2.76
% Gravel (>4.75 mm)	0	0	1	6	8	6	18	9	2	2
% Sand	18	24	14	8	16	16	26	27	10	11
% Fines (<0.075 mm)	82	76	85	86	76	78	56	64	88	87
LL	33	31	36	36	42	39	35	32	57	49
PL	20	20	20	21	20	21	18	18	19	19
PI	13	11	16	15	22	18	17	14	38	30
USCS	CL	CL	CL	CL	CL	CL	CL	CL	CH	CL
AASHTO	A-6	A-6	A-6	A-6	A-5	A-6	A-6	A-6	A-7-6	A-7-6
Maximum Dry Density (kg/m^3)	1770	1760	1670	1700	1770	1780	1880	1970	1630	1670
% OMC	16.0	16.8	17.1	16.8	17.9	17.4	16.5	14.1	20.4	20.6
Strength (kPa)	480	425	440	400	460	480	470	480	410	420

TABLE 4.1b Disturbed Sample Testing Results

Soil Property	US 40 & Oakland		US 59 & K-10				I-35 & Antioch		US-24 & Hog Creek	
			Upper Soil		Lower Soil					
	Hole #1	Hole #2	Hole #1	Hole #1	Hole #2	Hole #2	Hole #1	Hole #2	Hole #1	Hole #2
G_S	2.76	2.72	2.77	2.78	2.74	2.73	2.76	2.71	2.81	2.76
% Gravel (>4.75 mm)	1	20	0	0	0	0	5	3	2	1
% Sand	15	16	8	8	8	6	32	35	44	43
% Fines (<0.075 mm)	84	64	92	92	92	94	63	62	54	56
LL	45	45	41	42	63	57	28	29	41	34
PL	23	22	21	21	21	20	20	21	18	17
PI	22	23	20	21	42	37	8	8	23	17
USCS	CL	CL	CL	CL	CL	CH	CH	CL	CL	CL
AASHTO	A-7-6	A-7-6	A-7-6	A-7-6	A-7-6	A-7-6	A-6	A-6	A-7-6	A-6
Maximum Dry Density (kg/m^3)	1680	1700	1720	1720	1590	1600	1830	1790	1860	1800
% OMC	20.8	20.1	18.1	18.8	22.7	21.0	15.9	16.4	14.2	14.5
Strength (kPa)	280	320	420	390	410	400	400	360	400	420

Twenty different soils were encountered at the eight sites evaluated in this project. The specific gravity of all the soils ranged from a low of 2.69 to a high of 2.81.

Grain size distributions were determined for all soils. The sand content of the soils was low; as expected for soils found in eastern Kansas. The highest sand content was 20.0 percent and the low was 0.0 percent. All but two of the soils had less than 10.0 percent sand sized particles.

The liquid limit and plastic limit of all the soils were determined for classification of the soils. The highest liquid limit encountered was 63 and the lowest was 28. The plastic limits ranged from 23 to 8. Seventeen of the soils classified as low plasticity clays (CL) according to the Unified Soil Classification System (USCS). The remaining three were considered high plasticity clays (CH) in the USCS classification. When classified by the AASHTO system, 10 soils classified as A-6, one as A-5, and nine as A-7-6. Collapse testing from the two sites selected showed no collapse potential for these soils.

Compaction curves were developed for all soils and are presented in Figures 4.1 to 4.10. Maximum dry densities ranged from a high of 1970 kg/m^3 for a soil used in the lower half of the I-70/K-30 embankment to a low of 1680 kg/m^3 used in the US-40/Oakland embankment. Strength curves were developed using the compaction samples for unconfined compression specimens. The peak strength values ranged from 280 kPa to 480 kPa for these samples.

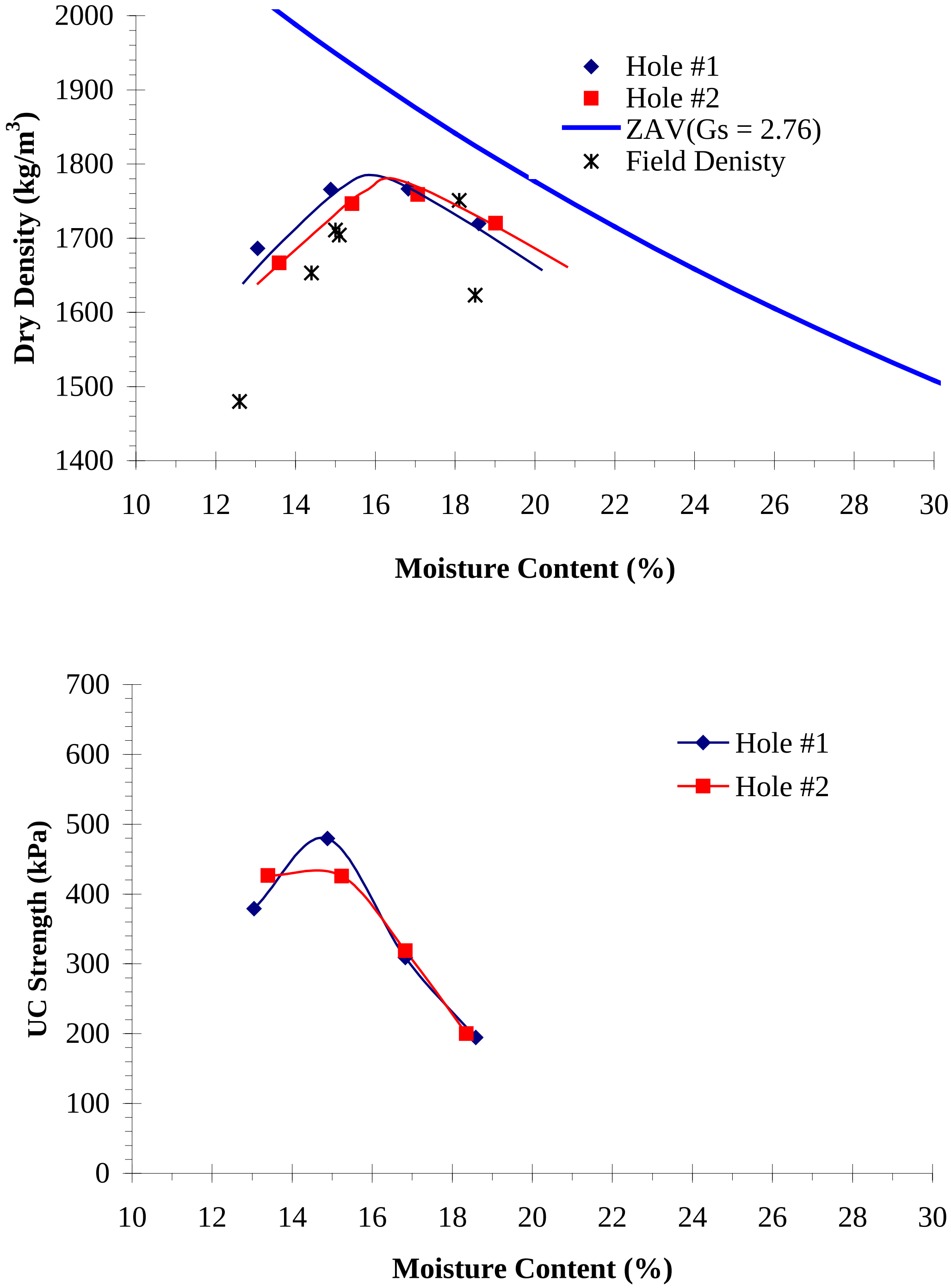
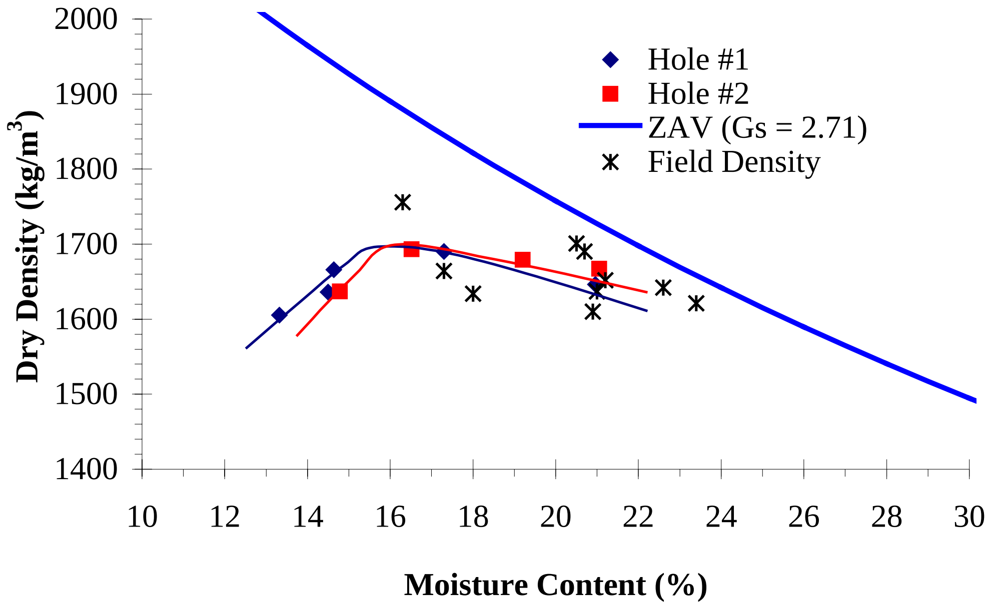


FIGURE 4.1 Brickyard Moisture-Density and Strength Relationships



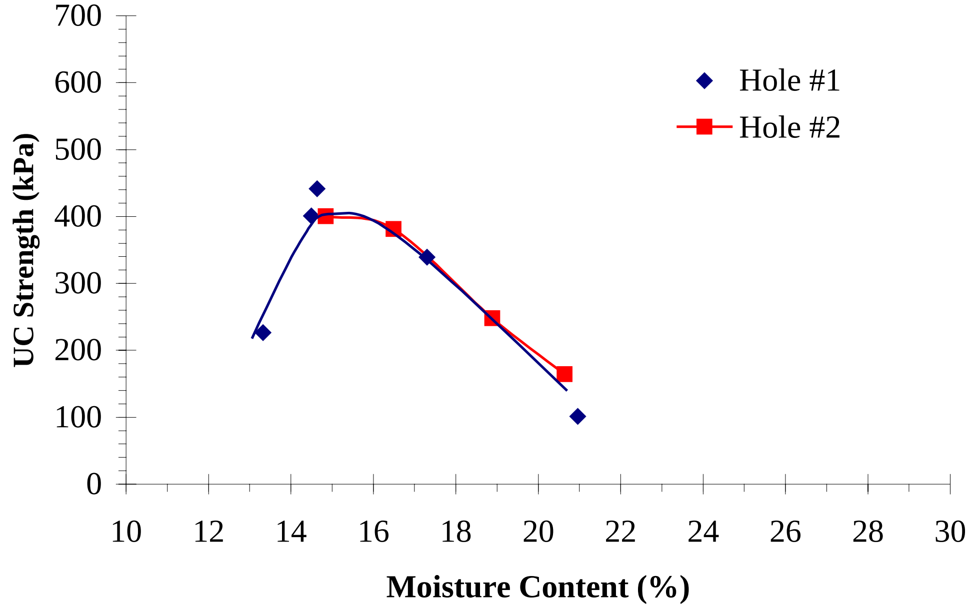


FIGURE 4.2 I-70 East of 350 Moisture-Density and Strength Relationships

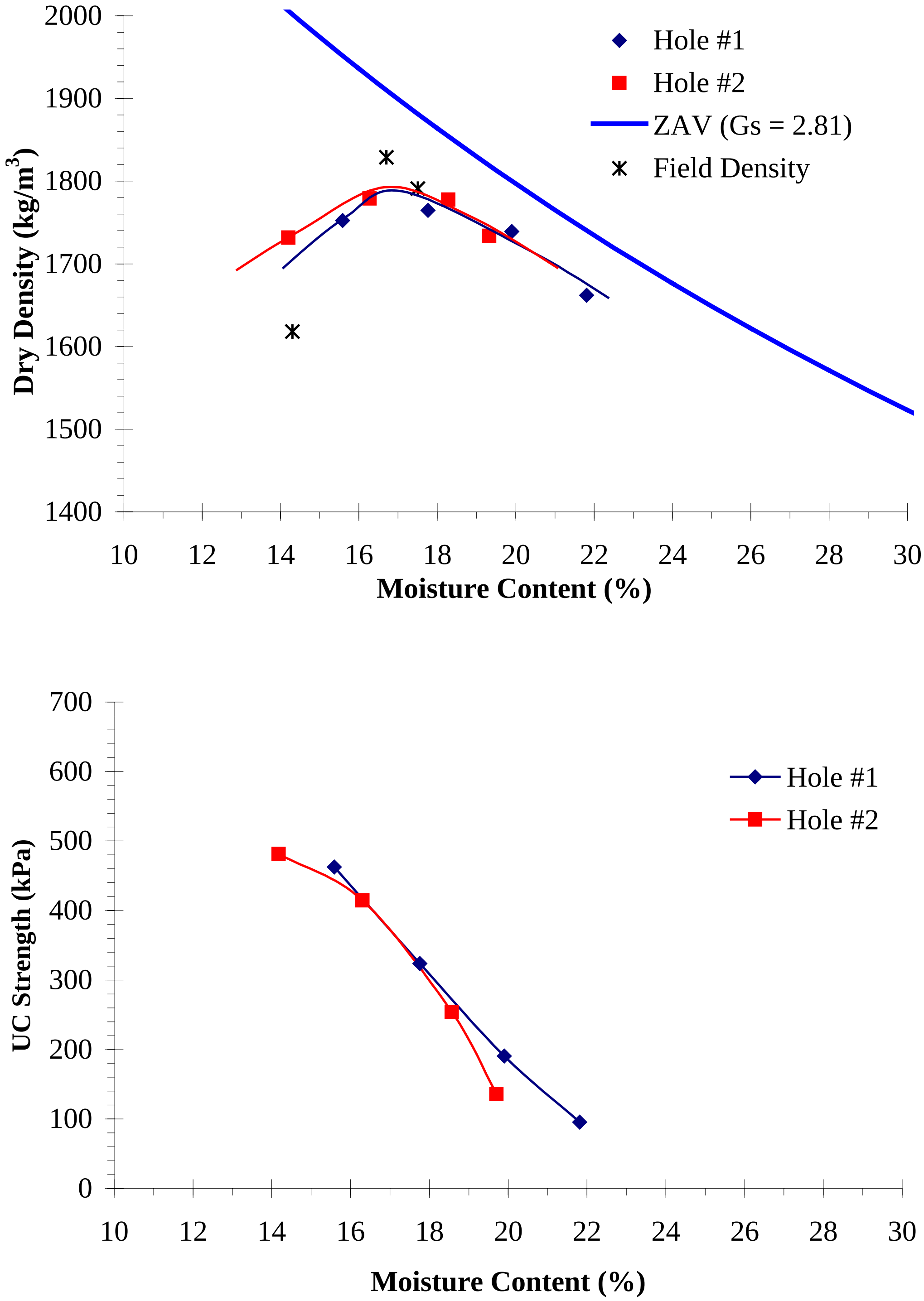
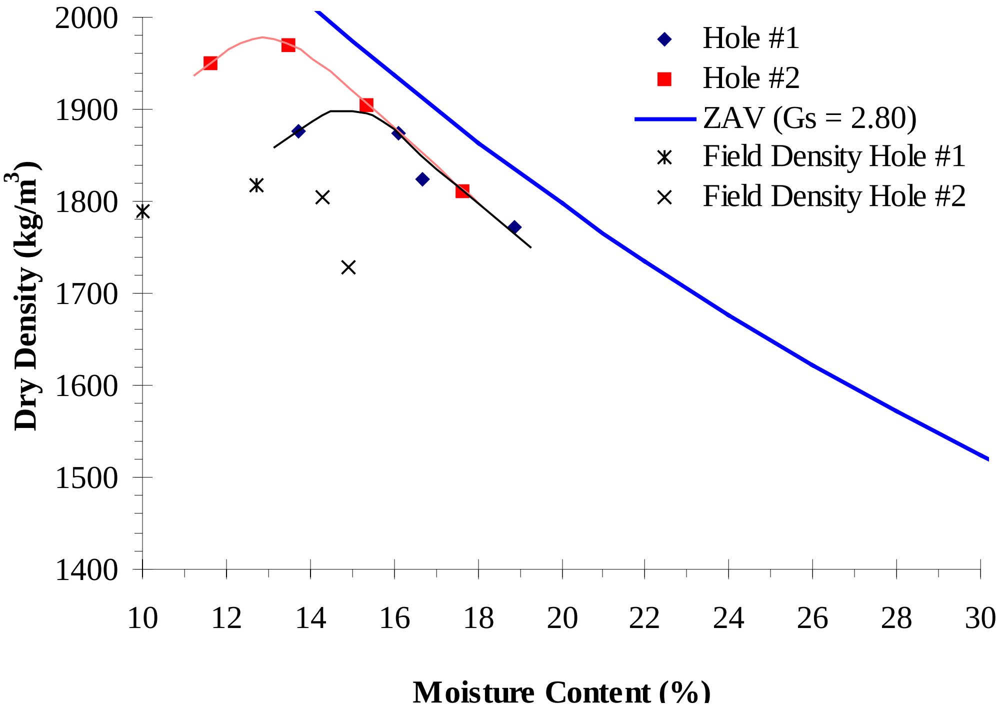
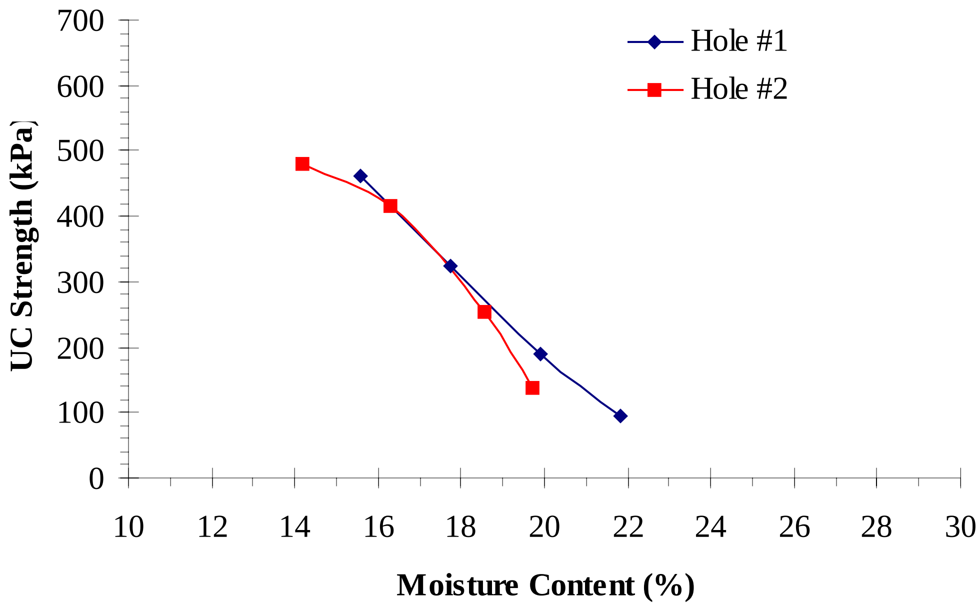
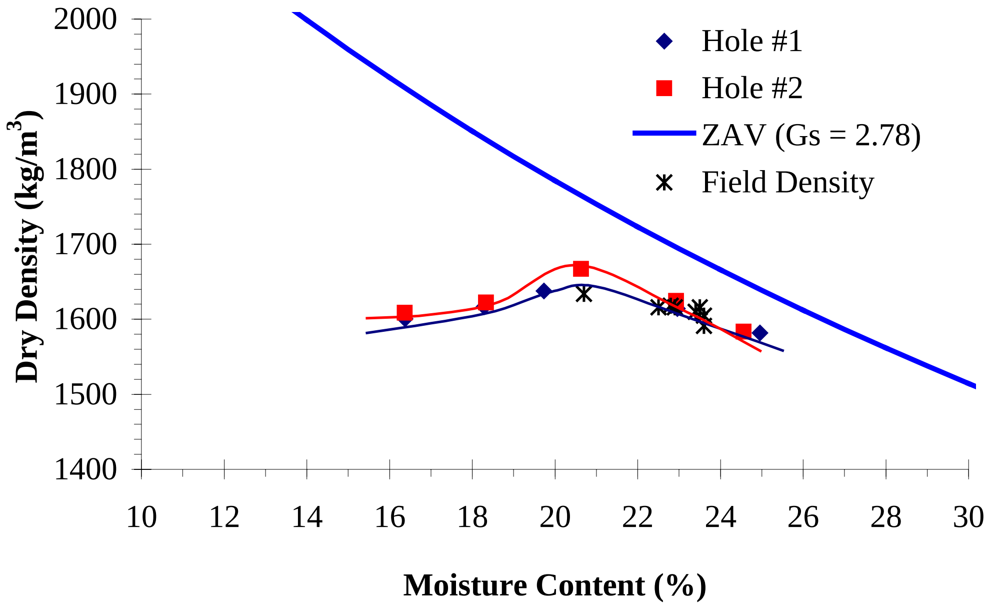


FIGURE 4.3 I-70 & K-30 Upper Soil Moisture-Density and Strength Relationships





<u>FIGURE 4.4 I-70 & K-30 Lower Soil Moisture-Density and Strength Relationships</u>



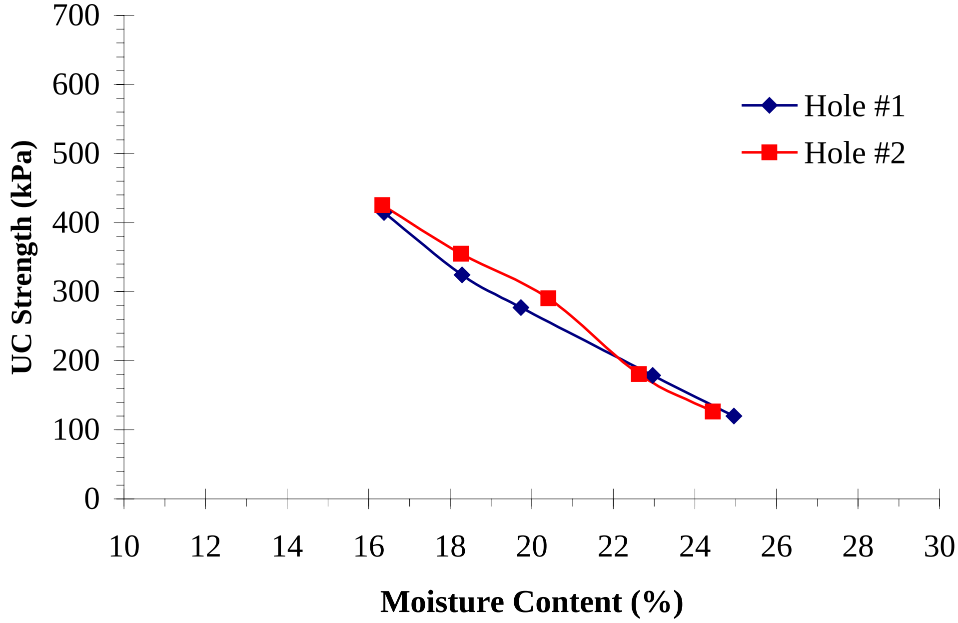
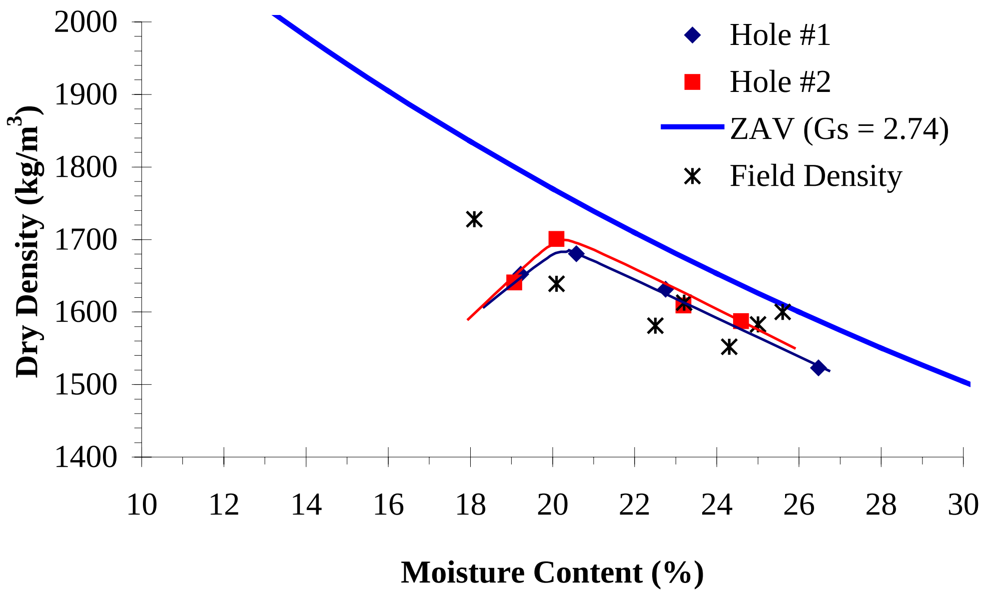


FIGURE 4.5 I-35 & 151st Moisture-Density and Strength Relationships



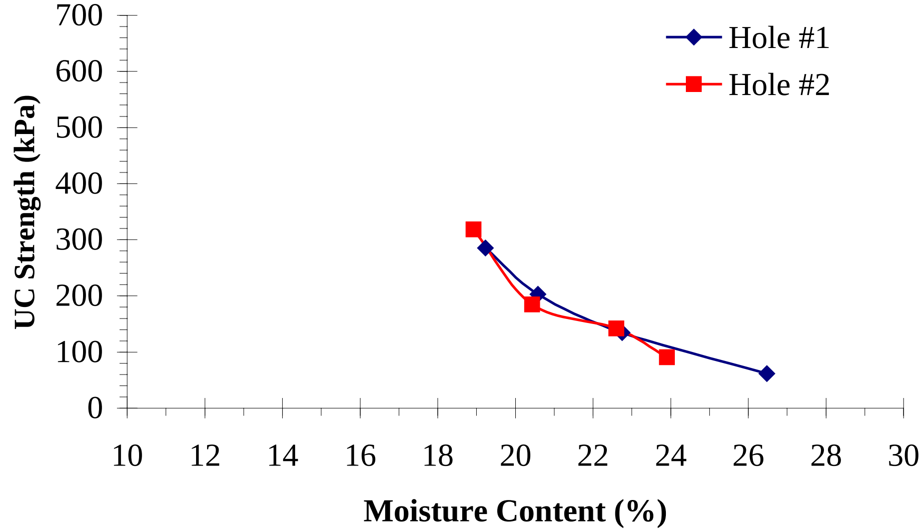


FIGURE 4.6 US-40 & Oakland Moisture-Density and Strength Relationships

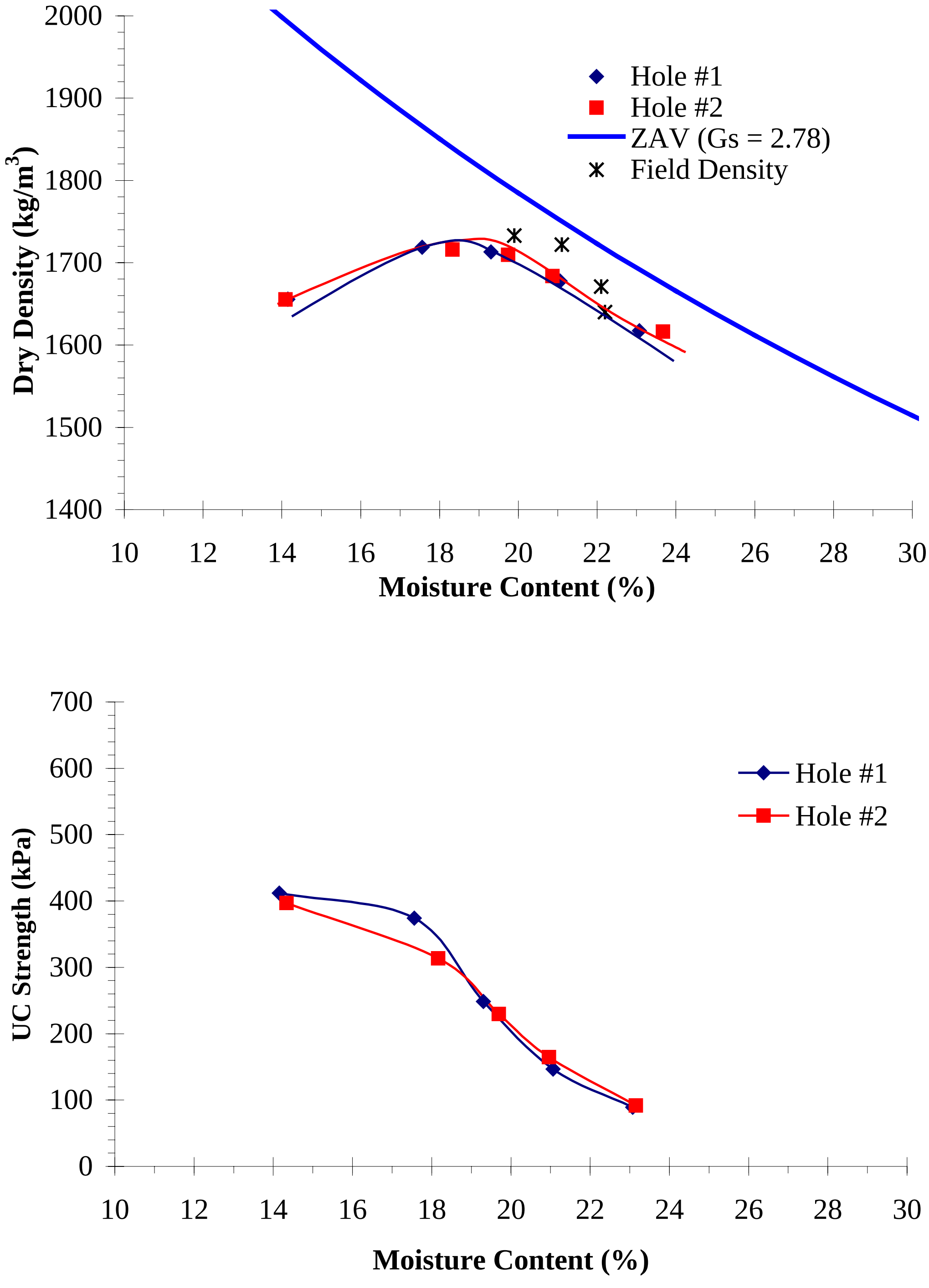
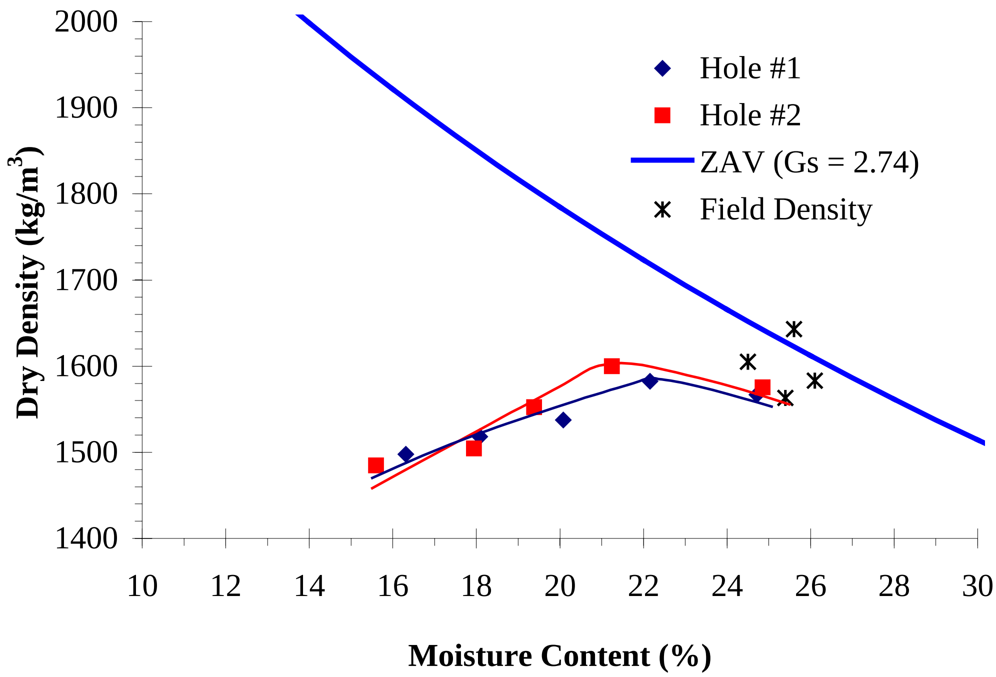


FIGURE 4.7 US-59 & K-10 Upper Soil Moisture-Density and Strength Relationships



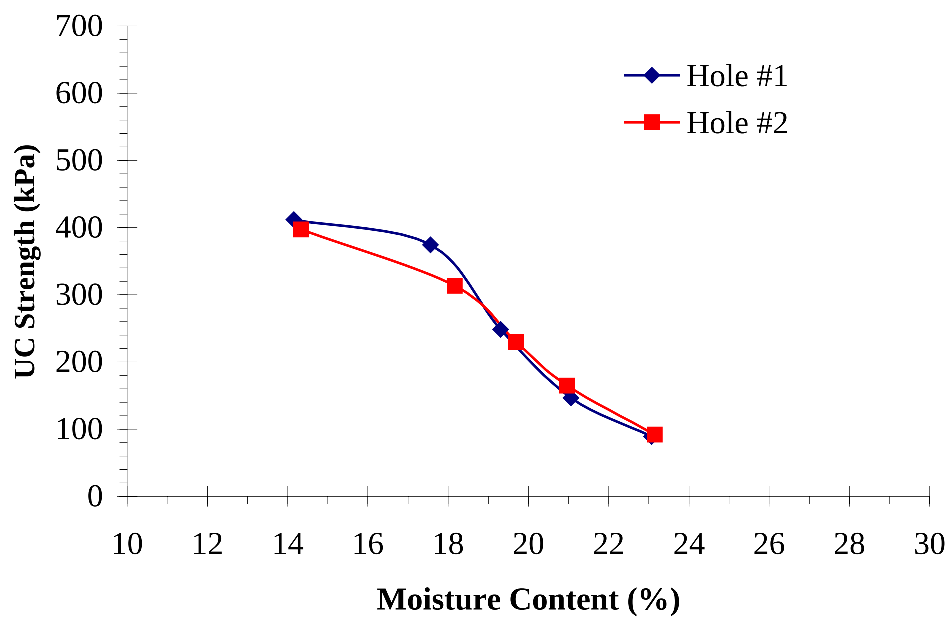
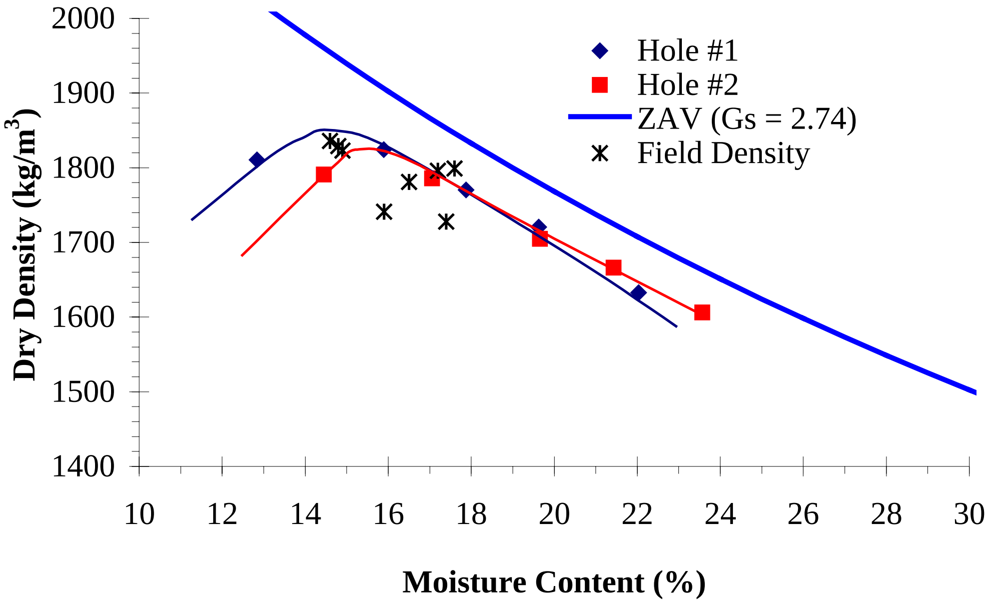


FIGURE 4.8 US-59 & K-10 Lower Soil Moisture-Density and Strength Relationships



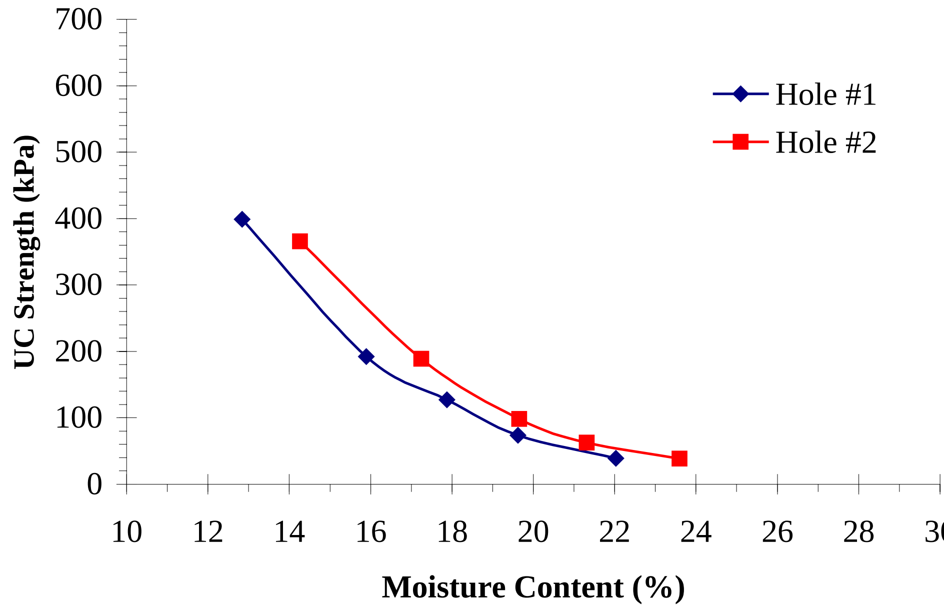


FIGURE 4.9 I-35 & Antioch Moisture-Density and Strength Relationships

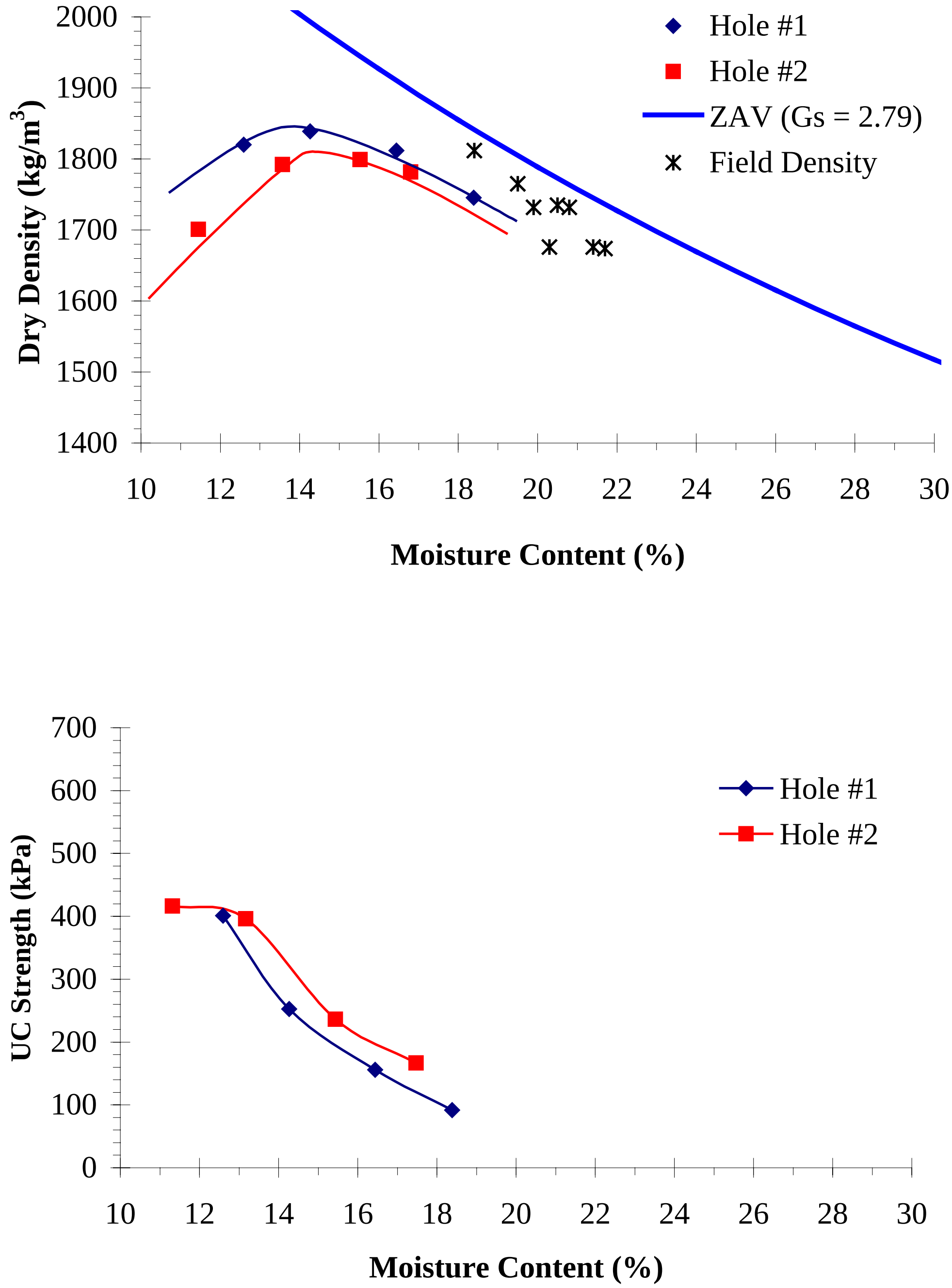


FIGURE 4.10 US-24 & Hog Creek Moisture-Density and Strength Relationships

4.2 Undisturbed Testing Results

4.2.1 Brickyard Undisturbed Testing Results

The first site selected by KDOT for sample collection was located in North Topeka at the intersection of 35th Street and Brickyard. The site was under construction when the samples were collected on September 26, 2000. No settlement of the embankment was reported. The sample collection process was slightly modified at the Brickyard site from the other seven sites. Hole #1 was south and Hole #2 was north of the right-of-way for 35th Street and approximately 25 meters from each other.

The depth of fill was estimated to be approximately 2.5 meters above a box culvert. Three samples were obtained from each hole at one-meter intervals. The *in situ* moisture content ranged from 12.6 to 18.5 percent. The dry density of each specimen was determined and the minimum was 1480 kg/m^3 and the maximum was 1750 kg/m^3. All but sample 1-3 easily met a 90 percent relative compaction level for standard effort as defined by KT-12 (AASHTO T 99). Unconfined compressive strengths ranged from 85 kPa to 245 kPa. One-dimensional consolidation testing yielded an effective preconsolidation pressure range of 130 to 450 kPa. A complete summary of the results of the undisturbed testing at Brickyard is presented in Table 4.2.

TABLE 4.2 Brickyard Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.4	18.1	1750	99	115	-	-	-
1-2	1.4	15.1	1700	96	240	220	0.14	0.03
1-3	2.4	12.6	1480	84	-	-	-	-
2-1	0.5	18.5	1620	92	85	130	0.18	0.01
2-2	1.5	15.0	1710	97	245	450	0.17	0.01
2-3	2.4	14.4	1650	94	-	250	0.23	0.04

4.2.2 I-70 East of Exit 350 Undisturbed Testing Results

The second site was located on the north side of Interstate 70 about 1.2 kilometers east of Exit 350. The embankment selected had been rebuilt earlier in the year because of excessive settlement problems. Samples were collected on September 28, 2000. New fill was placed on top of pre-existing fill in order to try to fix the settlement problem. Holes 1 and 2 were drilled within 2 meters of each other. This sample collection procedure was repeated for the remaining sites.

The remediation of the initial settlement problem called for approximately 3 meters of new fill material. Five samples per hole were collected at regular intervals. The *in situ* moisture content of the samples collected ranged from 16.3 percent to 23.4 percent. The range of dry densities obtained at this site was 1610 kg/m^3 to 1760 kg/m^3. The relative compaction for all 10 samples was greater than 95 percent of standard effort. The unconfined compressive strength of the soil was between 190 kPa and 350 kPa. The effective preconsolidation pressure range was between 280 kPa and 710 kPa. The results of the undisturbed testing are presented in Table 4.3.

TABLE 4.3 I-70 East of Exit 350 Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.5	23.4	1620	96	190	420	0.12	0.04
1-2	1.2	20.5	1700	101	-	710	0.21	0.03
1-3	2.3	22.6	1640	97	240	400	0.20	0.03
1-4	3.1	21.2	1650	98	300	280	0.17	0.03
1-5	3.7	17.3	1660	98	-	-	-	-
2-1	0.4	18.0	1630	96	-	550	0.16	0.04
2-2	1.5	21.0	1640	97	210	410	0.22	0.05
2-3	2.3	20.7	1690	100	315	310	0.17	0.03
2-4	3.1	20.9	1610	95	350	320	0.16	0.03
2-5	3.7	16.3	1760	104	-	-	-	-

4.2.3 Interstate 70 & Kansas Highway 30 Undisturbed Testing Results

The third site was located at the intersection of Interstate 70 (I-70) and Kansas Highway 30 (K-30). A bridge over I-70 was under construction and the embankment had been built earlier in the year. Samples were collected on October 3, 2000. The depth of fill was approximately 3 meters. The fill was composed of two different materials; an upper material composed of gray shale and a lower material composed of tan shale.

Four samples were recovered from Hole 1 and three were recovered from Hole 2. The recovery of any sample longer than 25 cm not including 1-1 and 2-1 was very difficult to obtain due to the firmness of the embankment. The material was too hard for the shelby tube to penetrate for the recovery of sample 2-2. Most of the samples were shorter than desired.

The *in situ* moisture content for the samples ranged from 9.7 percent to 17.5 percent. The dry densities were higher at this site and ranged from 1620 kg/m^3 to 1830 kg/m^3. All but sample 2-4 had a relative compaction greater than 90 percent standard effort. Only samples 1-1 and 2-1 had continuous segments long enough for unconfined compression tests according to ASTM specifications. The effective preconsolidation pressure ranged from 105 kPa to 790 kPa. Testing results are presented in Table 4.4.

TABLE 4.4 I-70 & K-30 Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.6	17.5	1790	101	340	790	0.18	0.03
1-2	1.2	14.3	1620	92	-	-	-	-
1-3	2.1	12.7	1820	97	-	190	0.13	0.03
1-4	2.8	9.7	1790	95	-	-	0.14	0.04
2-1	0.4	16.7	1830	103	270	350	0.13	0.03
2-2	-	-	-	-	-	-	-	-
2-3	2.1	14.3	1800	92	-	200	0.10	0.04
2-4	3.0	14.9	1730	88	-	105	0.17	0.04

4.2.4 Interstate 35 & 151st Street Undisturbed Testing Results

The fourth site selected for investigation was located in Olathe, Kansas, at the intersection of Interstate 35 (I-35) and 151st Street. The embankment at this location was constructed in 1994 and had experienced some mild settlement. Samples were collected on November 20, 2000. The embankment fill was approximately 7 meters thick.

Four samples were recovered from each hole to a depth of over 3 meters. Recovery lengths were good and the samples remained intact. The *in situ* moisture contents ranged from 20.7 percent to 23.8 percent. Dry densities were all near 1600 kg/m^3 with high and low values of 1590 and 1620 kg/m^3 respectively. Compaction was good at this site with all samples over the 95 percent relative compaction level for standard effort. Strength values were obtained for all samples. The weakest sample was 1-2 at 110 kPa and the strongest was 1-1 at 290 kPa. The effective preconsolidation pressure ranged from 190 kPa to 600 kPa. The results of the undisturbed testing are presented in Table 4.5.

TABLE 4.5 I-35 & 151st Street Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.5	22.8	1620	100	290	405	0.13	0.05
1-2	1.3	23.4	1610	99	110	510	0.22	0.08
1-3	2.3	23.5	1620	99	170	250	0.16	0.09
1-4	3.2	23.6	1610	99	240	400	0.14	0.06
2-1	0.5	20.7	1630	98	180	190	0.23	0.06
2-2	1.5	22.5	1620	97	270	260	0.21	0.05
2-3	2.4	23.6	1590	95	140	600	0.20	0.05
2-4	3.2	22.9	1620	97	250	550	0.15	0.06

4.2.5 US Highway 40 & Oakland Avenue Undisturbed Testing Results

The fifth site selected by KDOT was constructed in 2000 and had not been opened to traffic yet. No settlement of the embankment had been noticed at the time of sample retrieval. The embankment was located in Topeka, Kansas, at the intersection of US Highway 40 (US-40) and Oakland Avenue. Samples were collected on November 22, 2000. The embankment was at least 8 meters in height and was investigated to 4 meters for this project.

Four samples were recovered from each hole and recovery lengths were good. The *in situ* moisture contents ranged from 18.1 percent to 25.6 percent for the eight samples. Dry densities of the samples ranged from 1550 kg/m^3 to 1730 kg/m^3. Relative compaction of the fill exceeded 90 percent of standard compactive effort for all samples. Unconfined compressive strength was determined for all samples. The strongest sample was 1-3 at 385 kPa and the weakest was 2-4 at

135 kPa. Effective preconsolidation pressures ranged from 200 to 800 kPa. A summary of the results of the undisturbed testing is presented in Table 4.6.

TABLE 4.6 US-40 & Oakland Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.5	18.1	1730	103	245	200	0.10	0.02
1-2	1.6	22.5	1580	94	160	800	0.40	0.02
1-3	3.0	24.3	1550	92	385	550	0.21	0.03
1-4	4.1	25.6	1600	95	155	300	0.21	0.07
2-1	0.7	21.3	1680	99	265	400	0.19	0.03
2-2	1.7	20.1	1640	96	265	470	0.22	0.03
2-3	2.9	23.2	1610	95	375	310	0.22	0.05
2-4	4.2	25.0	1580	93	135	300	0.21	0.06

4.2.6 US Highway 59 & K-10 Undisturbed Testing Results

The embankment over US Highway 59 (US-59) in Lawrence, Kansas, is part of the South Lawrence Trafficway. The embankment had not experienced noticeable settlement or been opened to traffic on the sample collection date of December 1, 2000. The embankment was approximately 5 meters tall and was investigated to a depth of 3 meters for this project.

Four samples were collected in each hole. The fill material changed from light brown clay to dark brown, almost black clay at 1.2 meters. *In situ* moisture contents averaged 22.5 percent and ranged from 19.9 percent to 26.1 percent. The dry densities of the samples ranged from 1560 kg/m^3 to 1730 kg/m^3. Compaction of the fill was good with all samples over the 95 percent relative compaction level for standard compactive effort. Unconfined compressive strength was determined for seven of the eight samples and ranged from 180 kPa to 325 kPa. The effective preconsolidation pressure had a high and low value of 300 kPa and 800 kPa respectively. The results of the undisturbed testing are presented in Table 4.7.

TABLE 4.7 US-59 & K-10 Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.7	19.9	1730	101	190	300	0.13	0.04
1-2	1.5	22.1	1670	97	325	300	0.09	0.02
1-3	2.4	26.1	1580	100	180	305	0.22	0.06
1-4	3.3	24.5	1610	101	190	310	0.19	0.04
2-1	0.7	21.1	1720	100	325	400	0.14	0.02
2-2	1.5	22.2	1640	95	225	390	0.12	0.03
2-3	2.3	25.6	1640	103	-	400	0.26	0.08
2-4	3.2	25.4	1560	98	215	800	0.35	0.06

4.2.7 Interstate 35 & Antioch Undisturbed Testing Results

The seventh site selected for shelby tube investigation was located in Kansas City, Kansas, on the exit ramp for Southbound I-35 at Antioch Street. The samples were collected on December 20, 2000. The embankment was approximately 5 meters tall and was sampled to a depth of 3.5 meters.

Four samples were collected from each hole. The *in situ* moisture contents ranged from 14.8 percent to 17.4 percent. The dry densities ranged from 1740 kg/m^3 to 1840 kg/m^3. All samples were over 95 percent relative compaction for standard compactive effort. The samples were less intact than at previous sites and only four had segments long enough for unconfined compressive strength. Strength values ranged from 115 kPa to 280 kPa. The effective preconsolidation pressures ranged from 240 kPa to 500 kPa. The results of the undisturbed testing are presented in Table 4.8.

TABLE 4.8 I-35 & Antioch Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.9	17.2	1800	98	115	300	0.10	0.01
1-2	1.9	14.9	1820	100	-	390	0.11	0.02
1-3	3.1	14.8	1830	100	-	400	0.08	0.01
1-4	3.7	17.4	1730	95	-	240	0.08	0.01
2-1	0.8	14.6	1840	102	280	500	0.15	0.01
2-2	2.0	17.6	1800	100	235	500	0.09	0.02
2-3	2.4	16.5	1780	99	190	400	0.05	0.01
2-4	3.8	15.9	1740	97	-	-	0.03	0.004

4.2.8 US Highway 24 & Hog Creek Undisturbed Testing Results

The final site selected by KDOT for this project was located approximately 8 km east of Basehor, Kansas, on US Highway 24 (US-24). This embankment was constructed in 1997 and has experienced settlements that are unacceptable to KDOT. Hole 1 and Hole 2 were not drilled on the same day due to scheduling difficulties. Hole 1 was sampled on January 2, 2001, and Hole 2 was sampled on January 30, 2001. The fill was over 4 meters thick and was investigated to a depth of 3.5 meters on both days.

Four samples were collected from each hole. The *in situ* moisture contents ranged between 18.4 percent and 21.7 percent. The dry densities for the eight samples ranged from 1690 kg/m^3 to 1730 kg/m^3. Relative compaction for the site was acceptable with all eight samples at or above the 90 percent compaction level for standard compactive effort. The unconfined compressive strengths for all the samples were low. Strengths ranged from 90 kPa to 220 kPa. Both sample 1-1 and 2-1 were not intact enough to obtain strength information. The effective preconsolidation pressure range was 190 kPa to 500 kPa. The results of the undisturbed testing are presented in Table 4.9.

TABLE 4.9 US-24 & Hog Creek Undisturbed Testing Results

Sample	Depth (m)	Natural Moisture Content (%)	Dry Density (kg/m^3)	Relative Compaction (%)	Strength (kPa)	σ'_p (kPa)	C_c	C_r
1-1	0.6	18.4	1710	92	-	500	0.20	0.04
1-2	1.3	20.3	1680	90	90	300	0.20	0.02
1-3	2.3	19.5	1770	95	180	190	0.19	0.02
1-4	3.5	20.5	1740	94	185	340	0.20	0.02
2-1	0.7	21.7	1670	93	-	320	0.18	0.03
2-2	1.4	21.4	1680	93	140	190	0.21	0.04
2-3	2.3	20.8	1730	96	145	490	0.25	0.04
2-4	3.5	19.9	1730	96	220	300	0.20	0.01

4.3 Field Testing Results

Two types of field test were performed in this project, Dynamic Cone Penetrometer (DCP) and pocket cone penetrometer. The results of the field-testing are presented in Table 4.10a to 4.10d. The average California Bearing Ratio (CBR) was determined for the top meter of the embankment using the correlation recommended by the USACE discussed in Chapter 3.

DCP testing was performed according to USACE recommendations at six of the eight embankments investigated in this project. Winter conditions ruined the surface of the last two embankments for DCP testing at the time of sample collection.

A pocket cone penetrometer was used to estimate the field strength of the shelby tube samples. An average of three measurements was recorded on the boring logs as q_p in tons per square foot. This was converted to kilopascals and reported in Tables 4.10a-4.10d.

TABLE 4.10a Field Testing Results

Brickyard

Sample	q_p (kPa)	Average CBR (%)
1-1	383	16.1
1-2	431	
1-3	431	-
2-1	407	16.7
2-2	407	
2-3	407	-

I-70 East of 350

Sample	q_p (kPa)	Average CBR (%)
1-1	263	7.4
1-2	407	
1-3	263	-
1-4	192	
1-5	431	
2-1	431	7.6
2-2	144	
2-3	144	-
2-4	383	
2-5	431	

TABLE 4.10b Field Testing Results

I-70/K-30

Sample	q_p (kPa)	Average CBR (%)
1-1	431	32.9
1-2	431	
1-3	431	-
1-4	431	
2-1	431	26.0
None	-	
2-3	431	-
2-4	431	

I-35 & 151st

Sample	q_p (kPa)	Average CBR (%)
1-1	407	5.3
1-2	335	
1-3	383	-
1-4	263	
2-1	383	7.7
2-2	383	
2-3	287	-
2-4	383	

TABLE 4.10c Field Testing Results

US-40 & Oakland

Sample	q_p (kPa)	Average CBR (%)
1-1	359	29.6
1-2	383	
1-3	287	-
1-4	144	
2-1	335	12.1
2-2	431	
2-3	383	-
2-4	192	

US-59 & K-10

Sample	q_p (kPa)	Average CBR (%)
1-1	383	6.6
1-2	383	
1-3	287	-
1-4	192	
2-1	431	10.2
2-2	335	
2-3	335	-
2-4	120	

TABLE 4.10d Field Testing Results

I-35 & Antioch

Sample	q_p (kPa)	Average CBR (%)
1-1	431	N/A
1-2	431	
1-3	431	
1-4	431	
2-1	383	N/A
2-2	335	
2-3	287	
2-4	287	

US-24 & Hog Creek

Sample	q_p (kPa)	Average CBR (%)
1-1	383	N/A
1-2	144	
1-3	287	
1-4	144	
2-1	192	N/A
2-2	144	
2-3	144	
2-4	239	

Another way to utilize the DCP data is to plot resistance vs. depth. This is commonly done by calculating penetration per blow and plotting them, as shown in Figures 4.11a and 4.11b. In these figures the closer the line remains to the y-axis the more resistive the embankment was to penetration. Therefore, high numbers on the x-axis represent a low resistance/CBR and low numbers represent a high resistance/CBR.

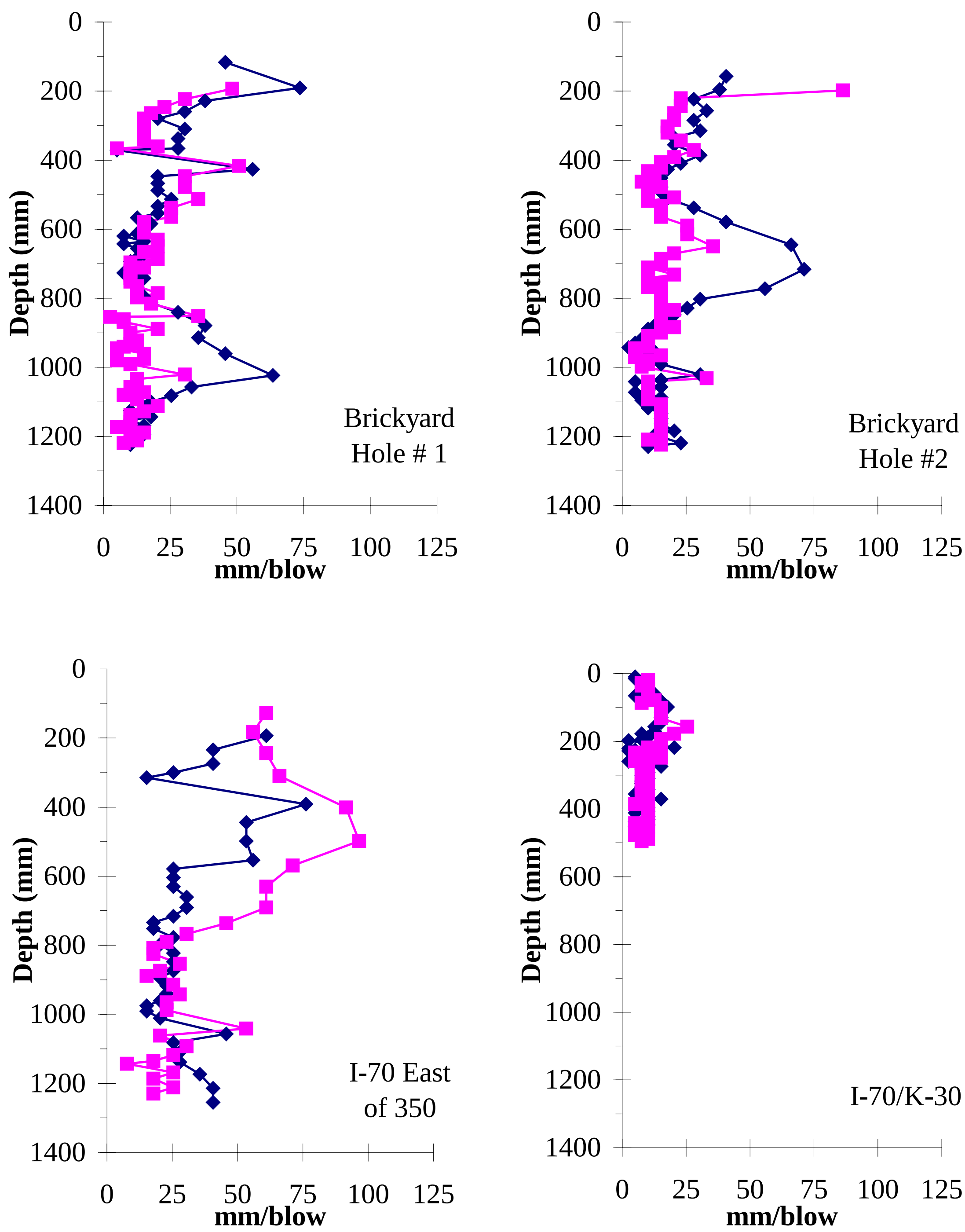


FIGURE 4.11a Penetration Resistance vs. Depth

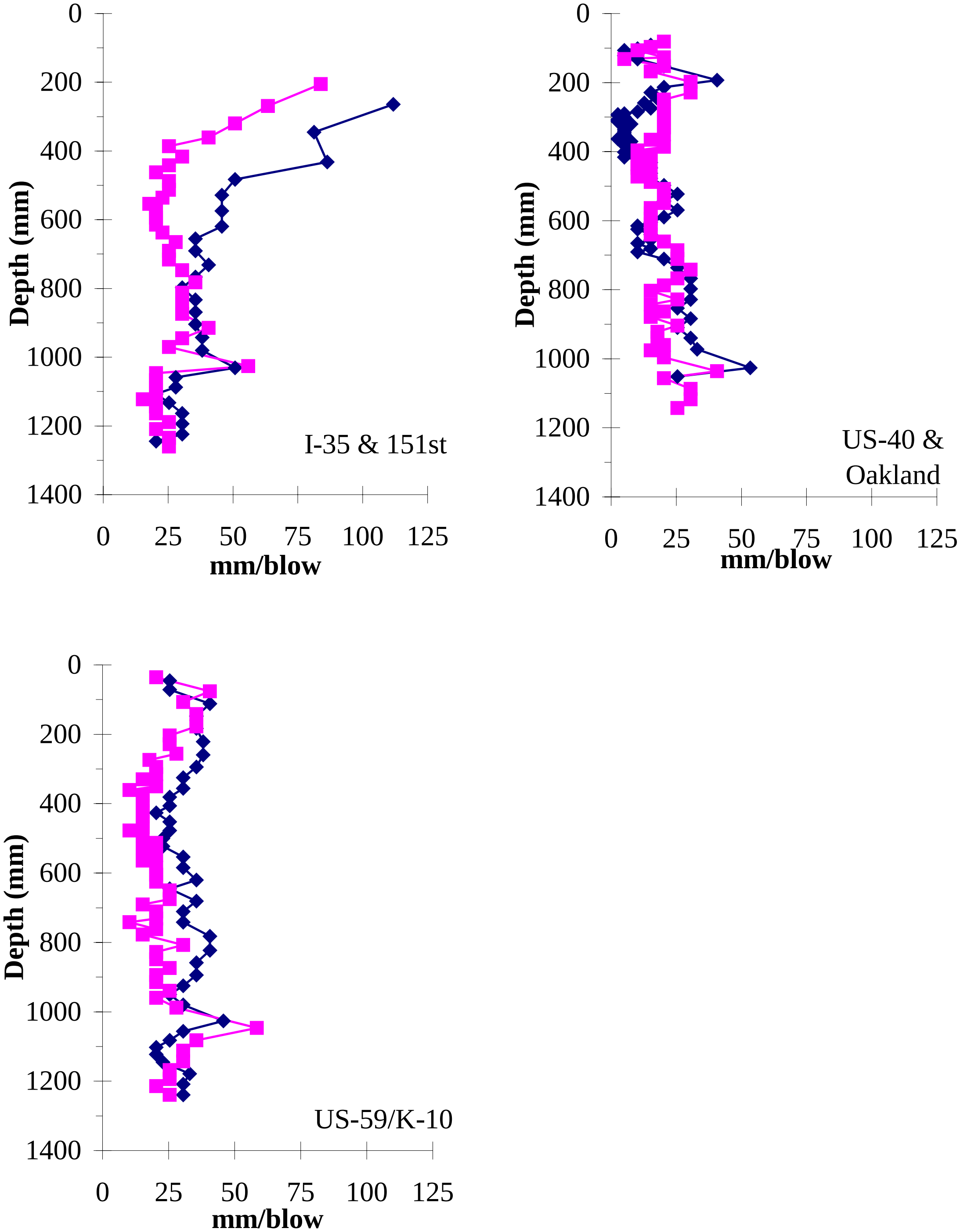


FIGURE 4.11b Penetration Resistance vs. Depth

4.4 Compaction Survey Results

A phone survey was conducted in order to determine the status of compaction specifications around the United States. All 50 states were contacted and 32 states, including Kansas, responded to the questions presented in Chapter 3. The survey included questions about existing specifications and whether those specifications and improper construction techniques could be contributing to embankment failures in their states.

4.4.1 Moisture Control for Compaction Specifications

Most of the states that responded to the survey had some type of moisture control within their compaction specifications. Table 4.11 presents the type of moisture control each state specified.

Many different approaches to moisture content specifications are in use. Five have no specified moisture control and only require that the moisture content be sufficient to provide adequate compaction. Other states permit a wide range around optimum such as ±5 percentage points. Other states permit a narrower range centered on optimum moisture content and five require optimum moisture content only.

Kansas has five different moisture content control levels depending on the project and soil type. However, the most specified moisture control level is MR 90. This moisture control is usually paired with Type B Compaction, the visual compaction requirement specification under review in this project.

TABLE 4.11 Moisture Control Specifications

Moisture Control Level	Number of States
±5 %	2
±3 %	1
±2 %	7
OMC	5
None	5
-4 to +5	1
-4 to +3	1
-2 to +1	1
0 to +3	1
±20% OMC	1
<120% OMC	1
@80% OMC	1
<110% OMC	1
Visual	2
Other	2

4.4.2 Density Control Requirements

Some type of density control was required by all states responding to the survey. The current Kansas specification has five levels of compaction requirements to choose from. Three of the choices; Type AAA, Type AA, and Type A, require a specific relative compaction. The remaining two are less specific about the exact relative compaction level and are both visual compaction specifications. Type B requires visual compaction verification by sheepsfoot roller walkout and Type C is visual compaction verification by the engineer's judgment. In Kansas, the most commonly used compaction specification is Type B. Table 4.12 presents how other states specify density control.

TABLE 4.12 Density Control Specifications

Relative Compaction Level (%)	Modified Compactive Effort (AASHTO T 180)	Standard Compactive Effort (AASHTO T 99)
96	-	2
95	1	19
93	-	1
92	1	-
90	1	2
Other	5	

As Table 4.12 shows, the majority of states are relying on a standard compactive effort specification to control density. No states are specifying relative compaction levels below the 90 percent compaction level. Kansas and Iowa were the only states relying primarily on visual compaction techniques. Colorado requires a minimum level of unit weight regardless of the type of material used. Nevada has implemented the use of the Harvard Miniature Mold for density control and others are using a one-point Proctor method.

4.4.3 Density Verification Methods

As presented in Table 4.13, over 75 percent of the states that responded use the nuclear density gauge to verify density. The two primary reasons that states may not be using the nuclear density gauge are the expense of the equipment and the inconvenience of having a nuclear source.

TABLE 4.13 Density Verification Methods

Primary Verification Method	States Using
Nuclear	25
Sand Cone	2
Balloon	2
Visual	2
1 pt Proctor	1

4.4.4 Lift Thickness

The maximum lift thickness permitted by Kansas is 200 mm (8 inches). As Table 4.14 shows, Kansas' specification is consistent with the other states that responded.

TABLE 4.14 Maximum Allowed Lift Thickness

Lift Thickness (mm)	States Using
150	3
200	16
254	3
305	8
None	2

4.5 Summary

The results for all the testing performed for this project has been presented in this chapter. No evaluation of the information was presented for any of the sites. The information gathered from the testing performed on the disturbed samples, the undisturbed samples, field testing, and information collected during the phone survey are discussed in the following chapters.

Chapter 5

Discussion

This chapter contains an analysis of the results of the testing program. The significance of settlement, relative compaction, consolidation testing, DCP testing, and severity of settlement data collected during the research are discussed.

5.1 Embankment Settlement

When designing an embankment out of soil the engineer must account for some amount of settlement. When soils are loaded some amount of strain will occur. The definition of strain can easily be related to settlement and is commonly found in the form of:

$$s = \frac{\Delta e}{1 + e_0} * H$$

where Δe is the change in void ratio for the anticipated load, e_0 is the initial void ratio and H is the height of the embankment.

5.1.1 Potential Magnitude of Embankment Settlement

Anticipated settlement for a range of initial conditions was estimated for the average material properties of the soils collected for this project. However, the exact settlement cannot be determined for individual fills because the initial unit weights were not determined during construction. The average relative compaction and void ratio were determined to be 97 percent and 0.643 at the time of sampling, respectively. The relative compaction level increased during construction of the embankment due to the weight of additional fill. Assuming the relative compaction at the time of construction was 95 percent, the corresponding void ratio was back calculated to be 0.676. The settlement under these conditions would be calculated as 1.95 cm/m.

For an embankment 4 meters tall the anticipated settlement would be 7.8 cm. For an assumed initial relative compaction of 92 percent, which would comply with the existing specification, the estimated embankment settlement would have been 19 cm for a 4-meter embankment. It is likely that a significant portion of the settlement occurred during the construction process and was compensated for by the placement of additional fill. However in some cases much of the settlement clearly occurred after the embankment was completed.

This calculation represents a worst-case analysis for the center of the embankment but does show that compliance with the existing Type B/MR-90 specification is insufficient to ensure satisfactory embankment performance. It is important to note that foundation settlement was not included in this example and would have to be analyzed separately and added to the estimated settlement of the embankment.

5.2 Relative Compaction

The relative compaction of a sample is calculated by dividing the dry density of a field sample by the maximum dry density determined by AASHTO T 99 (KT-12). The number is usually reported as a percent. When a density specification is written the relative compaction is a common standard for evaluating the effectiveness of compaction.

The *in situ* density was determined for each of the 63 shelby tube samples collected for this project. A summary of the statistical analysis performed on the data is presented in Table 5.1. Figure 5.1 shows the relative compaction for each of the samples in the order they were collected in the field. Figure 5.2 presents a histogram of the relative compaction data.

TABLE 5.1 Statistical Analysis of Relative Compaction

Statistical Property	Percentage
Average Relative Compaction	97%
Standard Deviation	± 4%
Minimum	84%
Maximum	104%
Samples with a Relative Compaction Greater Than 90%	97%
Samples with a Relative Compaction Greater Than 95%	73%

The results show that at the time of sampling 97 percent of the soil samples had a relative compaction greater than the 90 percent relative compaction target normally associated with Type B/MR-90 compaction, but only 73 percent had a relative compaction greater than 95 percent. These results suggest that Type B/MR-90 compaction is not sufficiently rigorous to prevent excessive settlement, given that settlement problems have been observed and the average compaction of the samples from the poorest sites was over 94 percent, as discussed later in Section 5.8.1.

The potential for settlement could be reduced by requiring a minimum relative compaction of 95 percent of standard effort (KT-12 or AASHTO T 99) or adopting an equivalent specification based on the modified compactive effort (AASHTO T 180). Adoption of a relative compaction standard would require the use of compaction testing equipment in the field.

5.3 Effective Preconsolidation Stress vs. Relative Compaction

A slight correlation (R^2 = 0.12) between the effective preconsolidation stress calculated from the consolidation tests performed in the testing phase of this project and relative compaction can be seen in Figure 5.3. Based on this data, the compaction process has a relatively small impact on the effective preconsolidation stress.

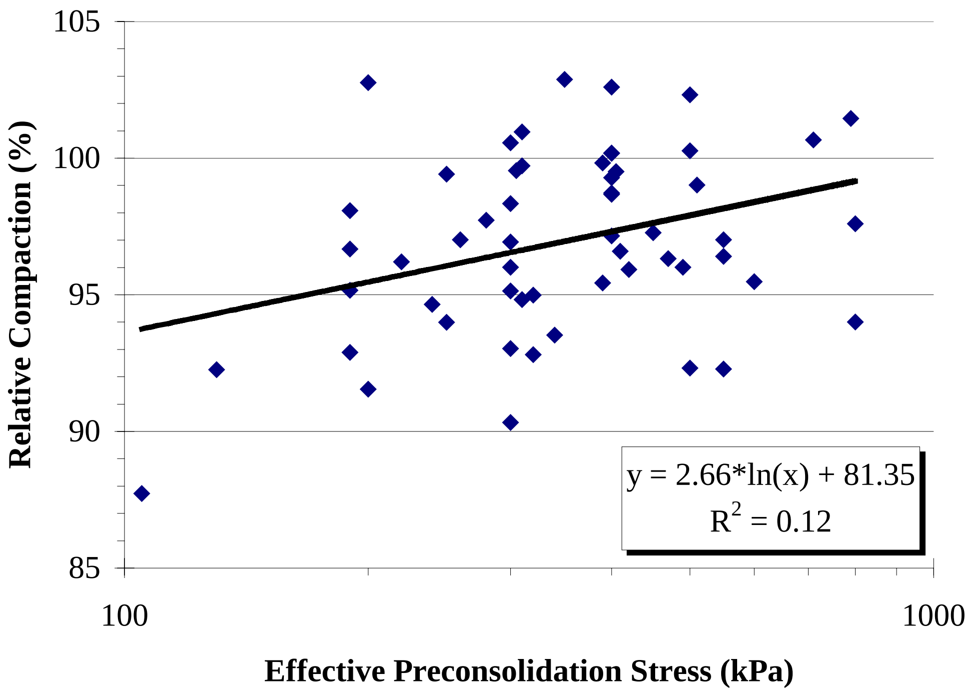


FIGURE 5.3 Effective Preconsolidation Stress vs. Relative Compaction

5.4 Effective Preconsolidation Stress vs. Depth

A correlation between effective preconsolidation stress and depth of fill was not observed. This was expected because the weight of the embankment is usually less than the preconsolidation pressure. The average effective preconsolidation pressure calculated from the consolidation tests performed for this project was 380 kPa. Assuming a unit weight of soil equal to 17 kN/m^3, an embankment would have to be over 22 meters tall before the effective preconsolidation pressure would be exceeded.

5.5 Dynamic Cone Penetrometer Evaluation

The results of the DCP testing performed on this projected were presented in Figures 4.11a and 4.11b. The conversion of the data to equivalent CBR values was presented in Tables 4.10a to 4.10d.

The quality and uniformity of compaction can be evaluated with the DCP to a depth of about 1.2 meters. DCP testing can locate soft layers by measuring the resistance of the soil to penetration. Examples of how the DCP data may be interpreted are presented in Figure 5.4. Each line in Figure 5.4 represents a separate DCP test.

An example of a possible soft layer from one of the sites investigated in this project can be seen in the I-35 & 151st graph. The two lines represent two tests performed approximately 1 meter apart. The spike in the resistance value at 1000 mm may indicate a soft zone. The upper portion of the plot may indicate moisture infiltration from precipitation and not poor compaction at the surface. The I-70 East of 350 site is an example of an embankment with low resistance values over the majority of the 1.2 meters tested. The plot shows a layer of low resistance near the surface with a thickness greater than 0.5 meters.

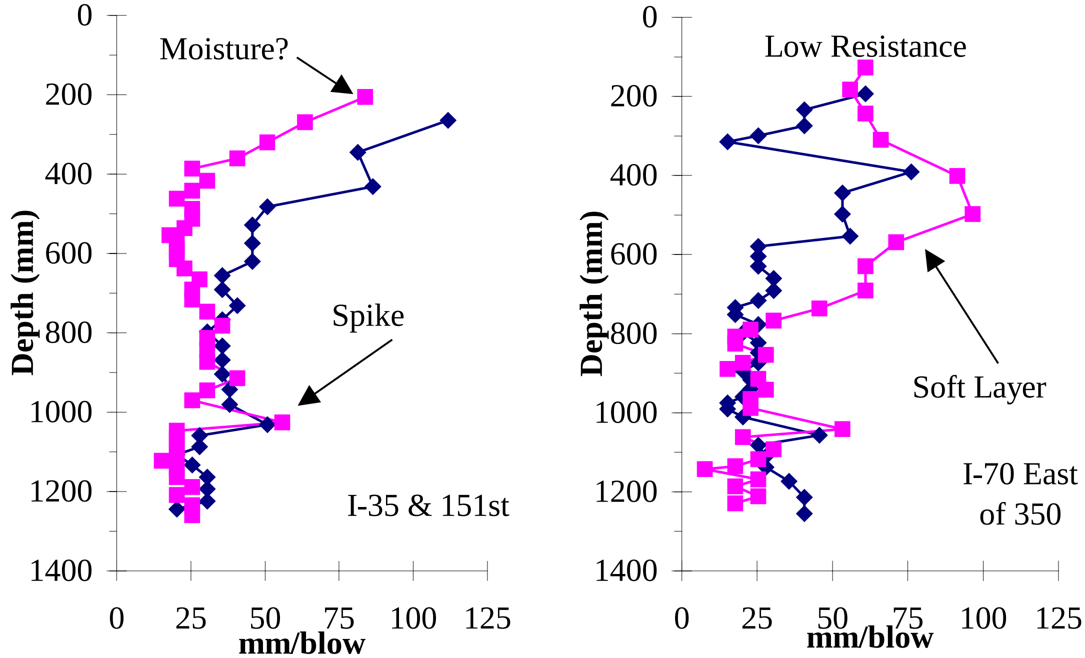


FIGURE 5.4 DCP Data Analysis

5.6 Collapse

A review of available literature showed that collapse phenomena have the potential to occur in all compacted soils. Testing was therefore conducted to evaluate the potential for collapse in the soils collected for this project. The results of a typical collapse test performed according to ASTM D-5333 are presented in Figure 5.5. Collapse testing was performed on undisturbed samples and remolded samples as described in Chapter 3. Collapse was not observed in any of the tests performed.

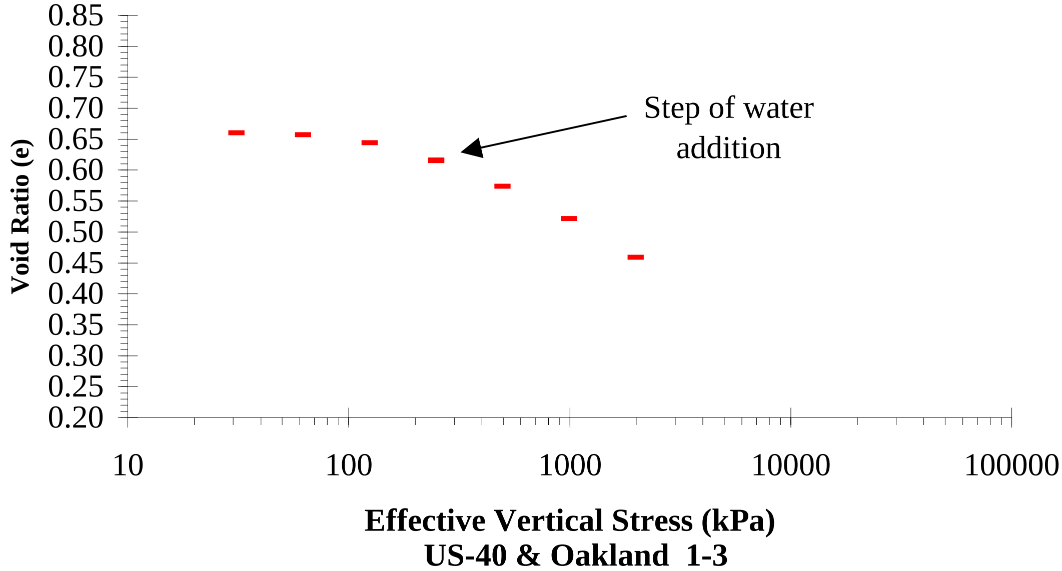


FIGURE 5.5 Example of a Typical Collapse Test Result

5.7 Correlations for Index Tests

Correlations between compression index and basic soil properties can be very useful to a design engineer. Consolidation testing is an expensive multi-day procedure and therefore is usually performed only on medium to large-scale projects. Index tests, such as Atterberg Limits, are performed routinely and correlations have been developed to use for design purposes. Terzaghi and Peck developed the correlation $C_c = 0.009*(LL-10)$ which is commonly used for preliminary calculations. Additional correlations have been developed using either liquid limit or void ratio to calculate compression index. Figure 5.6 shows the correlation between compression index and liquid limit for the data collected on this project. Figure 5.7 shows the correlation between compression index and sample void ratio for the data collected on this project. Terzaghi and Peck's correlation for normally consolidated clays is included in Figure 5.6 for reference.

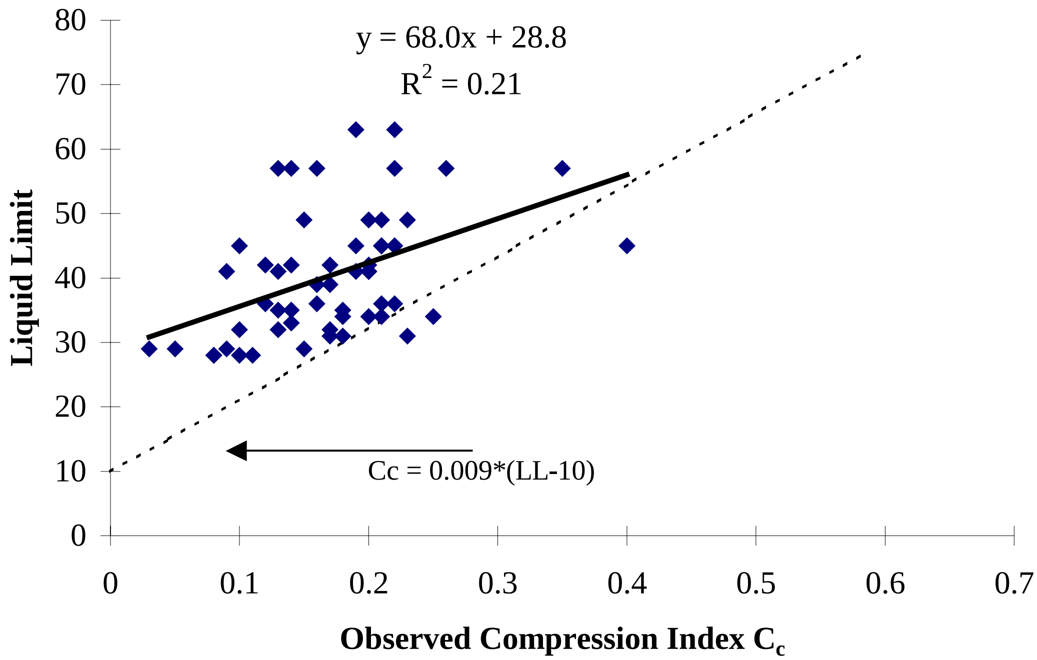


FIGURE 5.6 Liquid Limit Correlation

Figure 5.6 shows that the observed compression index is weakly correlated to liquid limit. A slightly stronger correlation between the compression index and sample void ratio is shown in Figure 5.7.

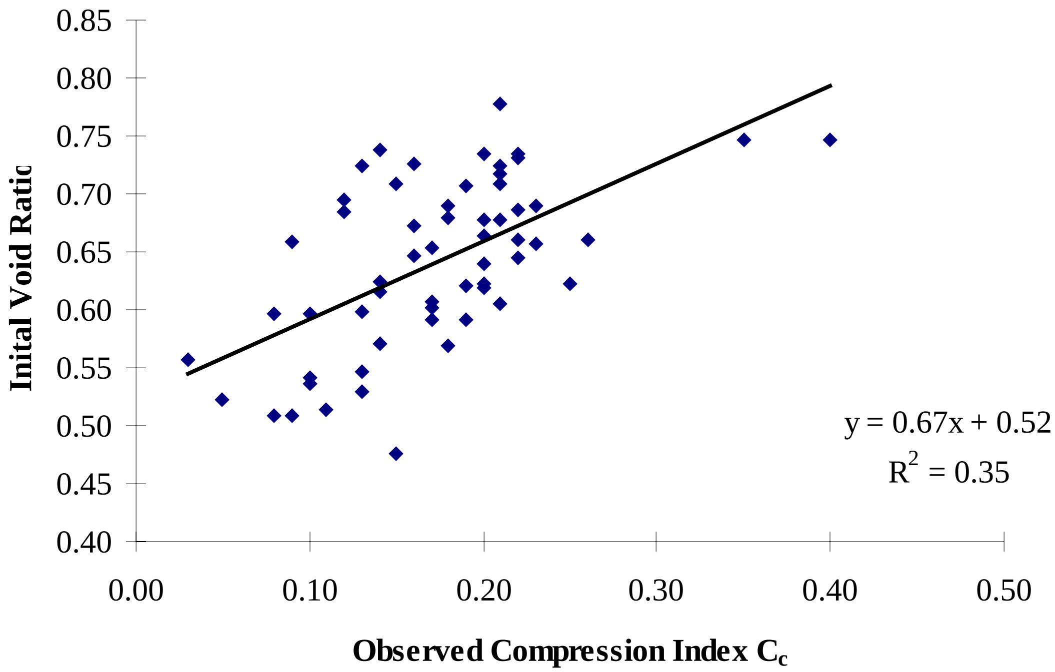


FIGURE 5.7 Sample Void Ratio Correlation

The recompression index (C_r) of a soil is usually approximated as 0.20 to 0.25 times the compression index (C_c). As shown in Figure 5.8, most of the recompression indices calculated from the consolidation tests do not fall between 0.25 and 0.20 Cc. Therefore, these should only be used for rough estimations of the recompression index of a soil.

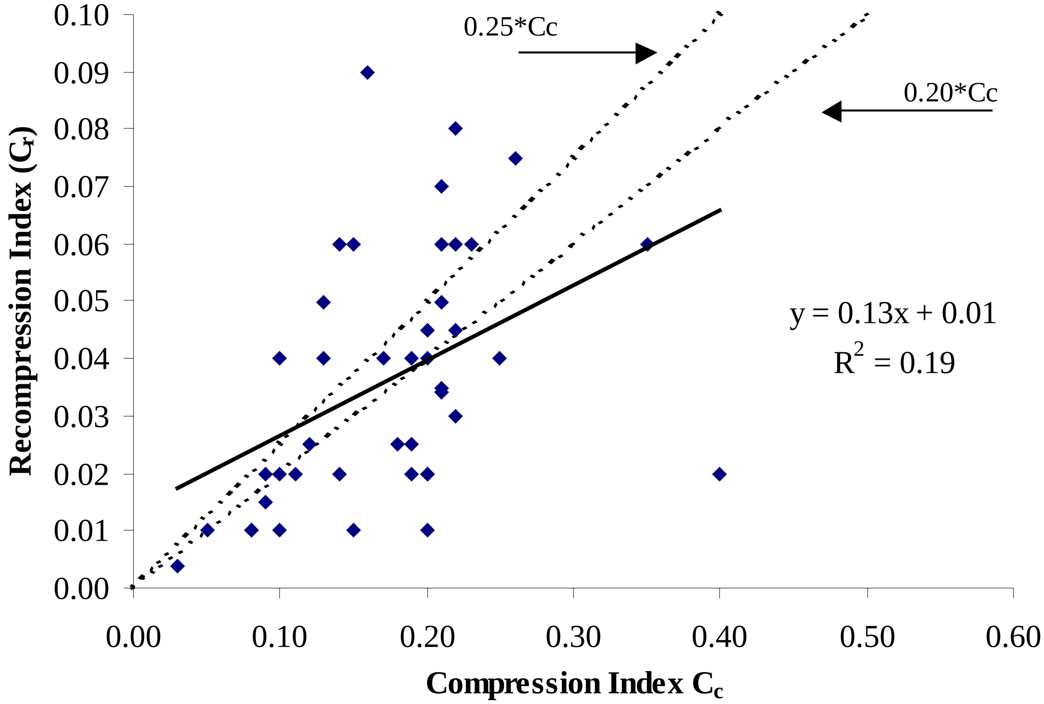


FIGURE 5.8 Compression Index vs. Recompression Index

5.8 Settlement Severity Analysis

A rating of the severity of settlement observed at each site as evaluated by KDOT personnel is presented in Table 5.2.

TABLE 5.2 Severity of Settlement

Site	Year Constructed	Settlement Severity	Rating
Brickyard	2000	1	Good
I-70 East of 350*	2000	10	Poor
I-70/K-30	2000	10	Poor
I-35 & 151st	1994	4	Fair
US-40 & Oakland	2000	1	Good
US-59/K-10	1996	1	Good
I-35 & Antioch	1998	1	Good
US-24 & Hog Creek	1997	7	Marginal

*data not included in Figures 5.9-5.11

5.8.1 Relative Compaction vs. Settlement Severity

The relationship between relative compaction and settlement severity is presented in Figure 5.9. This figure shows a modest trend towards greater compaction causing less severity in settlement problems. Samples from the marginal and poor sites have an average relative compaction more than three points lower than the fair and good sites. Additionally, there was much greater variability in compaction for the marginal and poor sites than for the good and fair sites. The relative compaction data from the I-70 East of Exit 350 embankment is not included in Figures 5.9 to 5.11 because the fill material test in this project was from a rehabilitated portion of the embankment.

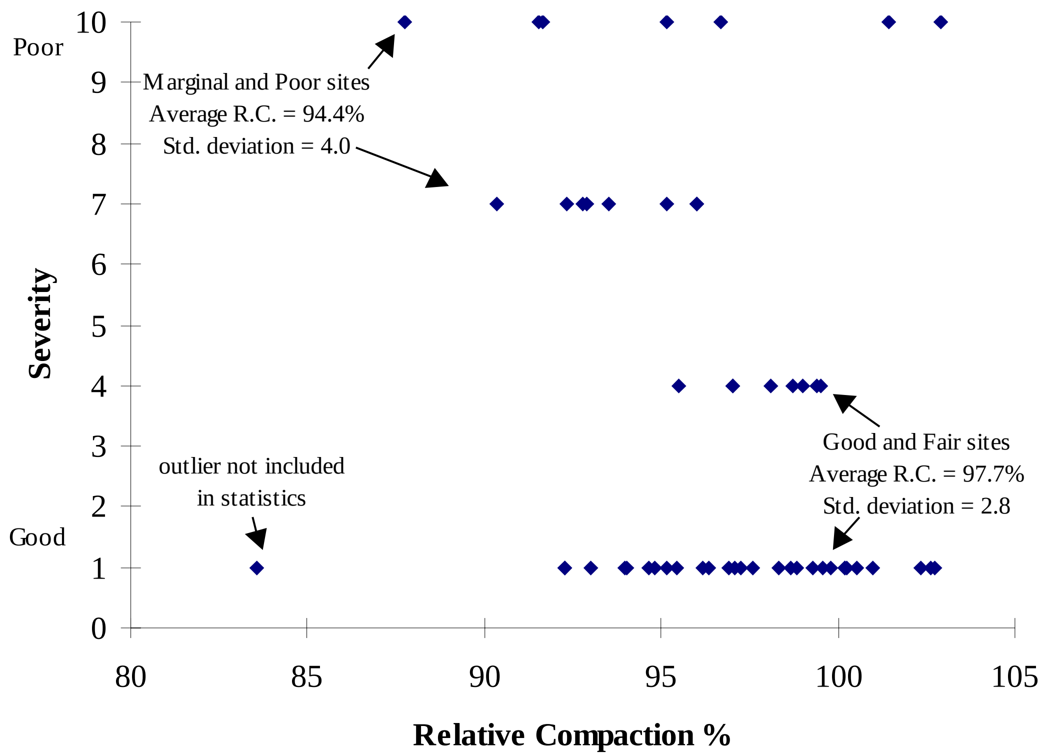


FIGURE 5.9 Relative Compaction vs. Severity

5.8.2 Settlement Severity vs. Deviation from Optimum Moisture Content

The moisture content at the time of sampling deviated from optimum moisture content by as much as 7.5 percent. As Figure 5.10 shows, the good and fair sites had average moisture contents slightly above optimum moisture, while the marginal and poor sites had moisture contents either significantly above or below optimum.

Poor compaction will result from moisture contents that are significantly different from optimum during the compaction process. The relationship between moisture and settlement severity shown in Figure 5.10 is not particularly strong, however this relationship may have been affected by changes in the moisture content due to infiltration between construction and sampling.

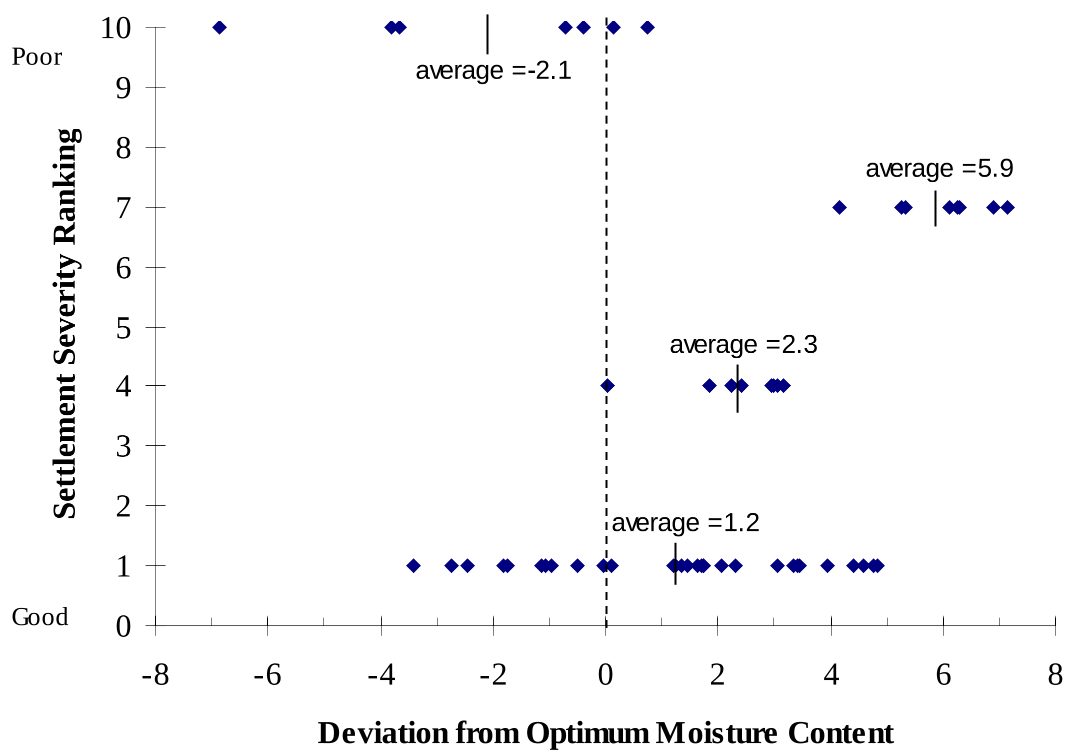


FIGURE 5.10 Deviation from OMC vs. Settlement Severity Ranking

5.8.3 Soil Classification vs. Settlement Severity

The data was reviewed to check for a correlation between soil classification and settlement severity. Based on the limited amount of data collected on this project, no correlation between these two variables was observed.

5.9 Moisture Content Deviation vs. Relative Compaction

Excessively high or low moisture contents during compaction can result in a low value of relative compaction. Therefore moisture content and relative compaction values were compared. At the time of sample collection the moisture contents of most of the samples were above optimum moisture content. The data is presented in Figure 5.11 and shows that extreme moisture content values correspond to lower relative compaction values.

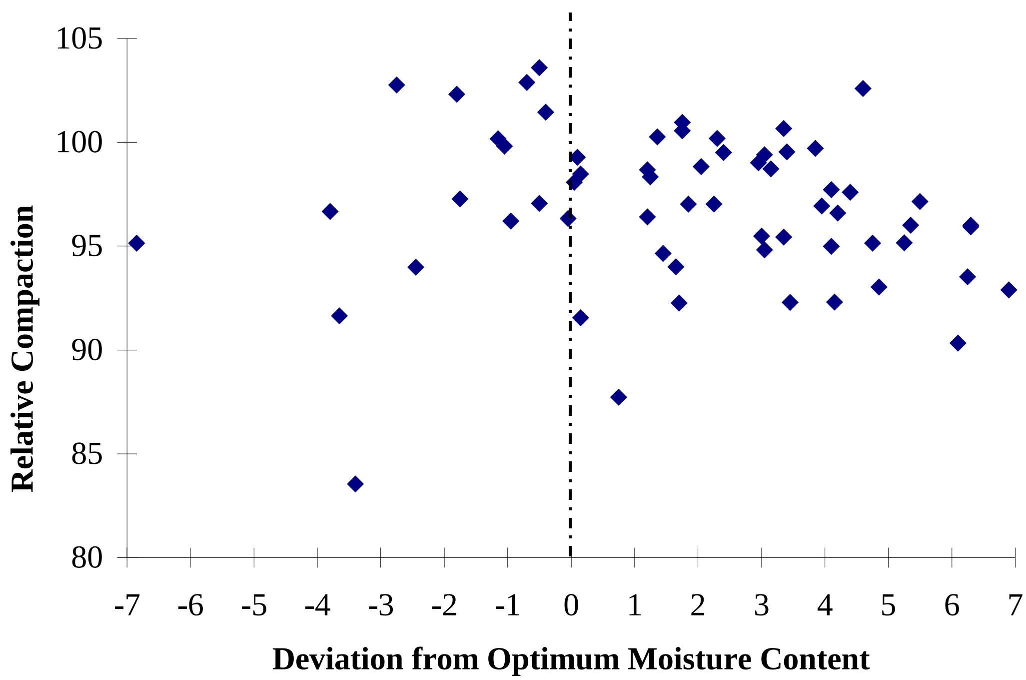


FIGURE 5.11 Deviation from Optimum Moisture Content vs. Relative Compaction

The relationship between moisture and relative compaction shown is weak, however this relationship may have been affected by changes in the moisture content due to infiltration between construction and sampling.

5.10 Survey

The compaction survey introduced in Chapter 3 offers some insight into compaction control methods currently used by surrounding states. The questions focused on moisture control, density control and verification, lift thickness, and compaction specification review status. The results of this survey are presented in Chapter 4, Tables 4.11 to 4.14.

5.10.1 Moisture Control

Over 75 percent of the states that responded to the survey have some type of moisture content control included in their compaction specifications. The moisture content specification varied from state to state. Some states permit a wide range around optimum moisture content while others strictly specify that the soil must be compacted at optimum moisture content only.

Moisture content specifications represent a balance between engineering requirements and constructability. The widest moisture content range specification reported was optimum ±5 percent (2 states). Five states specify the optimum moisture content with no range. The most common single specification was optimum ±2 percent (7 states).

5.10.2 Density Control

All states that responded to the survey specified some type of density control. Seventy-five percent of the states utilize standard compactive effort (AASHTO T 99 or equivalent) and 9 percent use modified compactive effort (AASHTO T 180 or equivalent). All states that specify a relative compaction standard require at least 90 percent of either standard or modified compactive effort depending on the compactive effort specified. The remaining 16 percent, which includes Kansas, do not incorporate density into their compaction specification and rely on other compaction control methods such as visual verification by sheepsfoot roller walkout.

Density specifications have limitations that are often overlooked by engineers. When using a density specification it is important to understand that maximum dry density varies with compactive effort and that a greater maximum density is possible with the application of a greater effort. It is also important to understand that the desired engineering properties, such as strength and permeability, may be competing and must be balanced when writing specifications.

5.10.3 Density Verification

The most common type of density verification tool used by states responding to the survey was the nuclear density gauge. States not using the nuclear density gauge are using simpler technologies such as the sand cone and rubber balloon methods for density verification. The nuclear density gauge was the most common choice because it is easy to use and results are obtained quickly. The two major drawbacks to using a nuclear gauge are expense and the inconvenience of handling a nuclear source.

5.10.4 Lift Thickness

Half of the states surveyed, including Kansas, specified a lift thickness of 200 mm (8 inches). The other half varied from no minimum lift thickness required to 305 mm (12 inches). The Kansas specification for lift thickness appears to be consistent with other states' specifications.

5.10.5 Specification Status

Over 80 percent of the states responded that they had no reason to review their compaction specification. One state believed that their specification was adequate but they were having trouble training new inspectors how to use it. Other states believed that their specification was adequate if followed. Some states are having trouble with specific types of soils but not general compaction specification problems. For example, the Virginia Transportation Research Council has been studying the "bump" at the end of the bridge problem for Virginia DOT. This was reported to be a common problem for many bridge approaches throughout the country (13).

KDOT and Iowa DOT were the only two states responding to the survey that rely on visual compaction verification methods. In 1997, Iowa DOT initiated a review of their compaction specifications. Based on the initial findings, Iowa DOT expanded the project into an extensive three phase review of the entire compaction process used by Iowa DOT and the contractors working for Iowa DOT. The next section contains a summary of the findings of that research to date.

5.10.6 Summary of the Iowa DOT Review of Compaction Specifications

Phase I, completed in May 1998, addressed the performance of the field personnel, the current (Pre-1997) specifications, and construction operations for both cohesive and cohesionless soils.

Bergeson concluded that field personnel were generally conscientious about trying to do a good job (7). However, they were misidentifying soils due to lack of knowledge and equipment, and relied heavily on soil design plan sheets for identification of unsuitable, suitable, and select soils.

The researchers concluded the methods described in the pre-1997 Iowa specifications might not be adequate for identifying unsuitable, suitable, and select soils. They also concluded that the one point Proctor is not a reliable identification method for all soils, and that sheepsfoot roller walkout is not a reliable indicator of degree of compaction, compaction moisture content, or adequate stability for all soils.

While observing construction operations for cohesive soils they found that soils were being placed wet of standard Proctor (AASHTO T 99) optimum moisture content and that soils were being compacted to near 100 percent saturation. Compaction of soils near complete saturation results in an embankment that has low shear strength and may result in positive pore pressure development within tall embankments. This can cause an embankment to have slope stability problems resulting in failures. Disking and lift leveling specifications were not always enforced and lifts were being placed on overcompacted soils.

Cohesionless soils were compacted with sheepsfoot rollers when vibratory rollers were more appropriate, and standard Proctor testing was used to develop compaction specifications for these soils rather than relative density. Monitoring compaction with cohesionless soils by Proctor testing can result is a gross overestimation of degree of compaction because the testing method is not appropriate for the cohesionless soils (7).

In Phase I, the researchers concluded that the current methods of embankment construction were not consistently producing quality embankments. Recommendations from the information gathered in Phase I lead to the initiation of Phase II.

In Phase II, Iowa DOT investigated construction with cohesive and cohesionless soils separately (34). They concluded that sheepsfoot roller walkout was not a reliable indicator of compaction quality and compaction was often performed wet of optimum moisture content, resulting in stability problems. Inconsistent lift thickness and roller passes were observed at several embankment projects. Soil disking, a useful tool for reducing clod size and lowering moisture content, was rarely demanded by field engineers as required by the current (pre 1997) specification. DCP testing was found to be a good quality control tool.

Experiments with various rollers and rolling patterns showed that rubber tired rollers and thicker lifts performed well. More research into this area was recommended.

Two new soil identification charts were developed from this research. The Iowa Soil Design and Construction (SDC) Charts A and B are charts designed to aid in the identification of soils. Chart A (Figure 5.12) was designed for cohesionless soils and requires a sieve analysis to identify suitable and select material. Chart B (Figure 5.13) was developed for plastic soils and requires the liquid limit and plasticity index of the soil. The legend for these charts is presented in Table 5.3.

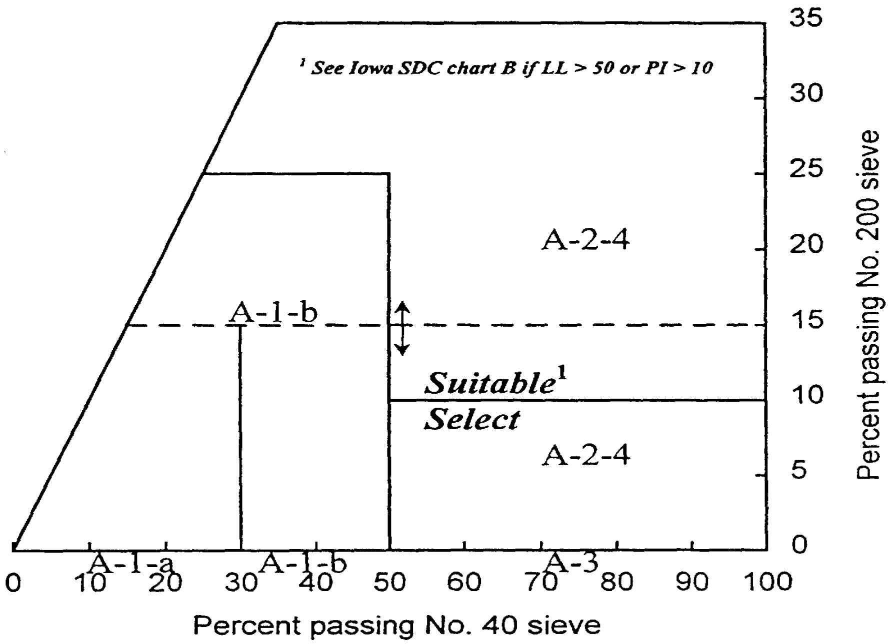


FIGURE 5.12 Iowa Soil Design and Construction (SDC) Chart A (34)

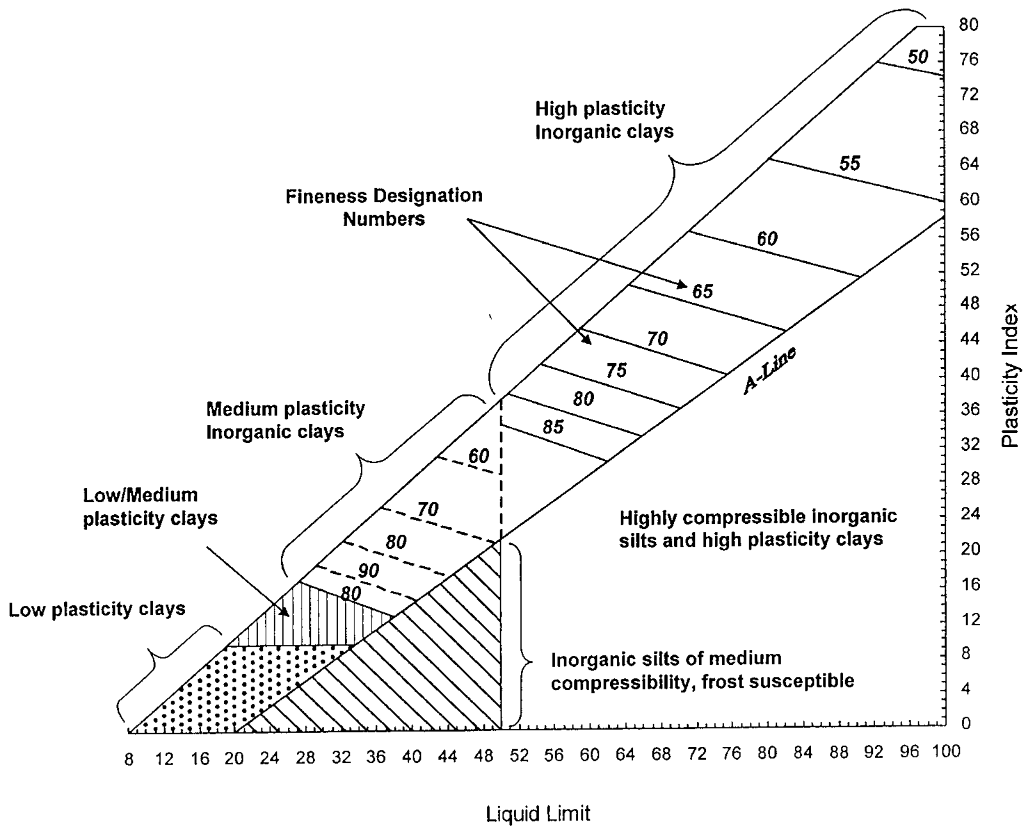
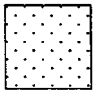
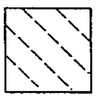
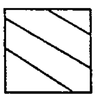
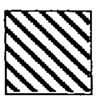
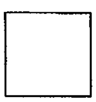


FIGURE 5.13 Iowa Soil Design and Construction (SDC) Chart B (34)

TABLE 5.3 Iowa Soil Design Chart Guidelines (34)

Legend	Designated Soil Regions
	Low plasticity clays • Select ≤ 45% passing the No. 200 sieve, F_{200}, and ≤ 70% F_{40} • Suitable 46% - 70% F_{200} • Unsuitable > 70% F_{200} (Type C disposal)
	Low/Medium plasticity inorganic clays • Select ≤ 60% F_{200} • Suitable 61 - 70% F_{200} • Unsuitable ≥ 70% F_{200} (Type C disposal)
	Medium plasticity inorganic clays • Select – Plots above A-Line (PI=0.73(LL-20), and F_{200} ≤ fineness designation • Suitable - F_{200} > fineness designation
	High plasticity inorganic clays • Suitable - Plots above A-Line (PI=0.73(LL-20), and F_{200} ≤ fineness designation • Unsuitable - F_{200} > fineness designation (Type B Disposal)
	Inorganic silts of medium compressibility • Unsuitable - Plots below A-Line (PI=0.73(LL-20), • Type B Disposal
	Highly compressible inorganic silts and high plasticity organic clays • Unsuitable < 3.0% carbon (Type A disposal) • Unsuitable ≥ 3.0% carbon (Slope dressing only)

Note (1) All soils other than "Highly compressible inorganic clays and high plasticity organic clays" containing 3.0% or more carbon are to be placed according to Type C disposal method.

Note (2) Shale is to be placed according to Type A disposal method.

The Iowa Moisture Content Control (MCC) Charts A and B, Figures 5.14 and 5.15 respectively, were designed to aid in moisture content control. Again, Chart A is for cohesionless soils and Chart B is for plastic soils. These charts, used in conjunction with the Iowa SDC charts, further aid field personnel in the proper identification of soils for proper placement and construction.

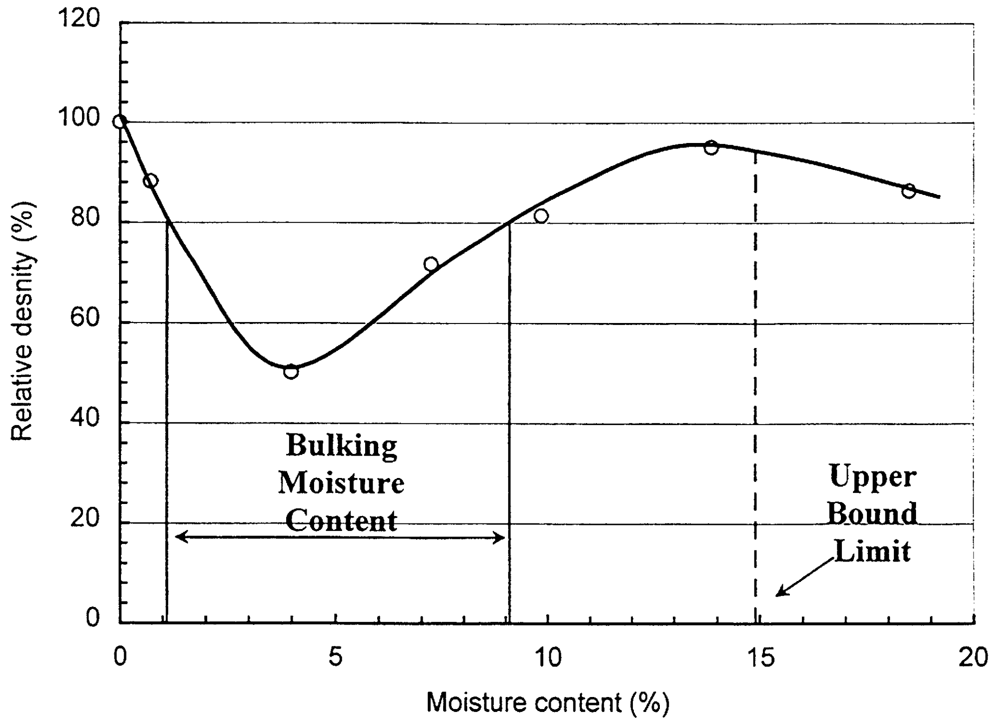


FIGURE 5.14 Iowa Moisture Content Control (MCC) Chart A for Granular Soils

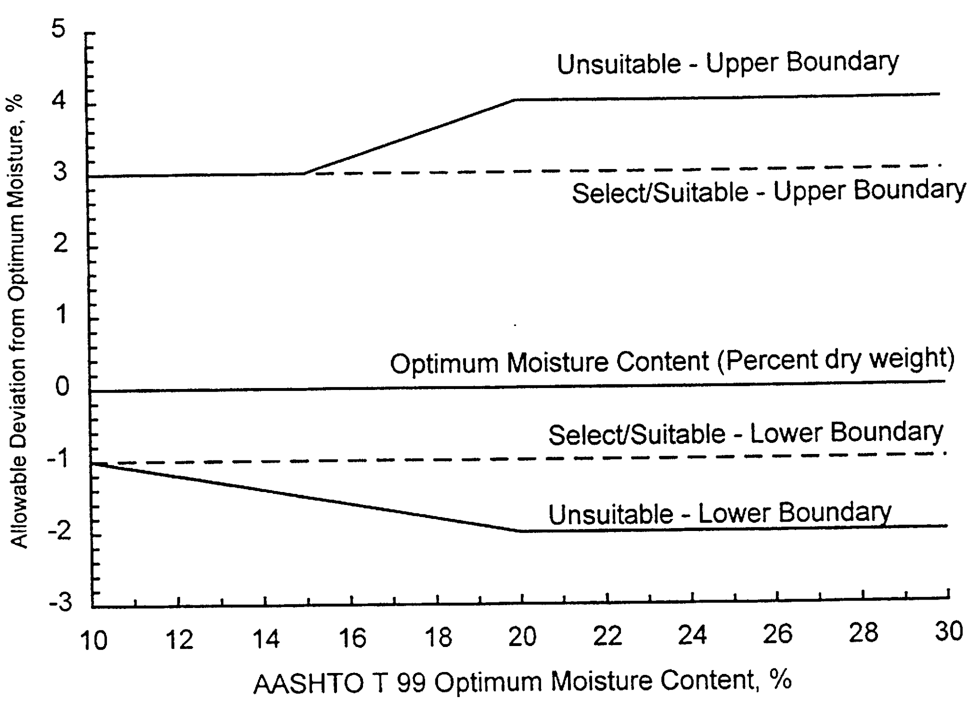


FIGURE 5.15 Iowa Moisture Content Control (MCC) Chart B for Plastic Soils

During Phase II a pilot project was developed to test the practicality and implementation of these recommendations. The pilot project developed into Phase III of this project. This includes a full-scale design project using only the new specifications and design charts developed through this research. A report is expected on the results of Phase III in 2002.

5.11 Summary

The relative compaction achieved with the KDOT Type B compaction specification was better, on average, than required based on the densities determined at the time of sampling. Given that settlement is a recurring problem this suggests that Type B/MR-90 compaction is insufficient to ensure satisfactory embankment performance.

Effective preconsolidation stress appears to be slightly related to relative compaction. No relationship between effective preconsolidation stress and depth of fill was found.

The DCP appears to be a useful tool for quality control of compacted fill. Location of soft layers up to 1.2 meters below the surface is possible with the DCP apparatus.

A review of the literature revealed that all compacted soils might be susceptible to wetting-induced collapse. Collapse tests were performed with soil from two sites. Neither site showed any collapse potential.

Correlations to determine the compressibility index of a soil from basic soil properties were compared to actual data. Correlations based on liquid limit and sample void ratios were evaluated. The correlations using void ratio were more strongly correlated than the ones using liquid limit. However, the void ratios were calculated from the samples at the time of collection. Assuming the embankment settled into a denser state this would affect the correlations using void ratio and not affect the correlations using liquid limit.

Of the 32 states responding to the survey conducted for this project, only Iowa and Kansas relied on a visual compaction specification. Iowa DOT is in the process of conducting a major review of their compaction specifications. Their initial findings suggest that KDOT may need to investigate their compaction specification in more detail. Iowa DOT research showed that sheepsfoot roller walkout is not a reliable indicator of compaction quality for all soils. Recommendations were made as a part of the Iowa study to implement new design charts and testing equipment for quality control of embankment construction.

Chapter 6

Conclusions

The following sections contain conclusions developed based on the test data and a review of the available literature. The most important conclusion about embankment settlement is that it cannot be completely avoided. Settlement can be expected to occur to some degree in the foundation or the embankment or both.

6.1 Relative Compaction Specifications

The compaction specification most commonly used by the KDOT, Type B MR-90, is apparently being met by current construction procedures. This is a visual specification that relies on sheepsfoot roller walkout to verify that an acceptable level of compaction is achieved. In fact a relative compaction greater than 90 percent of standard effort (KT-12 or AASHTO T 99) appears to be generally achieved by sheepsfoot roller walkout. The inconsistent performance of embankments, despite being compacted in accordance with the specification, suggests that Type B/MR-90 compaction is not sufficient to ensure satisfactory performance.

Performance was related to compaction for the embankments investigated. Embankments that performed well or fair had a greater average relative compaction (97.7 percent), more consistent compaction, and moisture contents slightly wet of optimum. The marginal and poorly performing embankments had lower relative compaction levels (94.4 percent) and more erratic moisture contents. These values were determined from samples taken months or years after construction. Relative compaction levels at the time of construction were almost certainly lower.

Increasing the minimum required relative compaction to 95 percent of standard effort as defined by KT-12 (AASHTO T 99) or adopting an equivalent specification based on modified compactive effort (AASHTO T 180) would decrease the amount of embankment settlement by decreasing the initial void ratio of the fill. Adoption of a specific relative compaction standard would also be consistent with the specifications of most of the states responding to the survey. Use of a specific relative compaction standard would necessitate the use of density verification equipment.

6.2 Density Verification Equipment

Adoption of a relative compaction specification would require the use of density verification equipment by field inspectors. A large majority of the states responding to the survey (25/32) use the nuclear gauge for density verification. The remaining states used the sand cone, balloon method, visual methods, or the 1 point Proctor. The U.S. Army Corps of Engineers drive cylinder test (ASTM D-2937) was also used successfully for cohesive soils by the Iowa researchers and has been used successfully by researchers at the University of Kansas.

6.3 Dynamic Cone Penetrometer

DCP testing appears to be a useful tool for evaluation of the compaction of soil. The equipment is portable, easy to perform, inexpensive and the data is easily reduced with the aid of a spreadsheet. The test is sensitive to environmental factors such as rain and frozen soils.

DCP testing could be used at regular intervals throughout the construction process to help locate soft layers from the surface down to 1.2 meters below the surface. This would permit evaluation of the fill at greater depth than possible with standard compaction verification technologies and give an opportunity for remediation before completion of the project.

Correlations between the DCP and density are not sufficiently developed to replace density testing. If it is used, the DCP should be used in coordination with density verification technologies and not in place of them.

6.4 Multi-State Survey of Compaction Specifications

The survey conducted to gather information about the status of compaction specifications in surrounding states was useful in evaluating potential compaction specification problems in Kansas. Kansas is one of only two states responding to the survey that still uses visual methods for verification of adequate compaction. More quantitative methods, such as the nuclear density gauge, are used in the majority of states responding to the survey. Iowa is the only other state that responded to the survey that still uses visual compaction control and it is currently engaged in a comprehensive review of its compaction specifications.

6.5 Preconsolidation and Collapse Potential

Based on the test data, the effective vertical stress will be lower than the effective preconsolidation stress for a large percentage of the embankments built by KDOT. This means that the embankment should always be in the recompression zone for the soil. This condition should limit the amount of settlement for embankments in Kansas because there will be no virgin compression experienced by the embankment.

An increase in relative compaction appears to slightly increase the effective preconsolidation stress. However, there was no evidence that a higher effective preconsolidation stress was related to the height of embankments typically built by KDOT. The average effective preconsolidation stress for all the consolidation tests performed for this project was 380 kPa. For the height of embankments typically built by KDOT all the soils evaluated on this projected should behave as overconsolidated clays. An embankment would have to be over 20 meters tall before virgin compression would occur.

As explained in Chapter 2, previous research has shown that all compacted soils are susceptible to wetting-induced collapse. However, the collapse tests conducted as a part of this research did not show that collapse was a problem for the soils investigated.

Chapter 7

Recommendations and Implementation Plan

While settlement of embankments cannot be eliminated, steps can be taken to reduce the magnitude of embankment settlement. Effort should be made to control excessive settlement so that the embankments perform as required. Specific recommendations for improving embankment performance are contained in the following sections.

7.1 Update Compaction Specifications

For the embankments investigated as a part of this research, the existing Type B/MR-90 specification was insufficient to ensure adequate embankment performance. Similar visual-based specifications have been phased out in most other states. It is therefore recommended that the visual standard be replaced with a relative compaction standard. It is also recommended that this standard be at least 95 percent of standard effort as defined by KT-12 (AASHTO T 99), which is the most common standard used by the states responding to the survey. An equivalent standard based on the modified compactive effort (AASHTO T 180) could also be established.

It is recommended that the moisture content specification be updated to specify a range about the optimum moisture content. As shown in Table 4.11, the most common single range specified among the states surveyed was optimum ±2 percent. The adoption of a slightly wetter range, such as –1 to +3 percent, may be preferable given the best performing embankments had an average water content of 1.2 percent above optimum.

Where possible, it is recommended that a construction delay be scheduled between the completion of the embankment and final dressing and paving. A scheduled delay would allow time for the embankment to settle during a less critical construction stage. Sufficient additional fill to reach final grade could then be placed prior to paving, or the embankment may be built slightly above final grade during the initial construction and the fill may be reworked or trimmed as needed prior to construction of the pavement.

7.2 Compaction Verification Technologies

Verification of compaction during construction is one of the most critical steps in embankment construction. It is recommended that field-testing by quantitative methods such as the nuclear density gauge, sand cone, or drive cylinder be implemented for all fills as these procedures provide more useful information than visual verification methods about the quality of compaction achieved during a project.

DCP testing equipment can provide field personnel with an additional method for evaluating the quality of construction throughout the embankment. Each meter of fill material can be evaluated in minutes with the DCP.

7.3 Review of the Iowa DOT Projects

It recommended the results of the ongoing Iowa research be monitored for application in Kansas. The quality of embankment construction and the effectiveness of current compaction specifications have been under review in the state of Iowa for about five years. The information already collected by Iowa DOT as well as the results from field tests currently underway may provide guidance in planning future research. Development of a joint study with Iowa should be considered.

7.4 Compaction Training Video

The feasibility of creating an education video for inspectors was established. However, it is not recommended that a video be produced if quantitative standards are implemented in place of the existing visual standard. Training on the proper use of the equipment in the field should provide ample opportunity to train inspectors how to verify that compaction specifications have been met.

7.5 Further Research

More extensive research is recommended to better evaluate embankment performance. In addition to monitoring or participating in the work performed by Iowa, it is recommended that relative compaction, moisture, and survey data collected during future construction activities be maintained such that it may be easily accessed during a future study. It is also recommended that formal, quantitative settlement severity criteria be established. Limits defining how much settlement constitutes a given settlement severity would improve the usefulness of the data.

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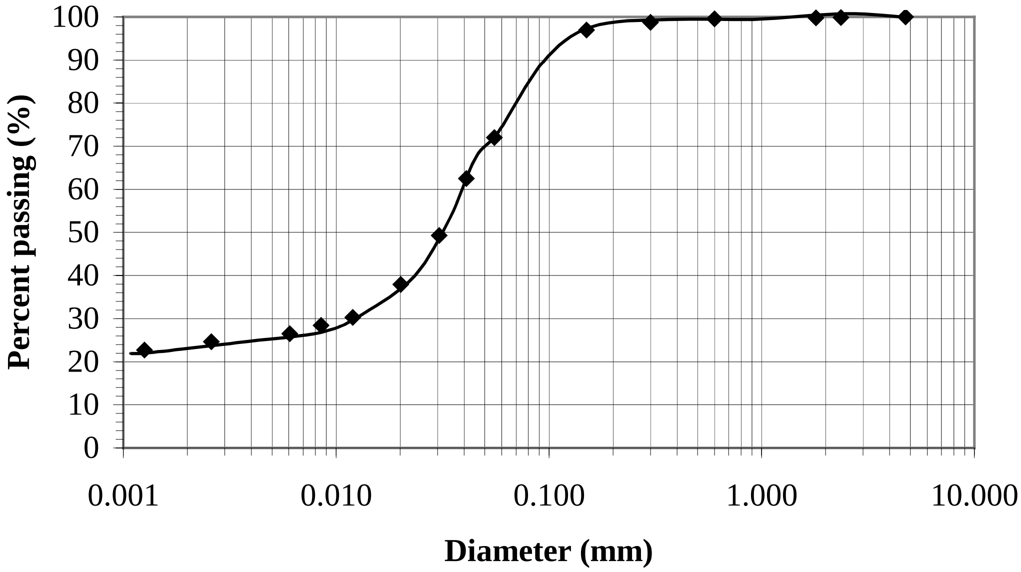
Appendix A

Grain Size Distribution Graphs



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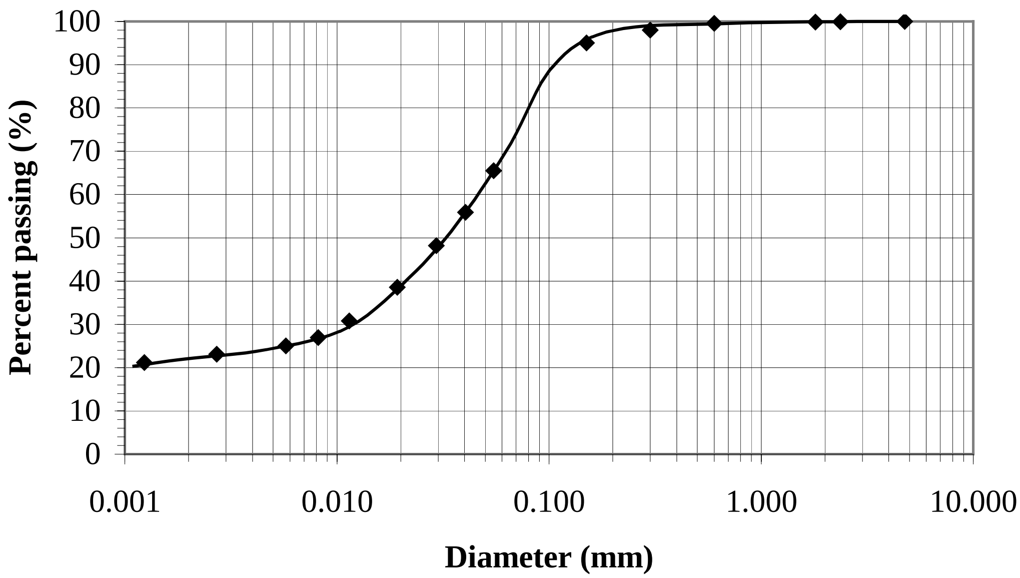

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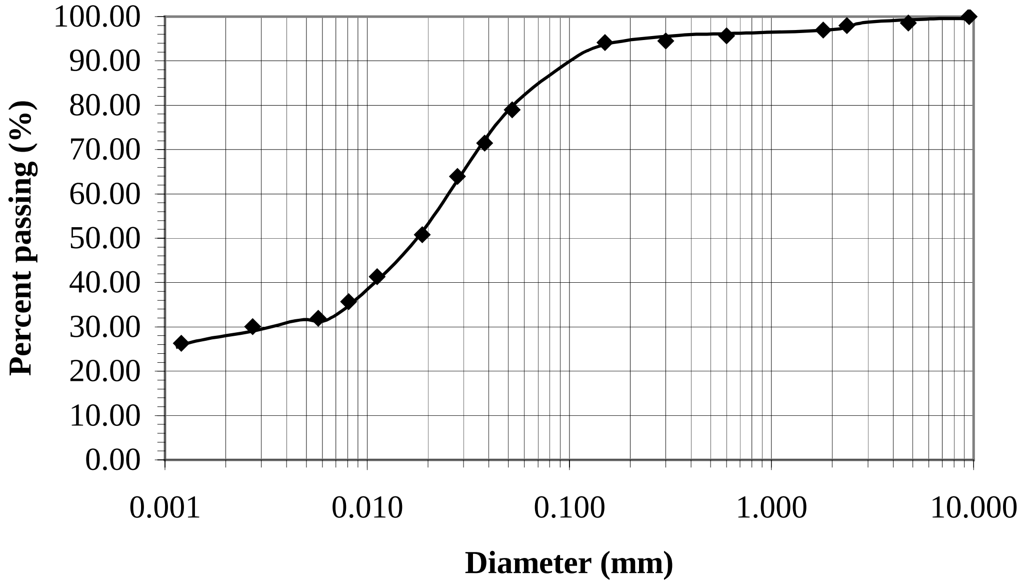

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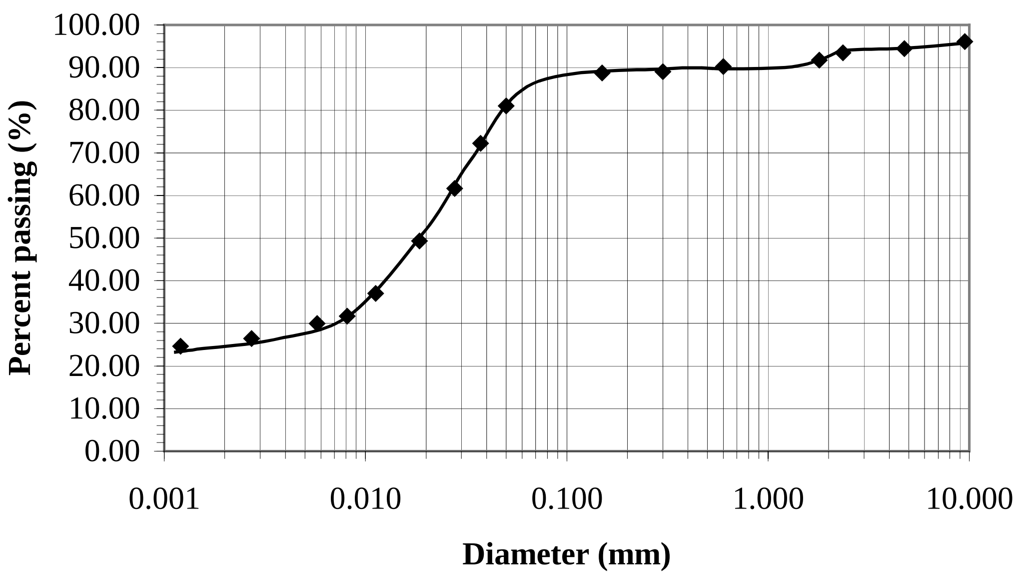

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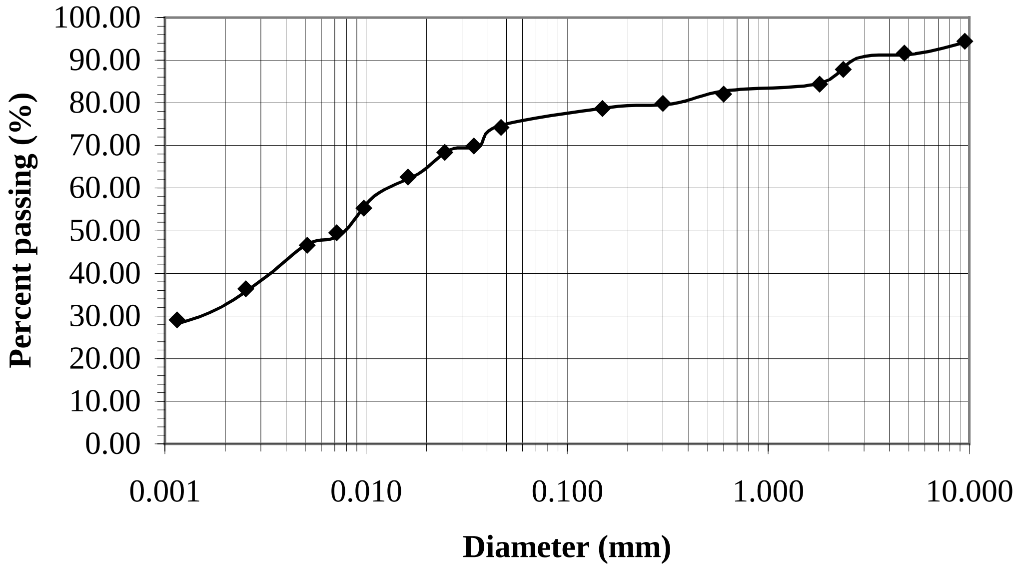

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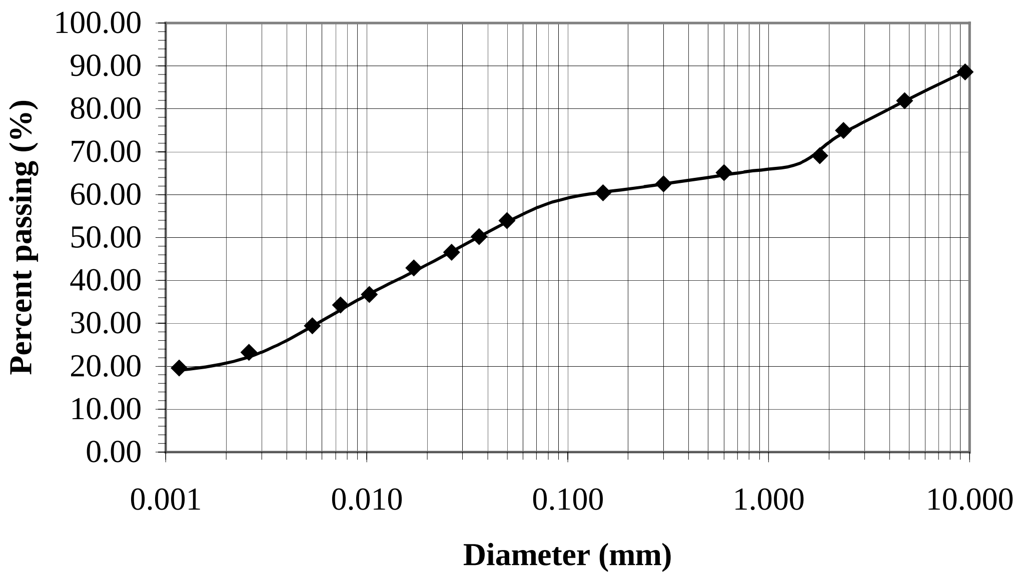

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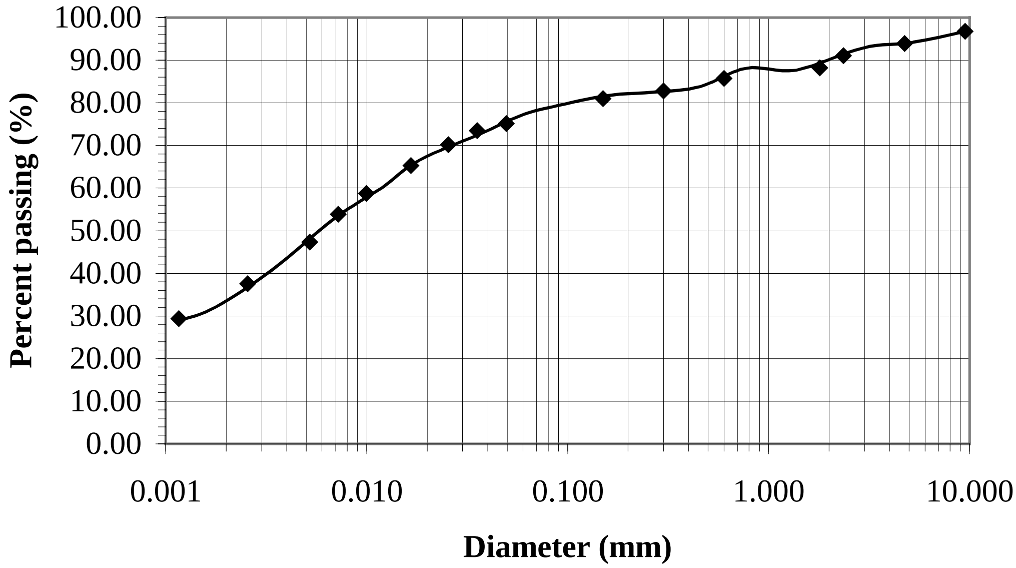

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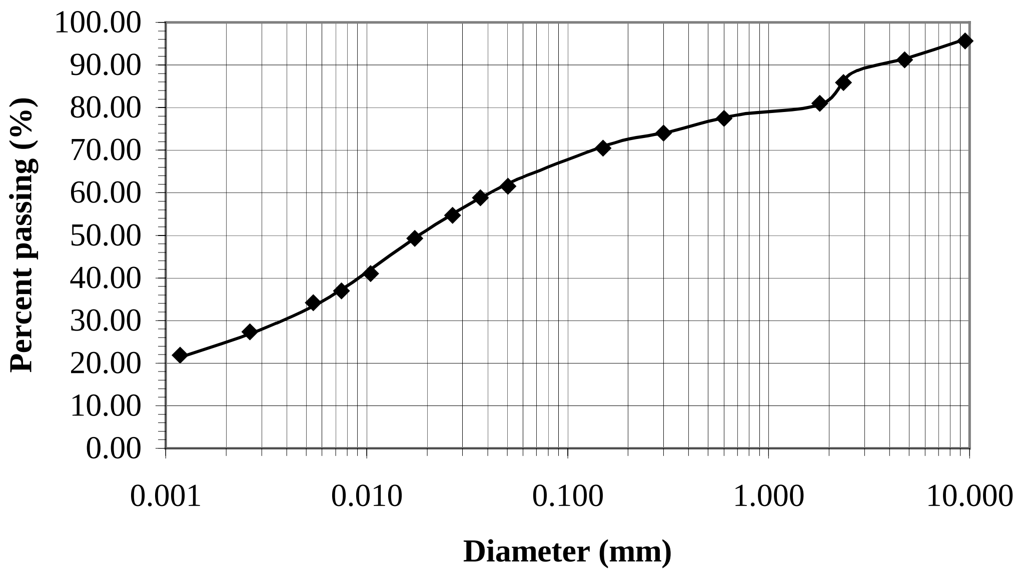

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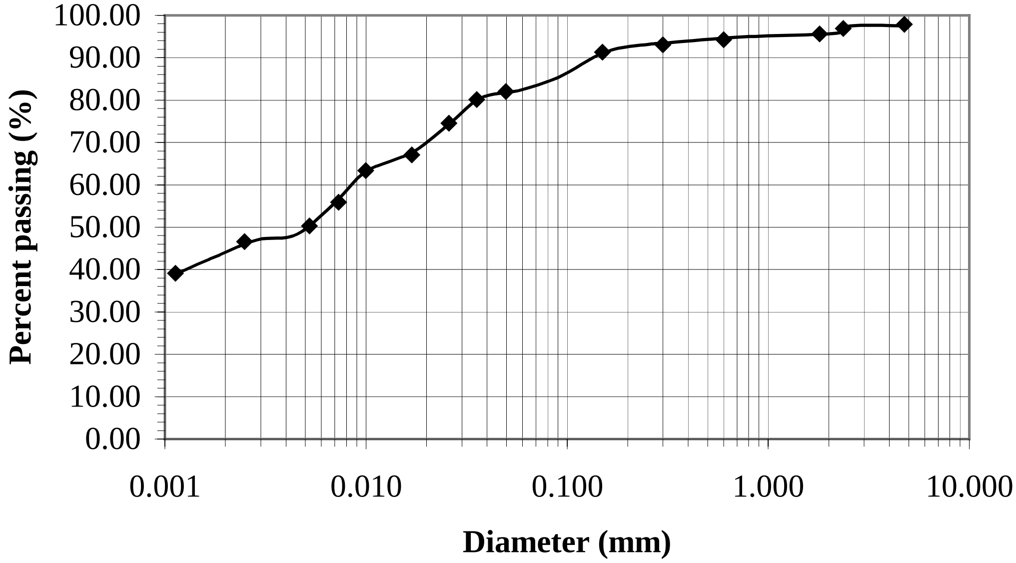

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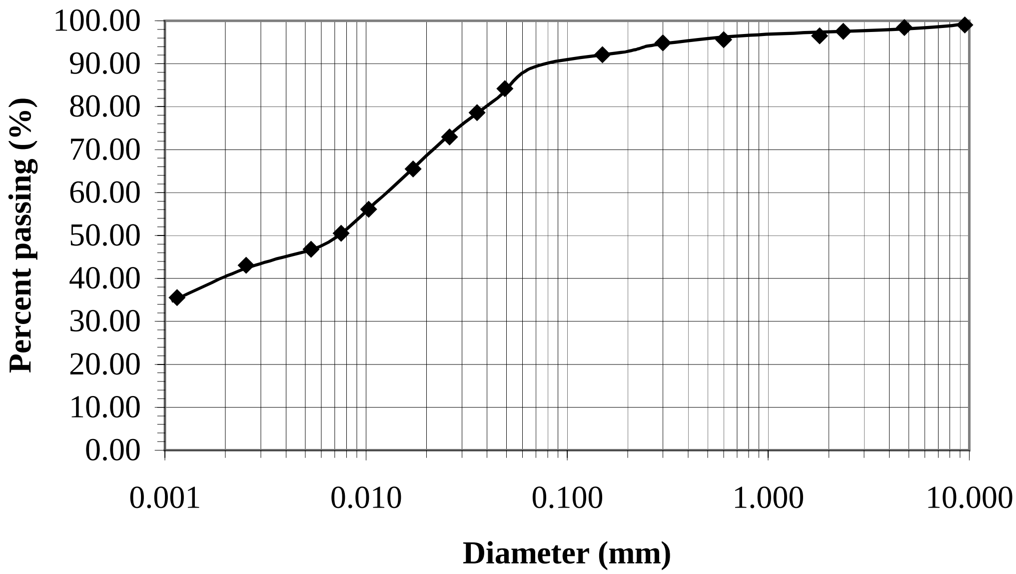

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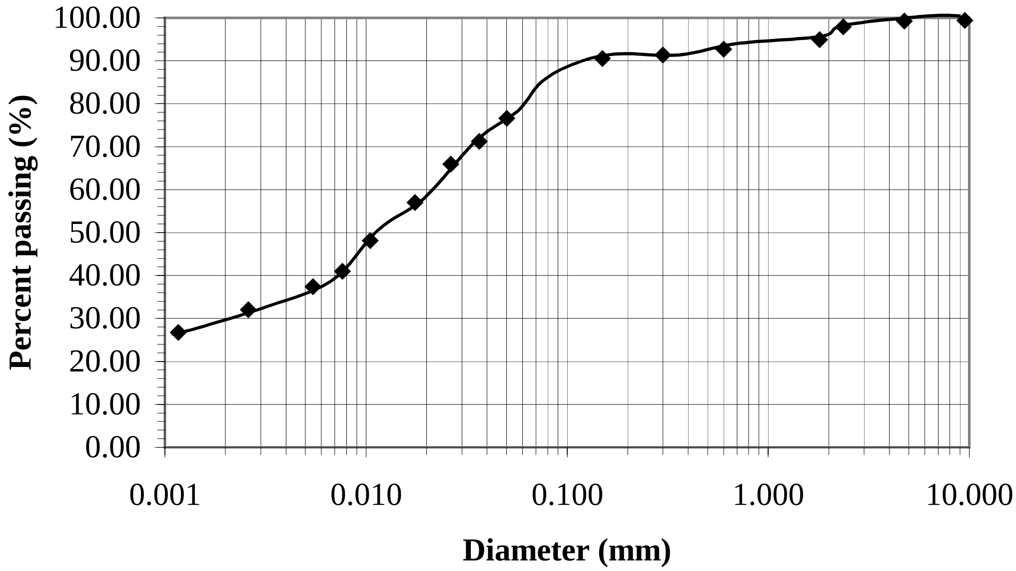

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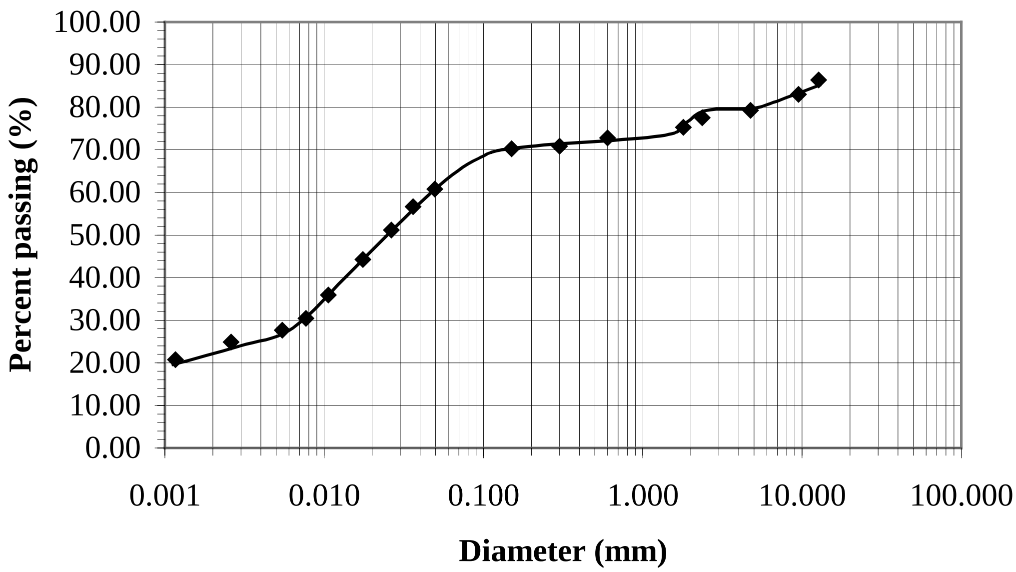

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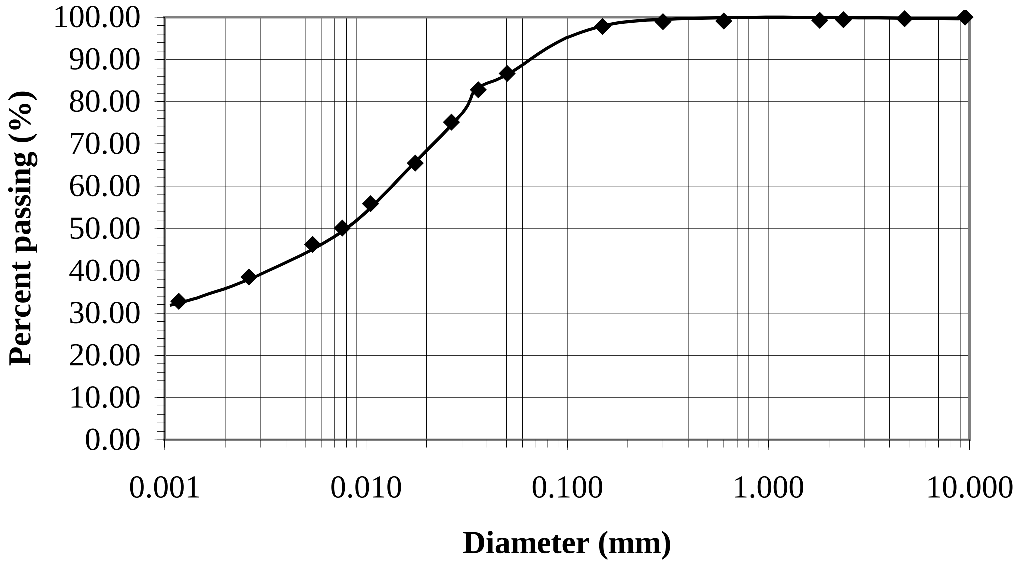

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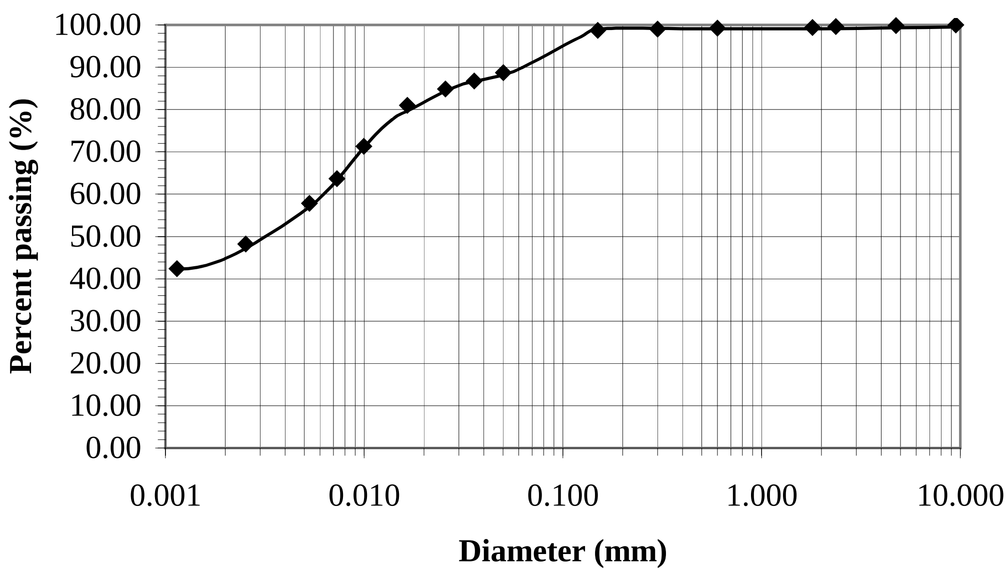

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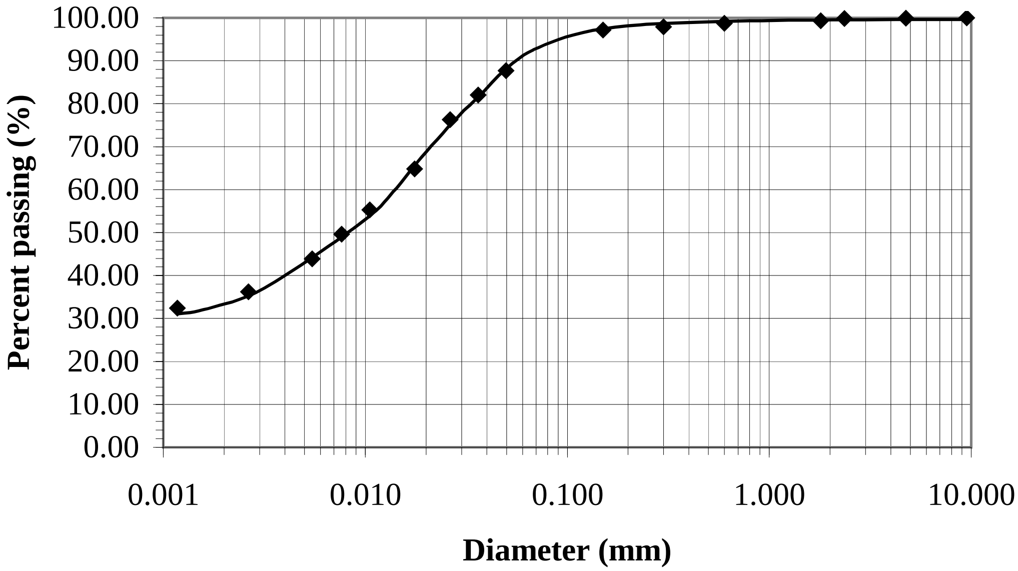

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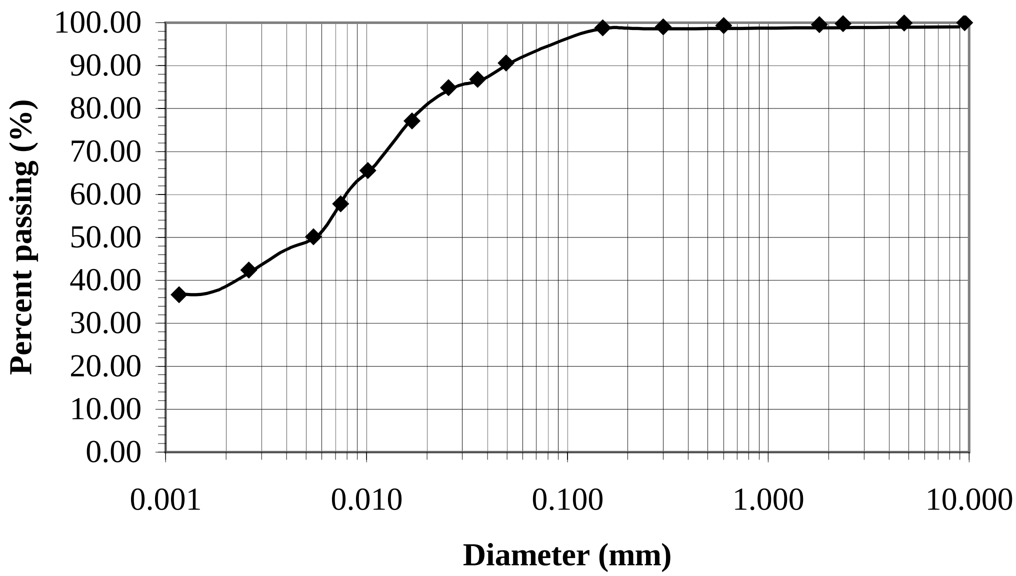

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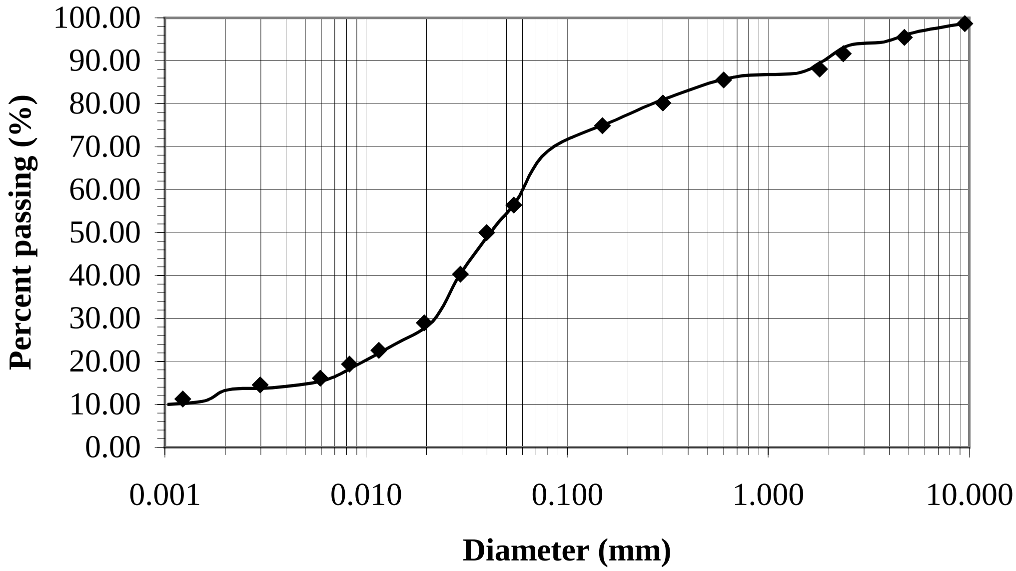

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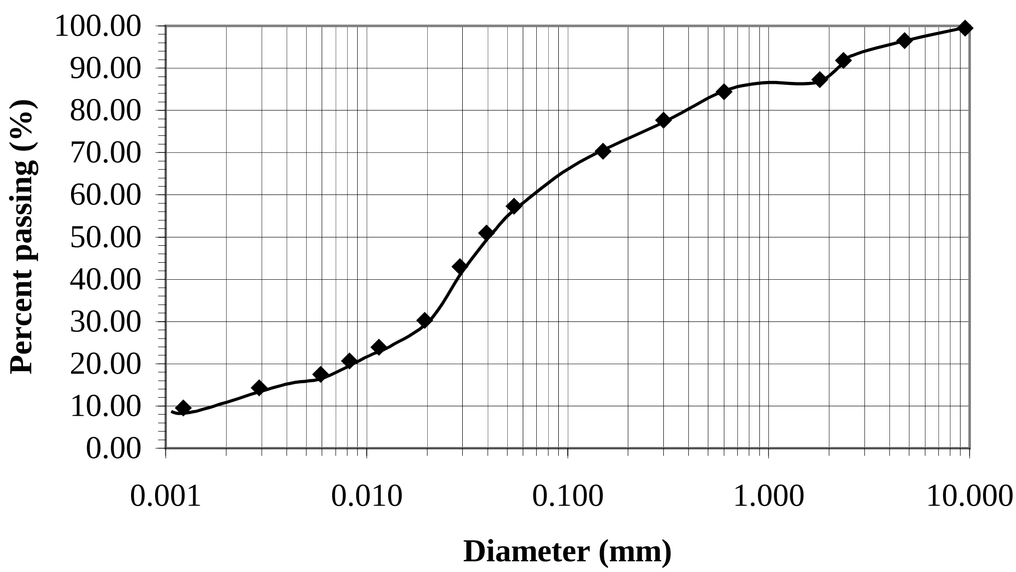

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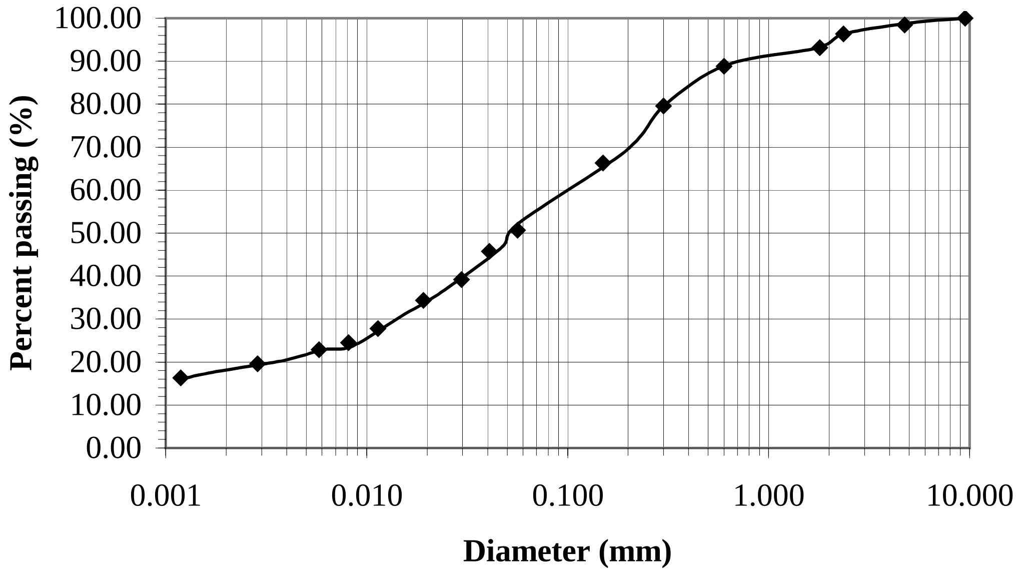
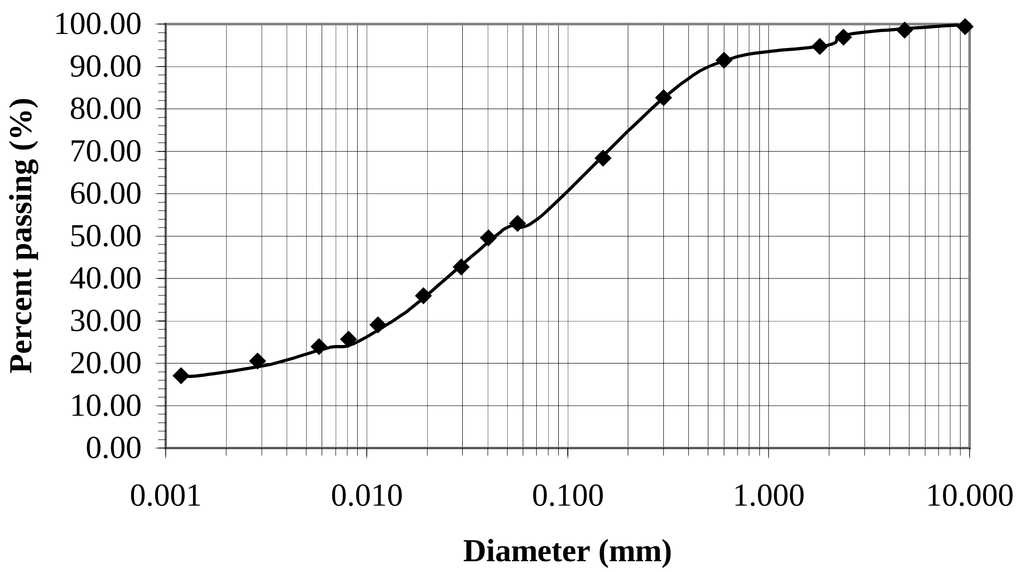


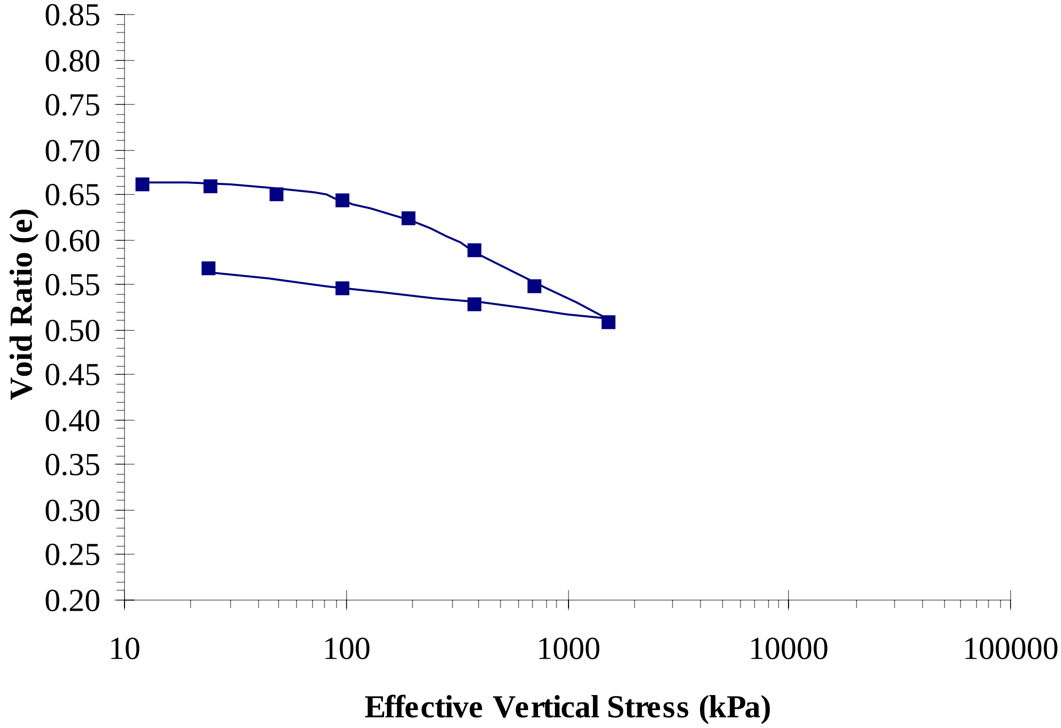
US-24 & Hog Creek Hole # 1



US-24 & Hog Creek Hole # 2

Appendix B

Consolidation Graphs



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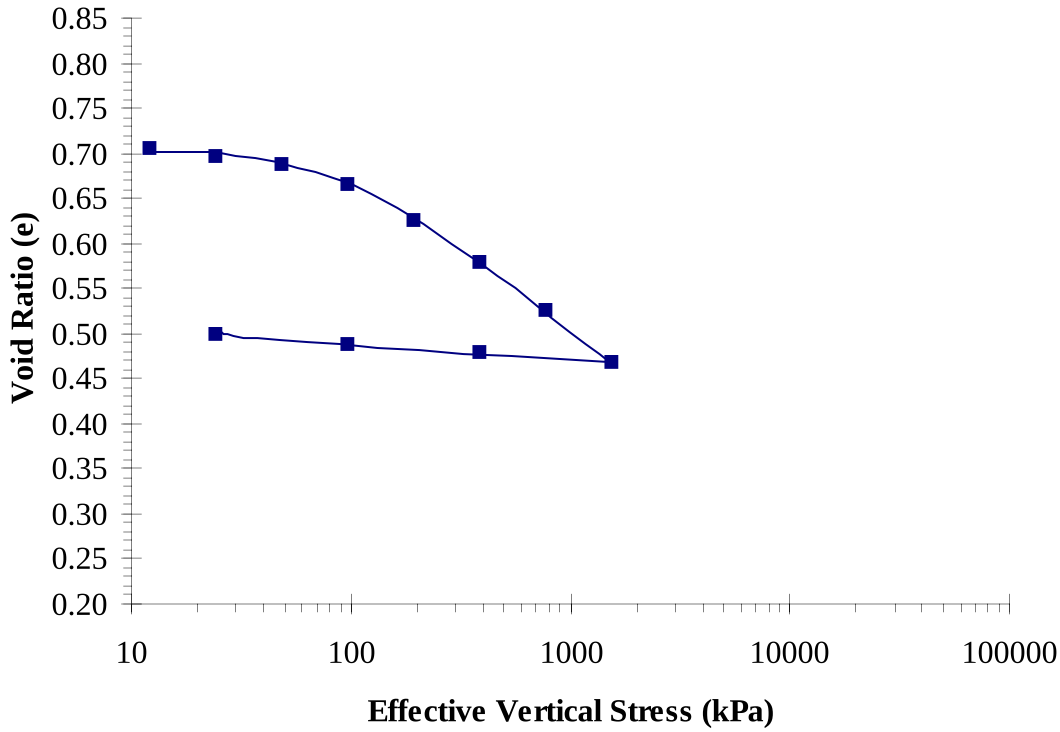

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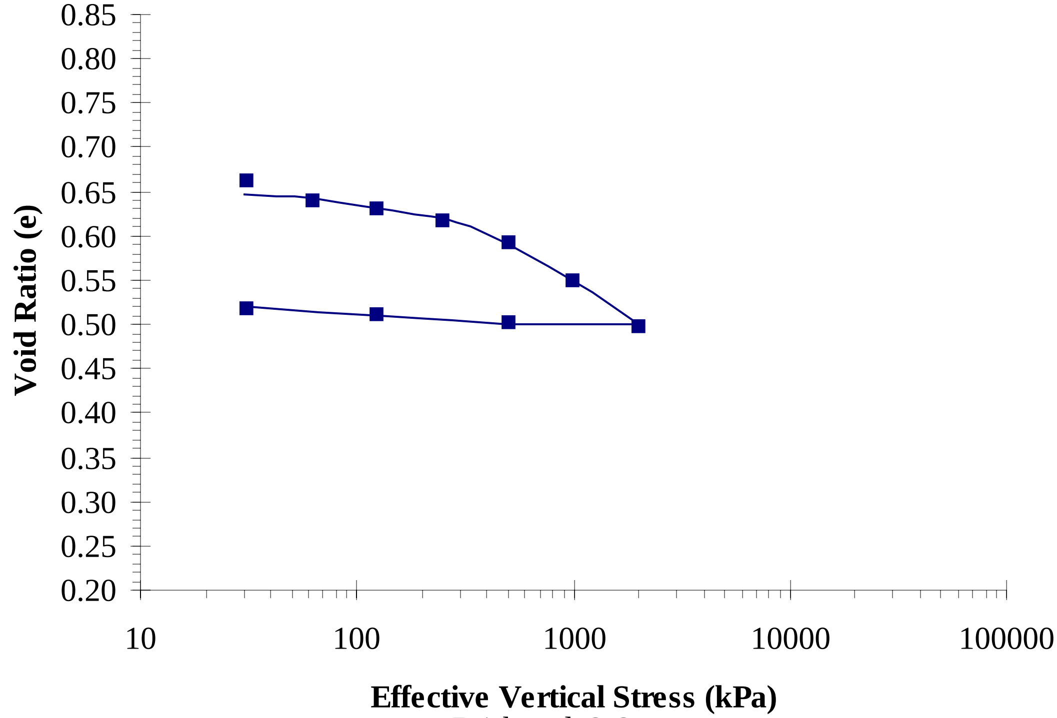

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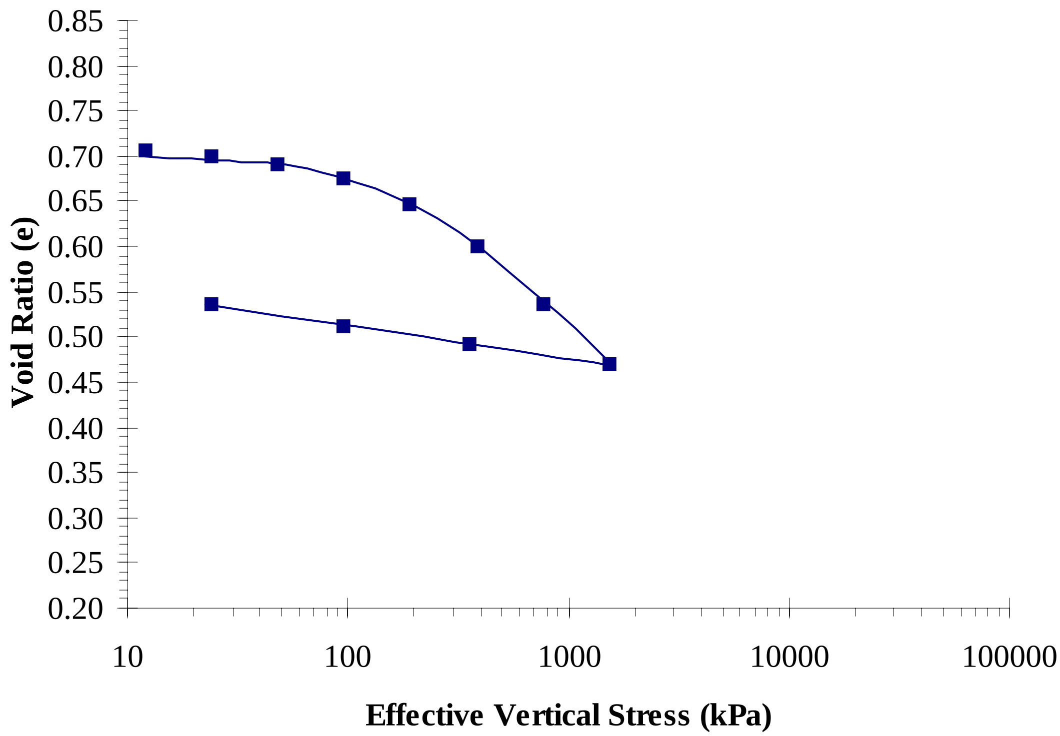

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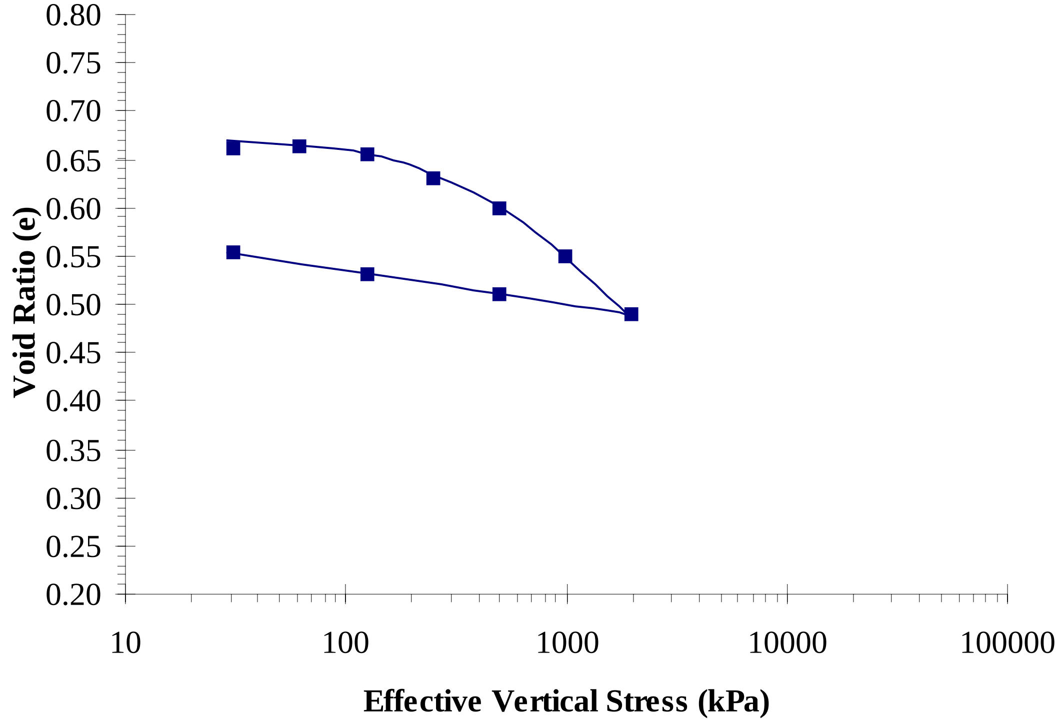

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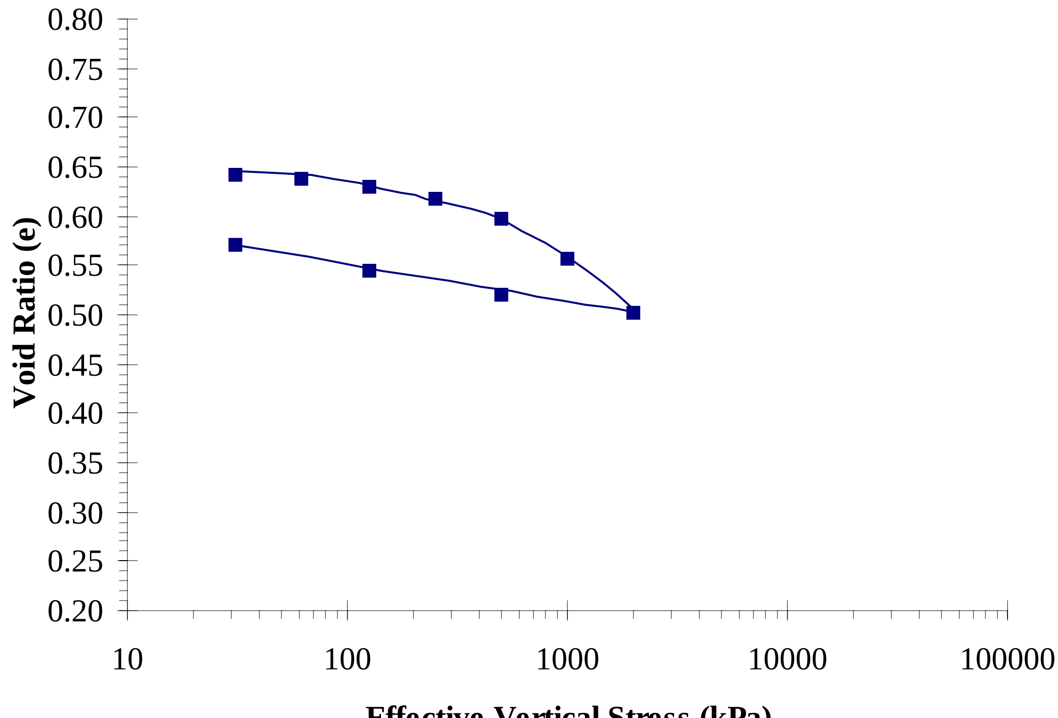

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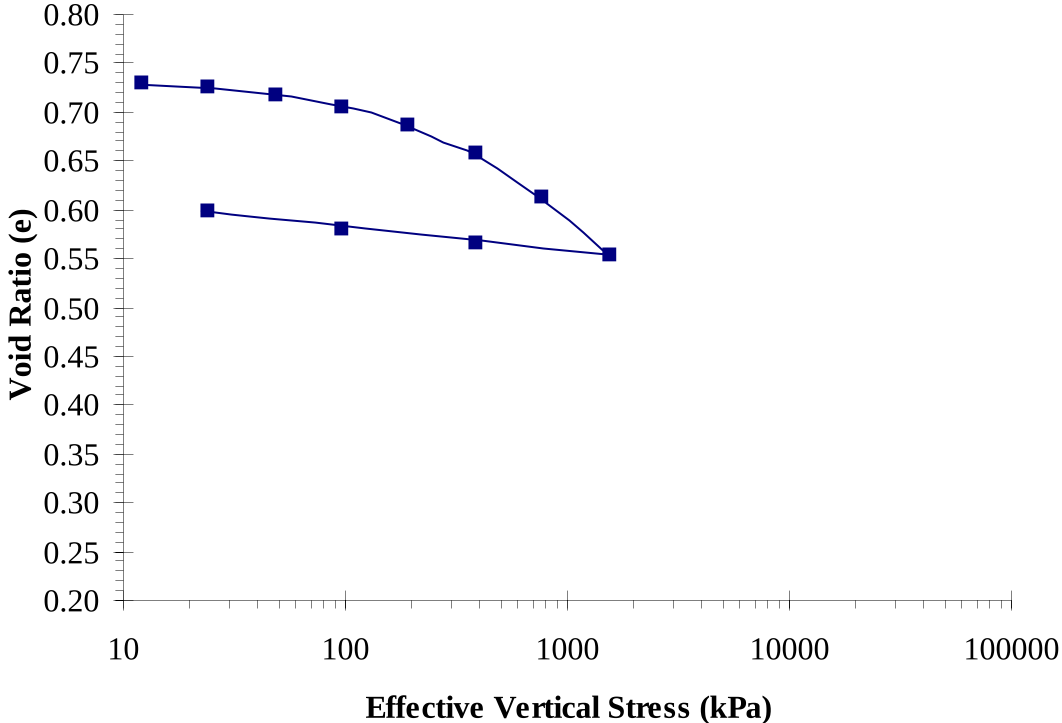

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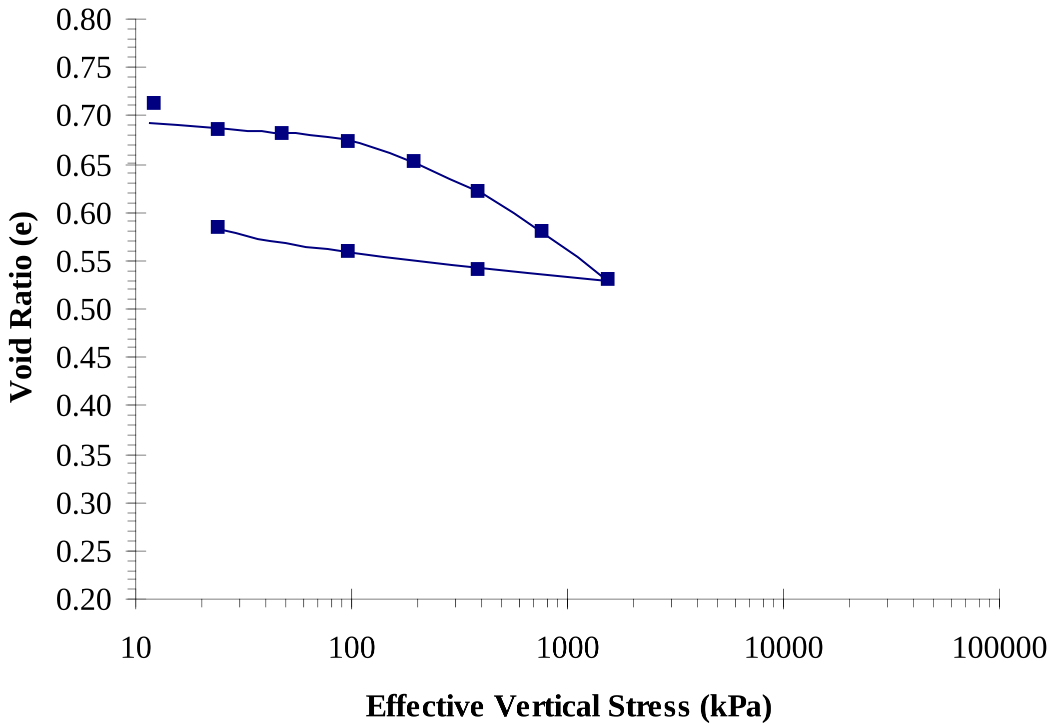

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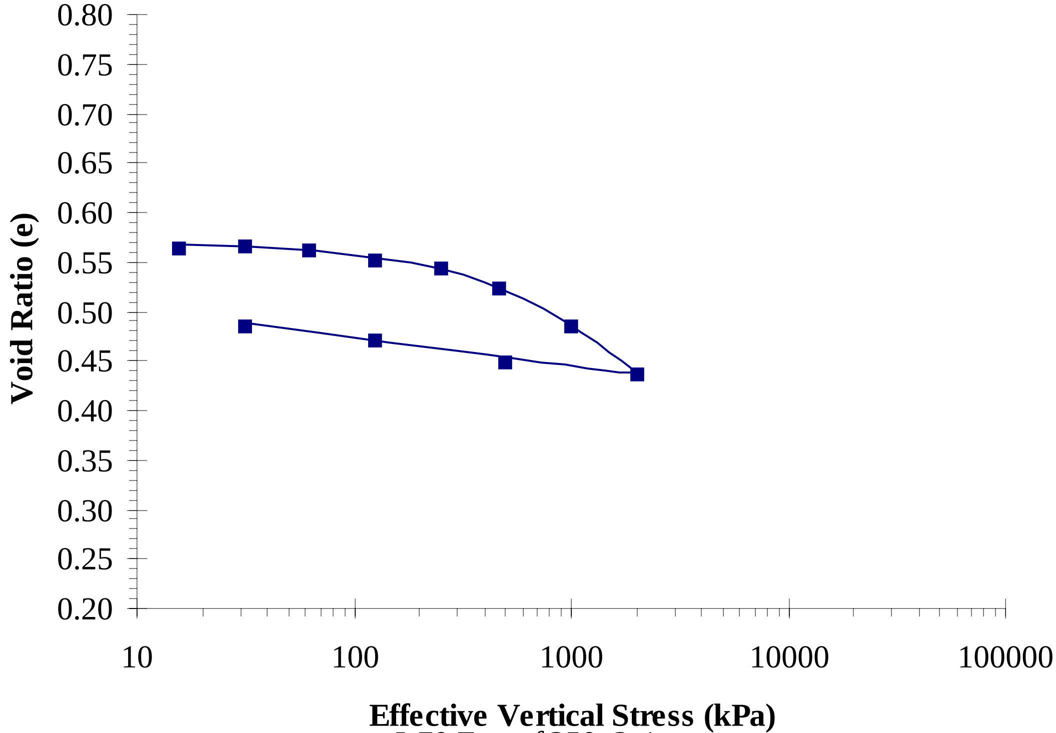

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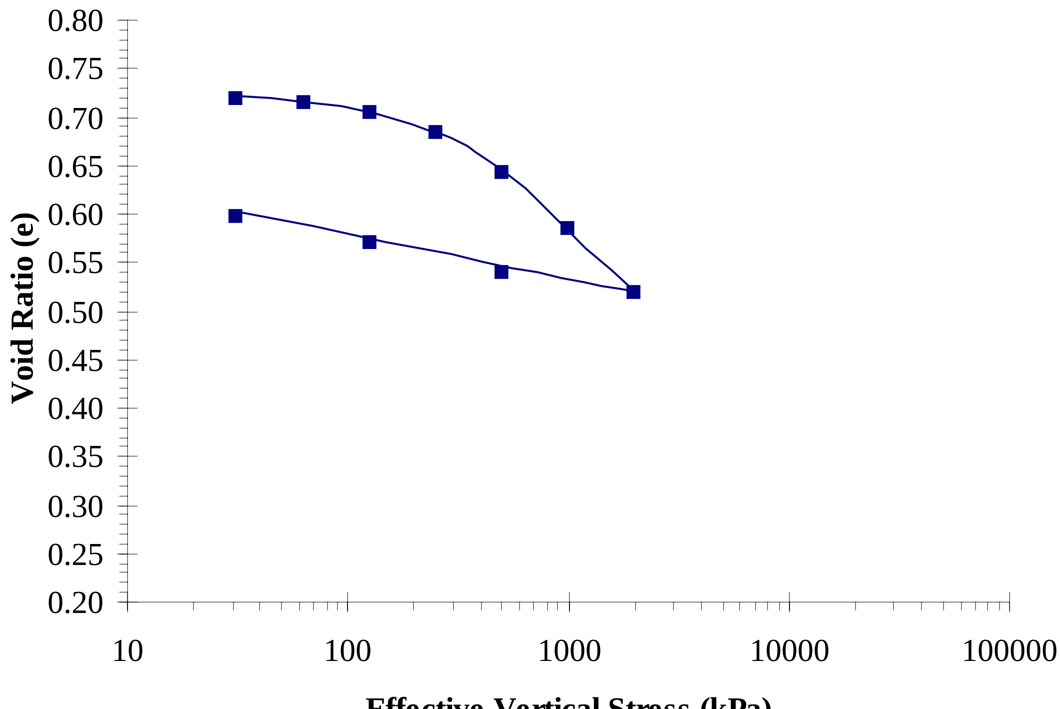

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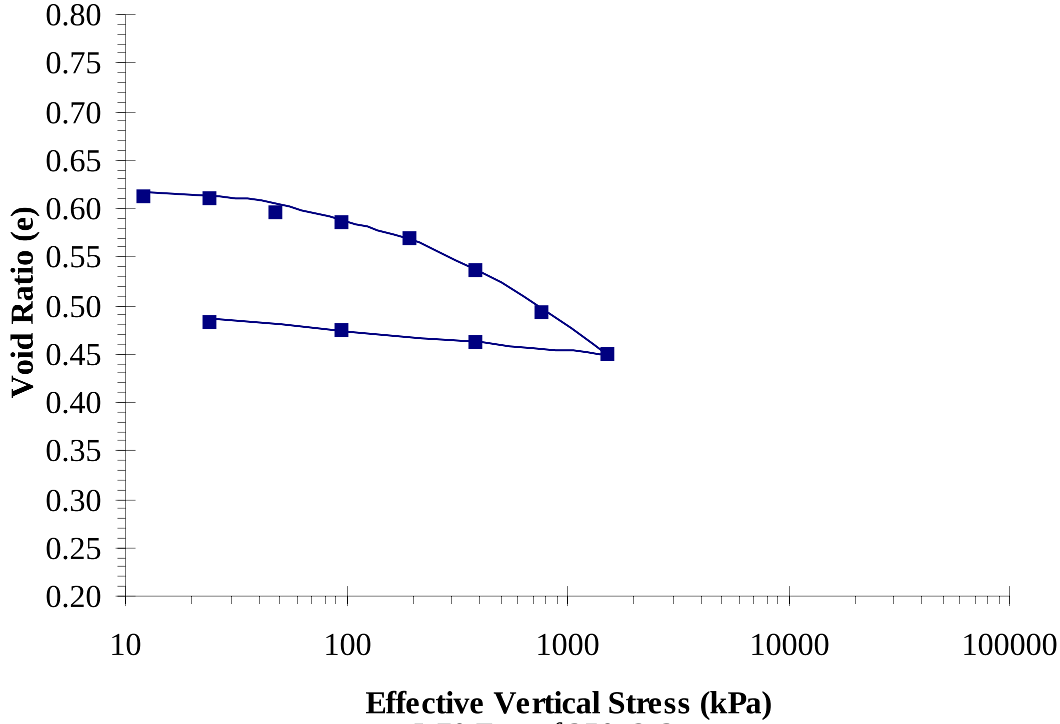

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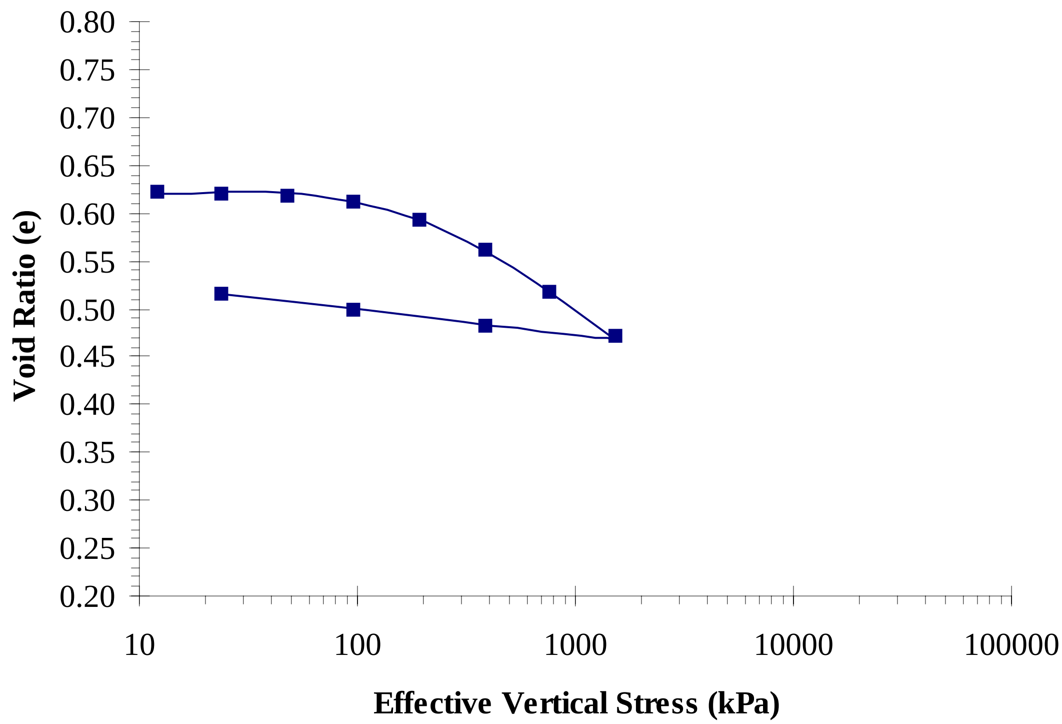

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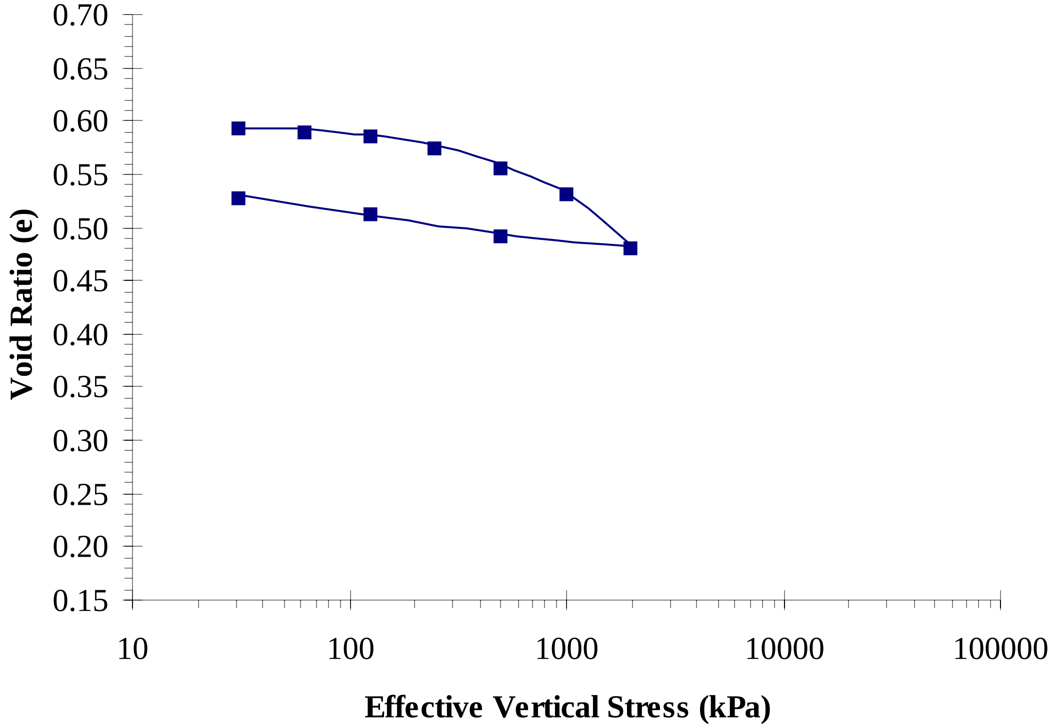

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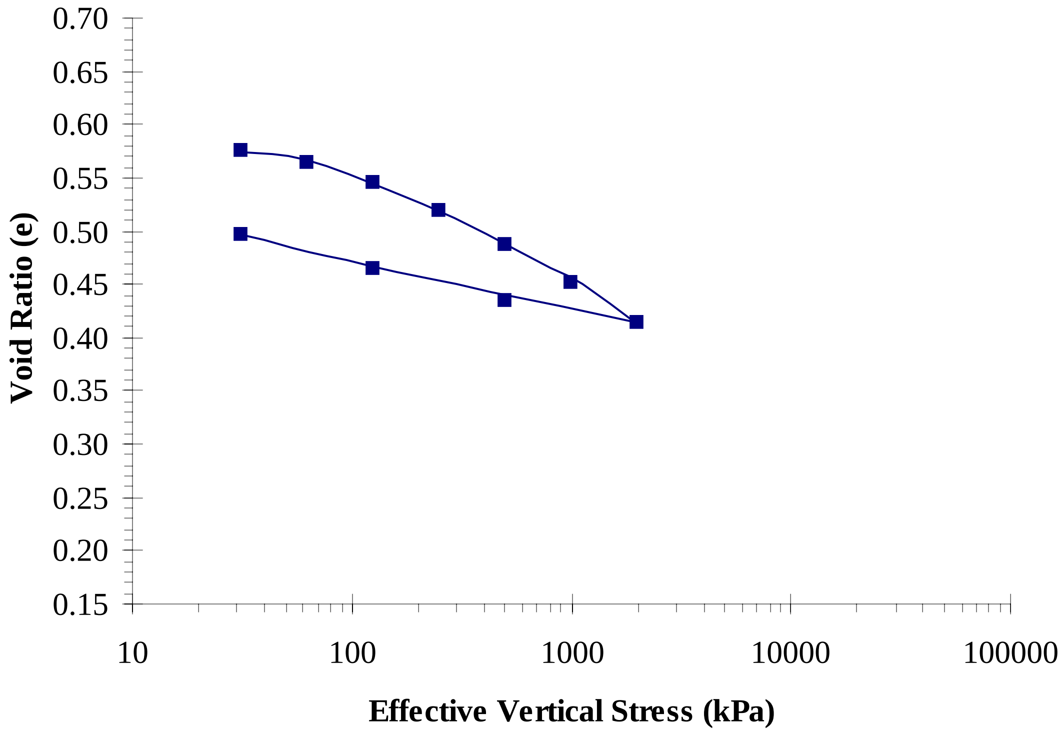

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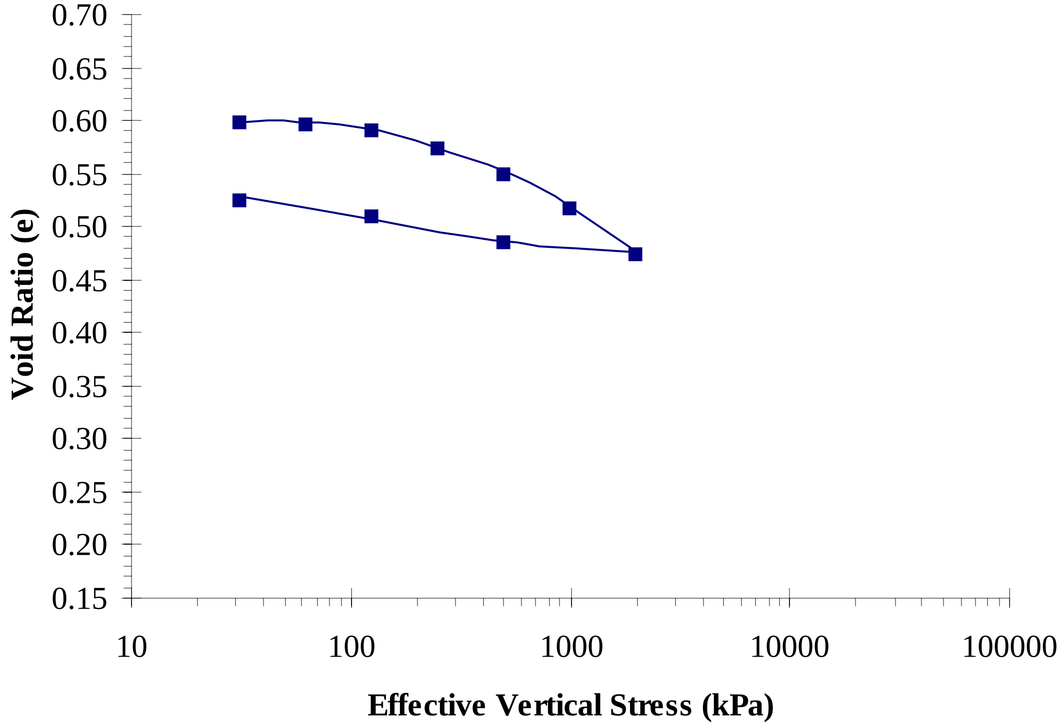

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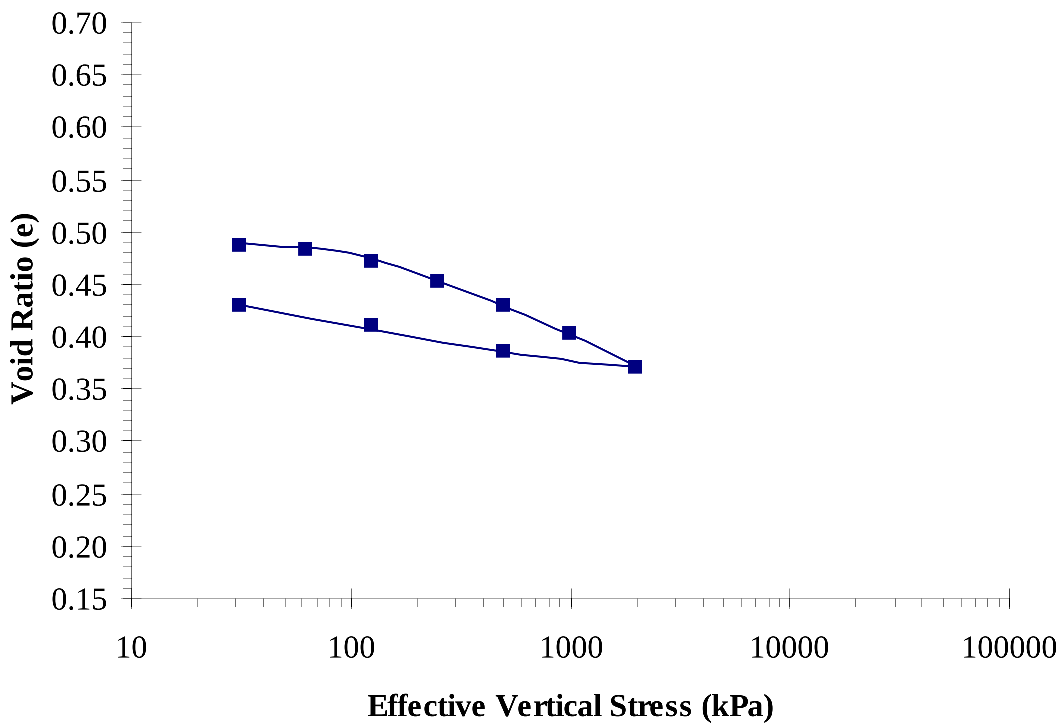

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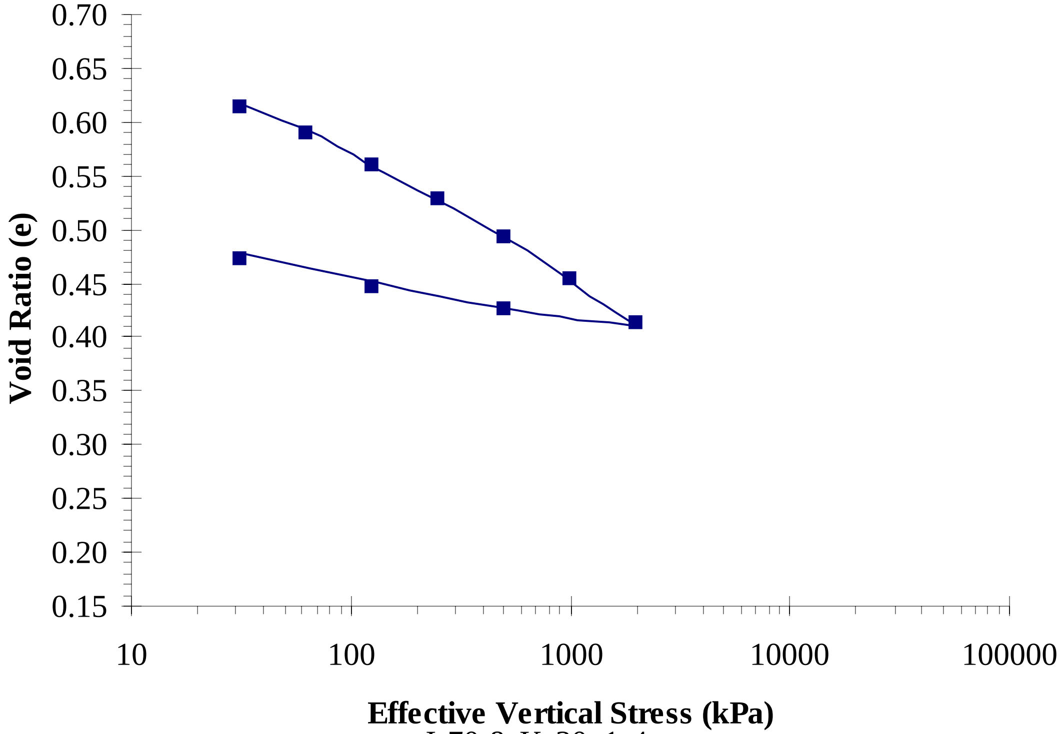

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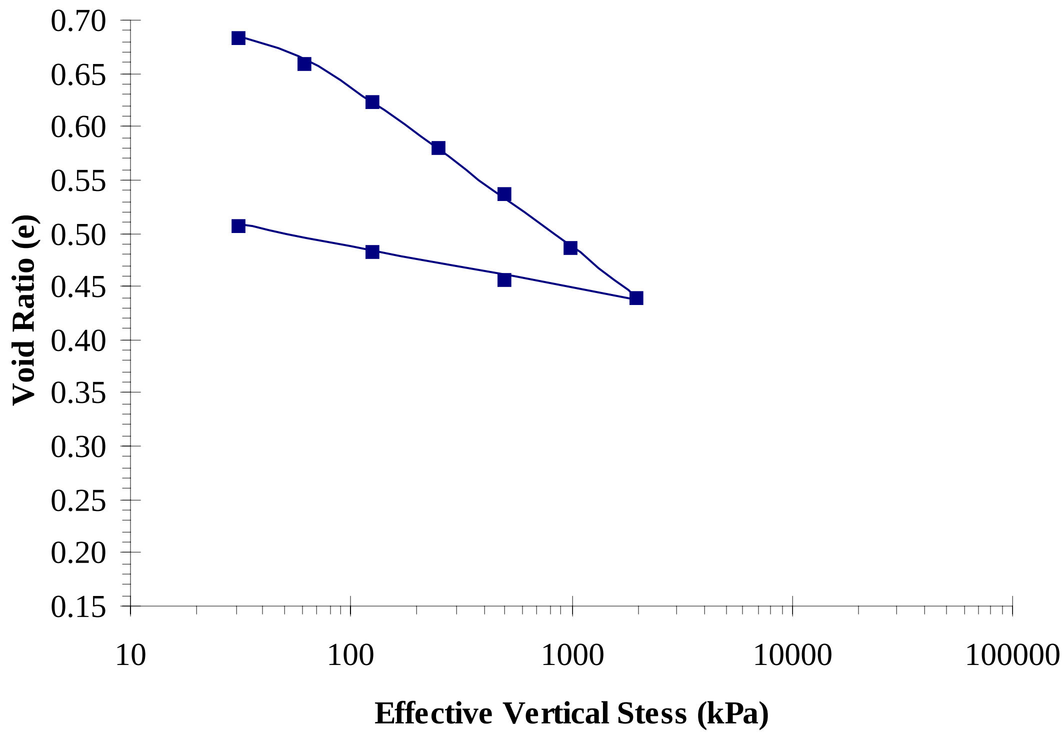

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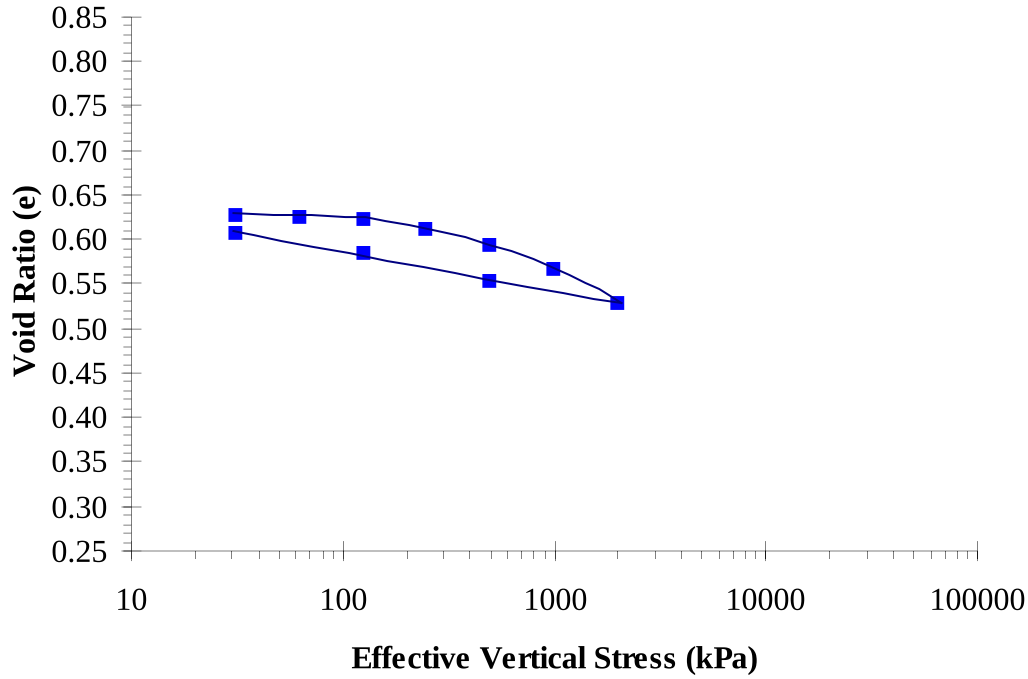

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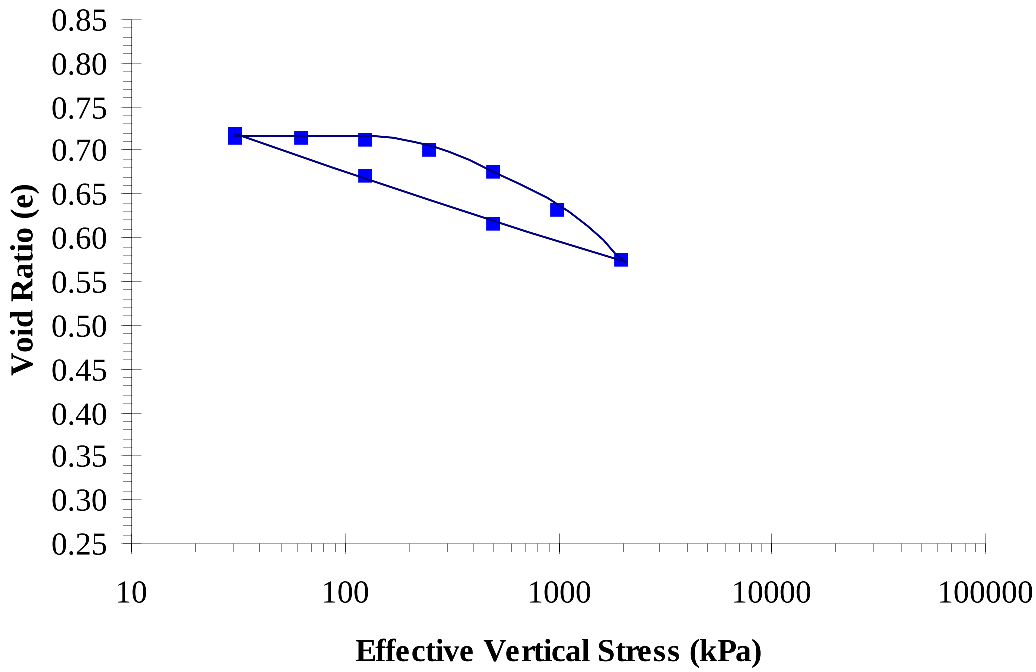

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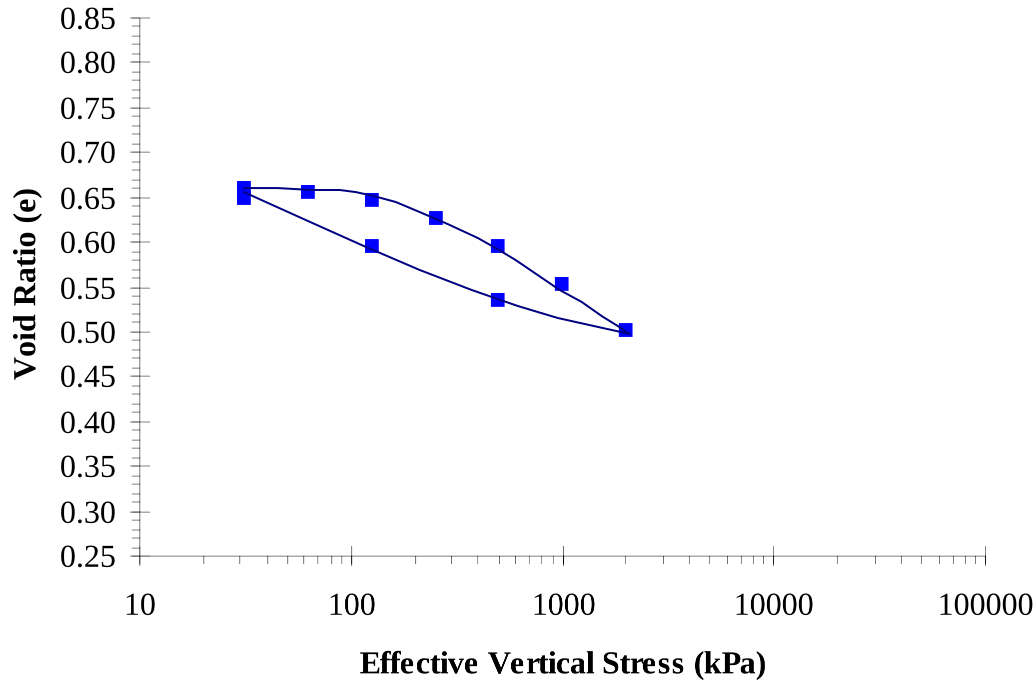

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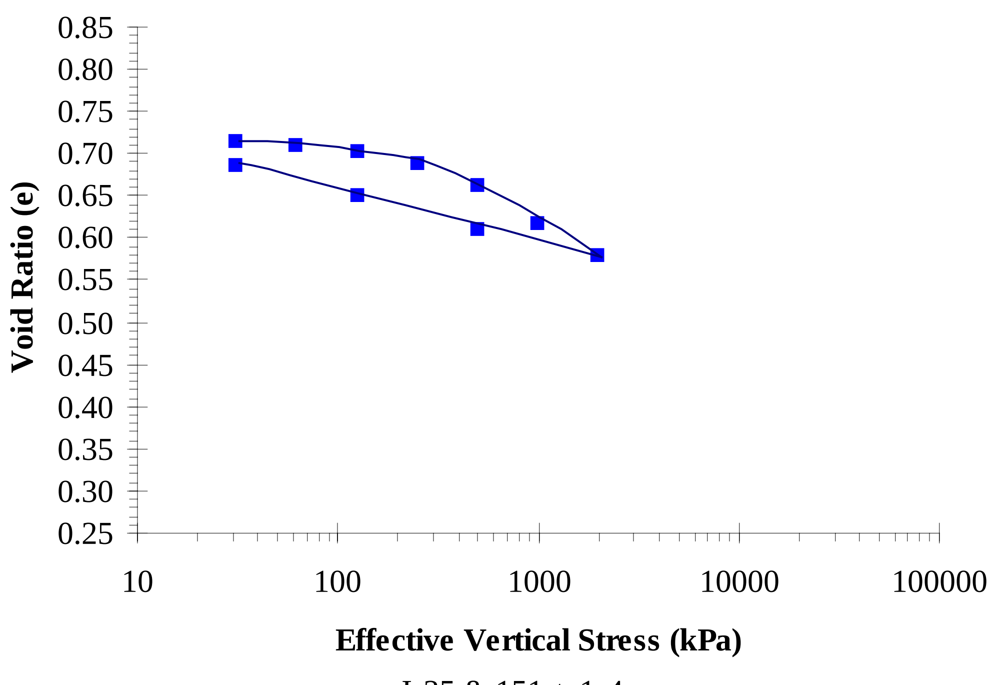

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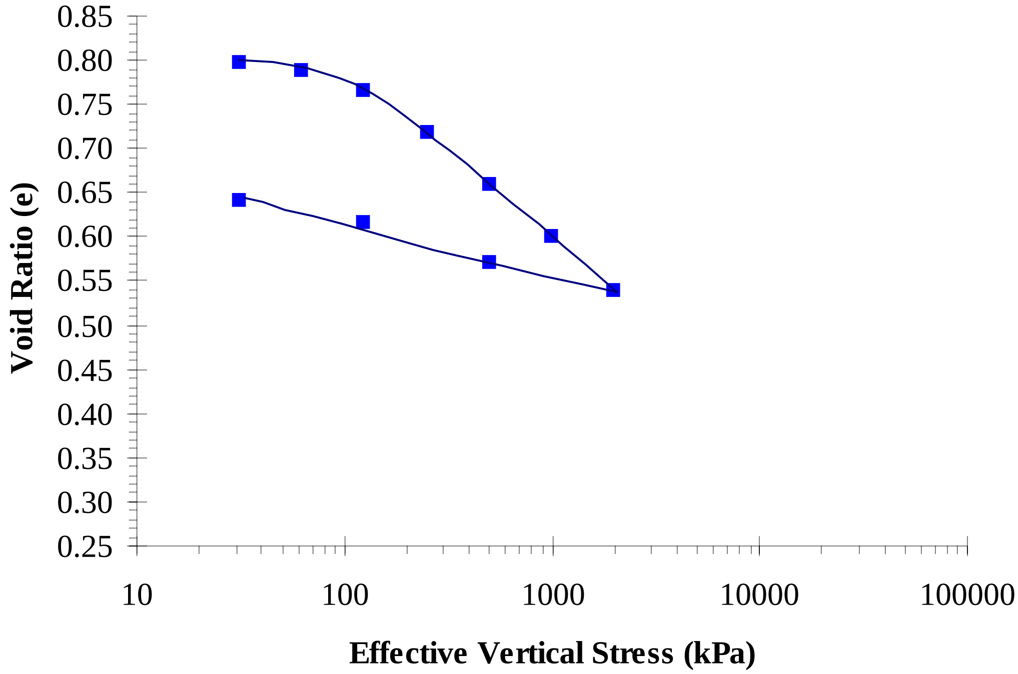

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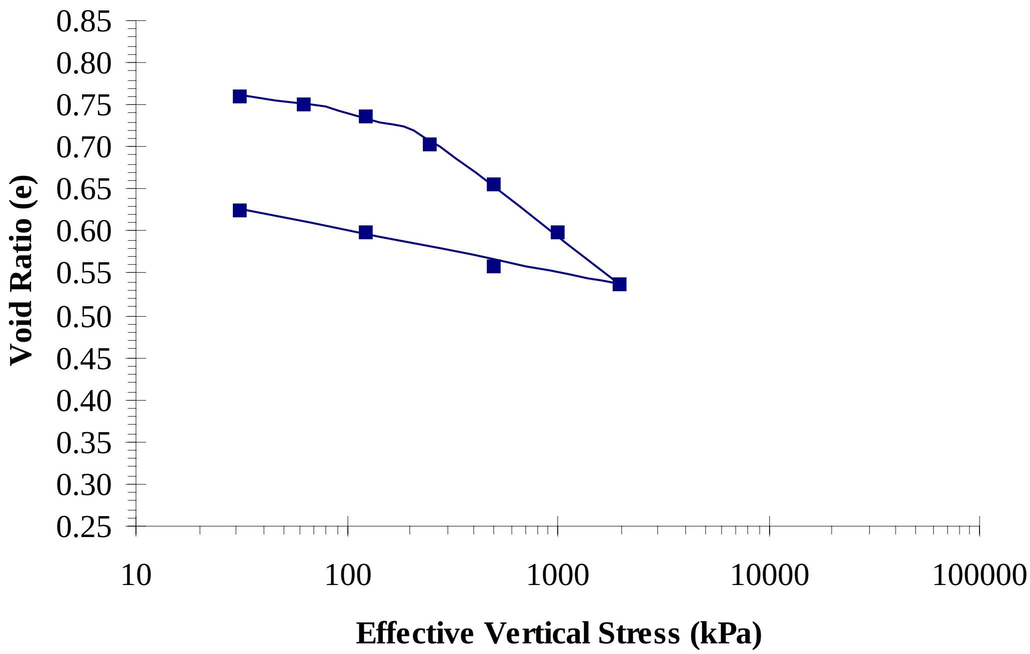

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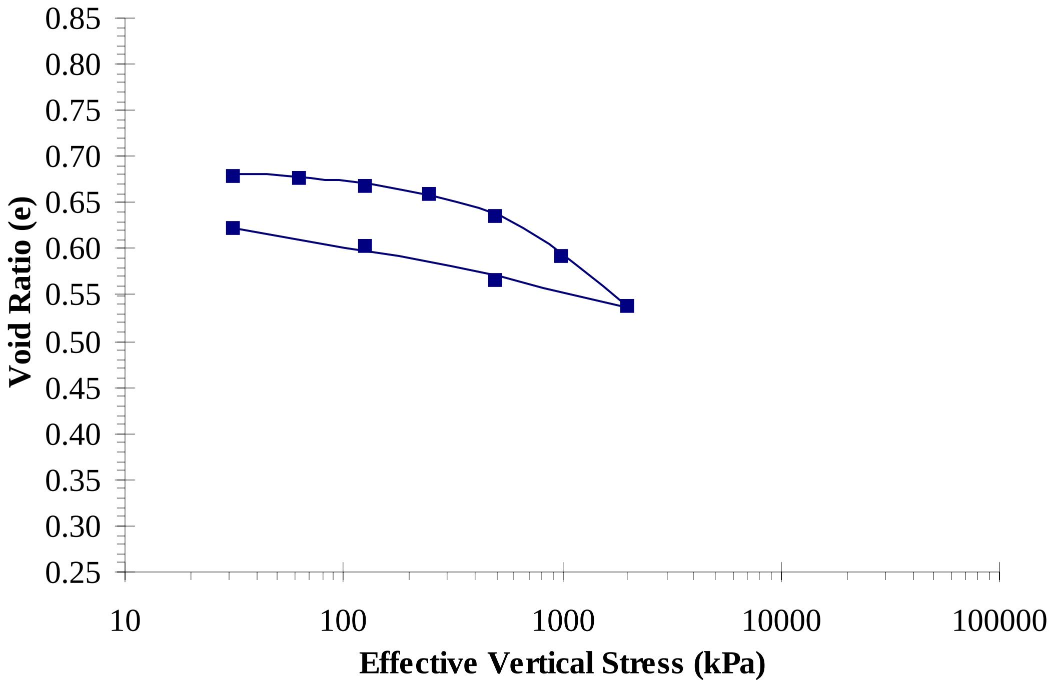

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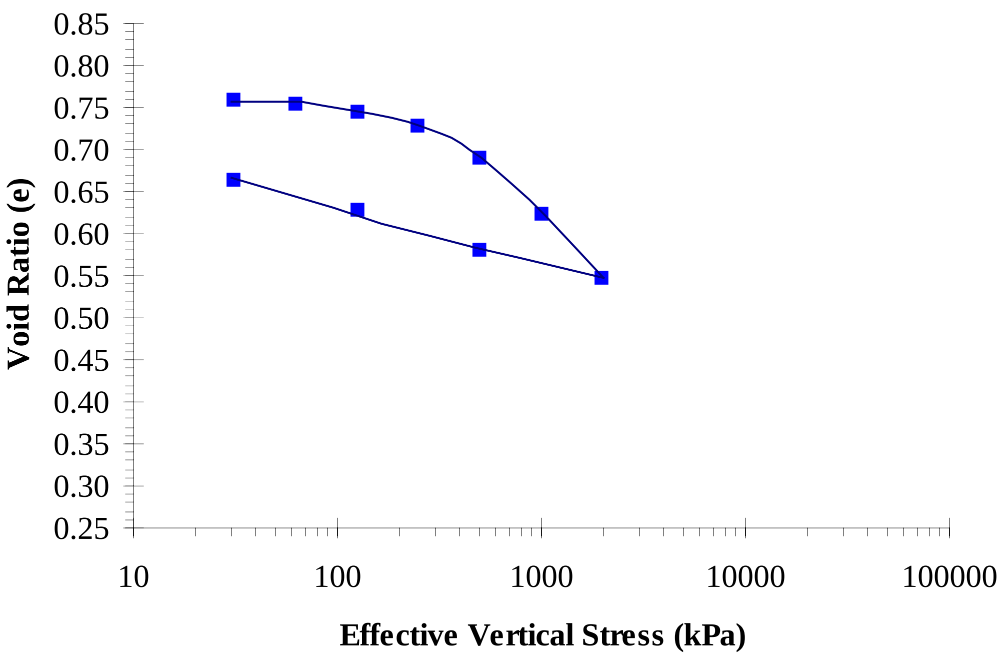

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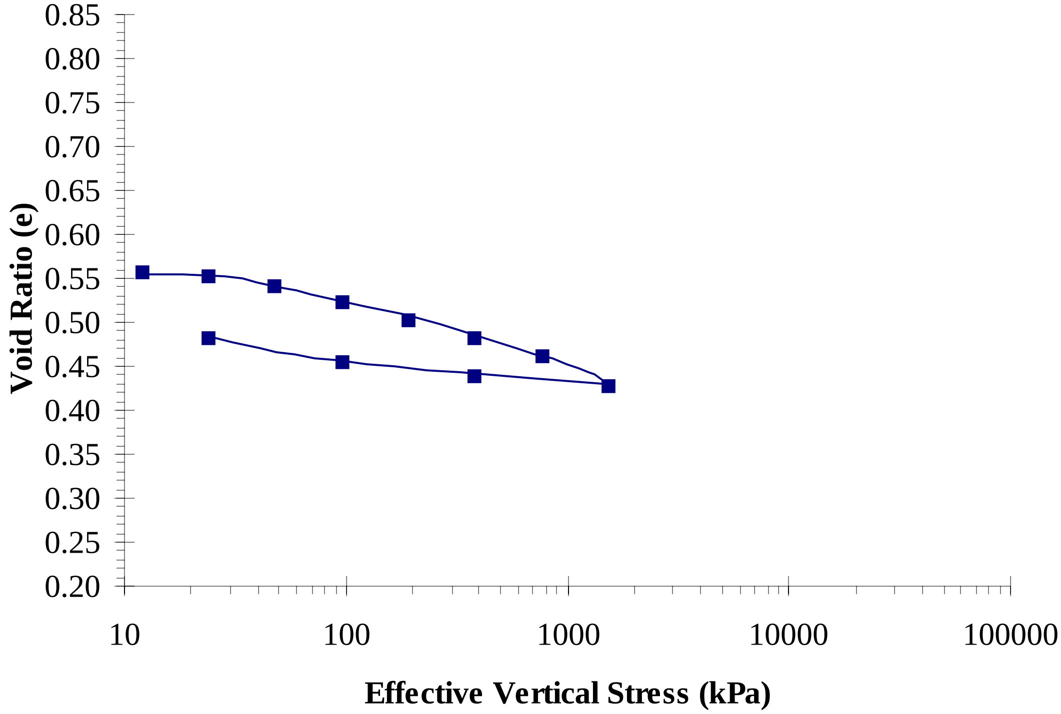

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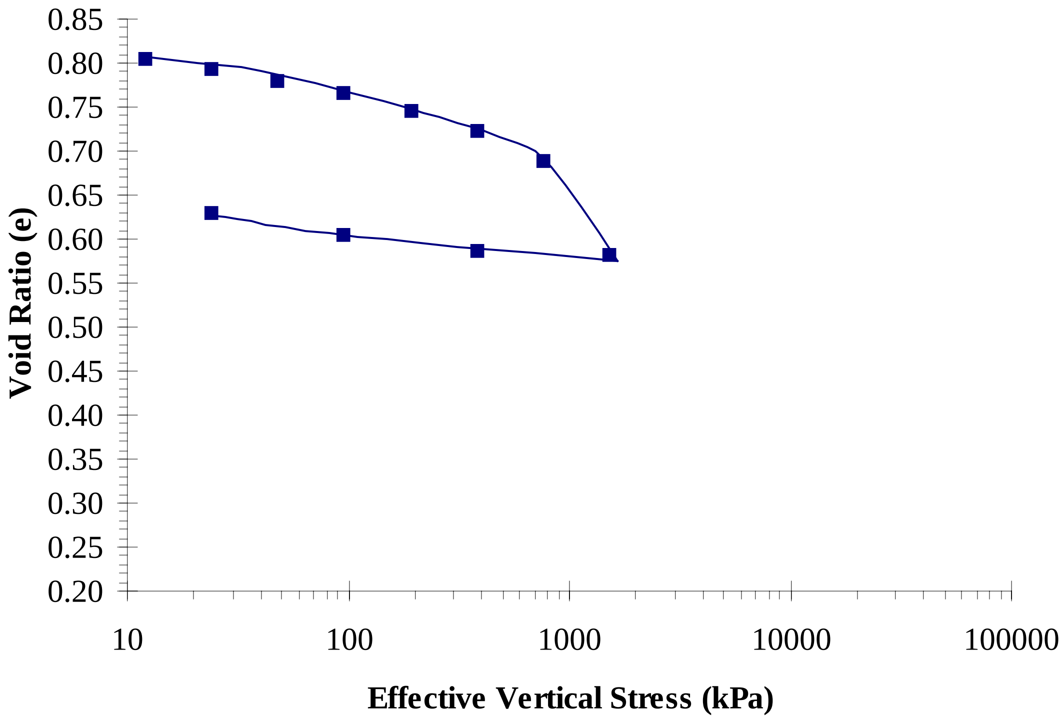

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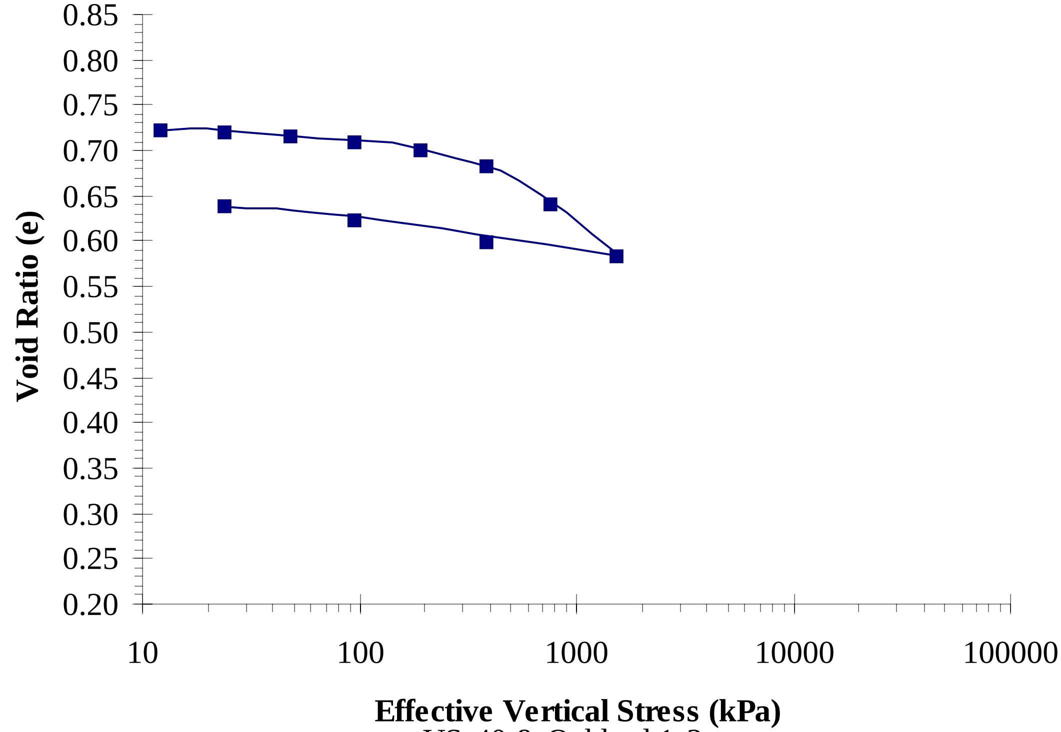

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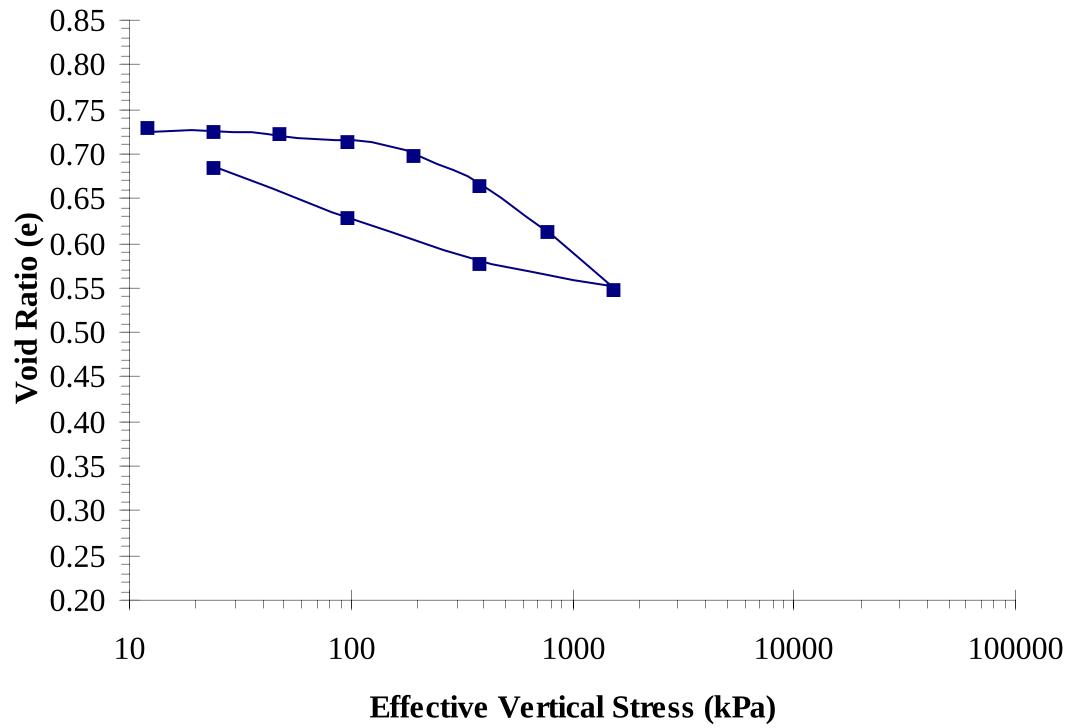

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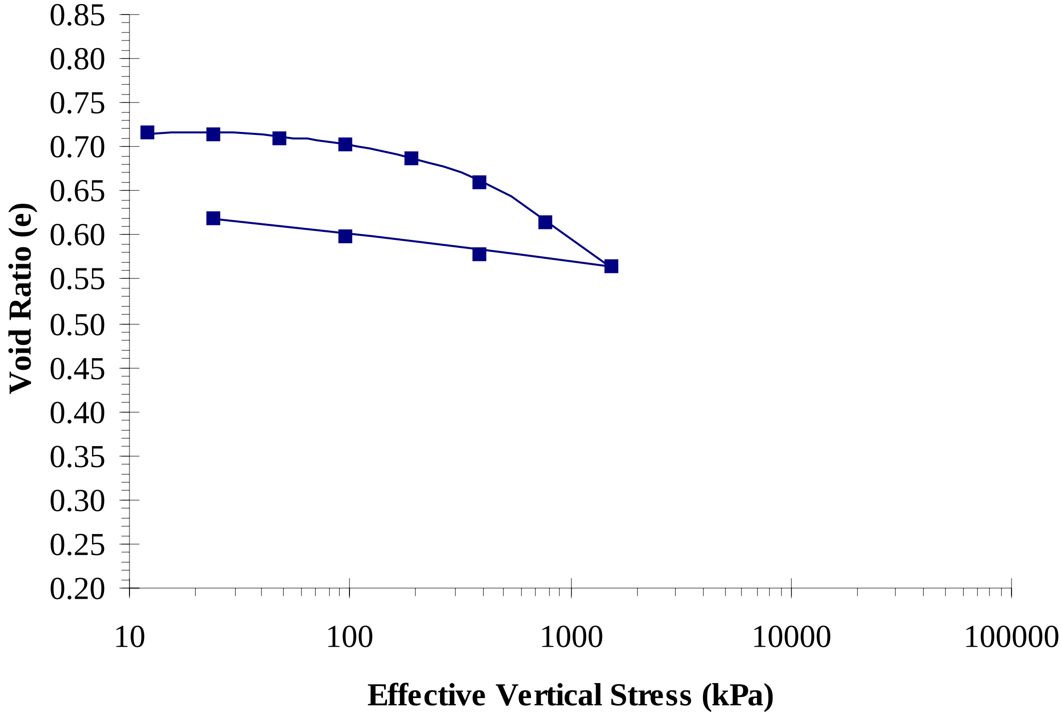

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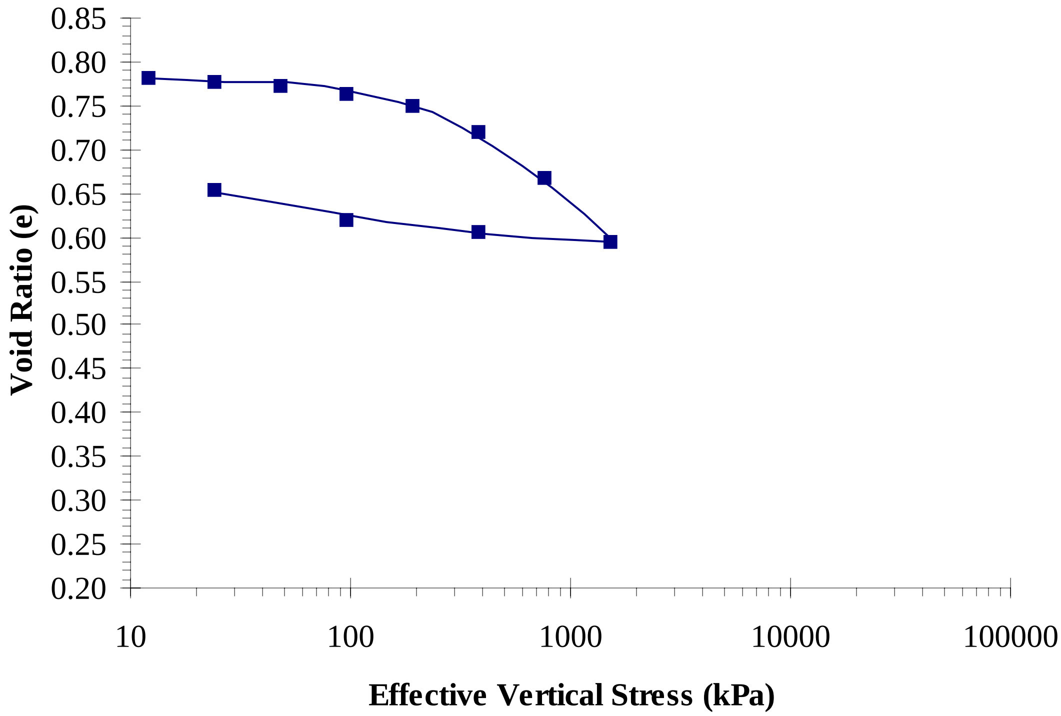

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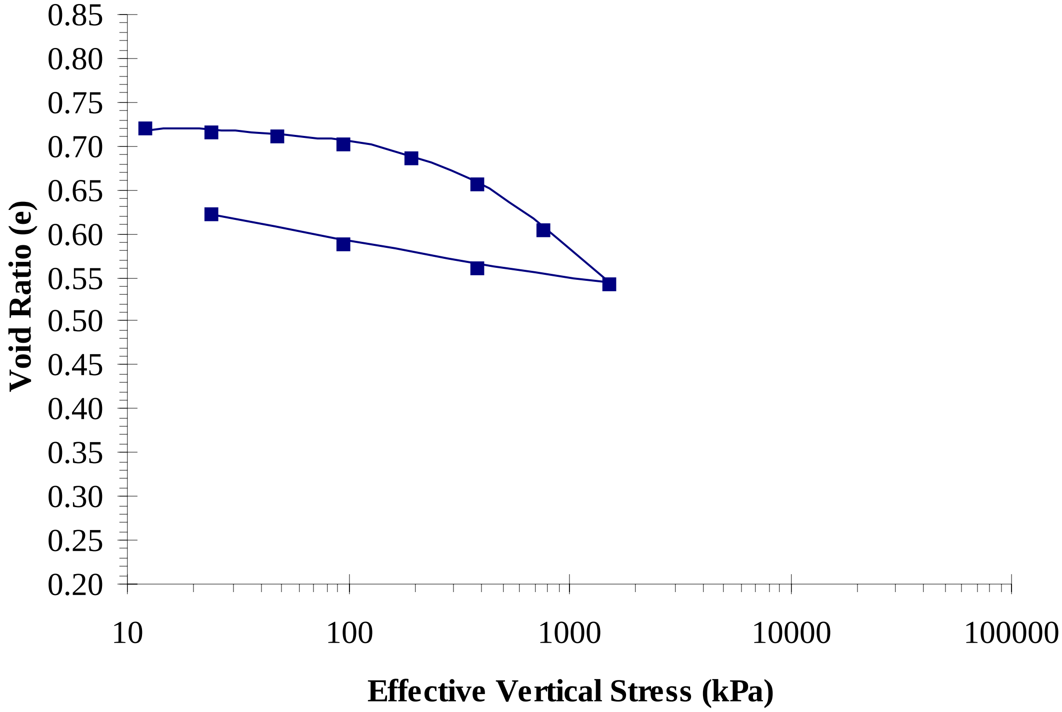

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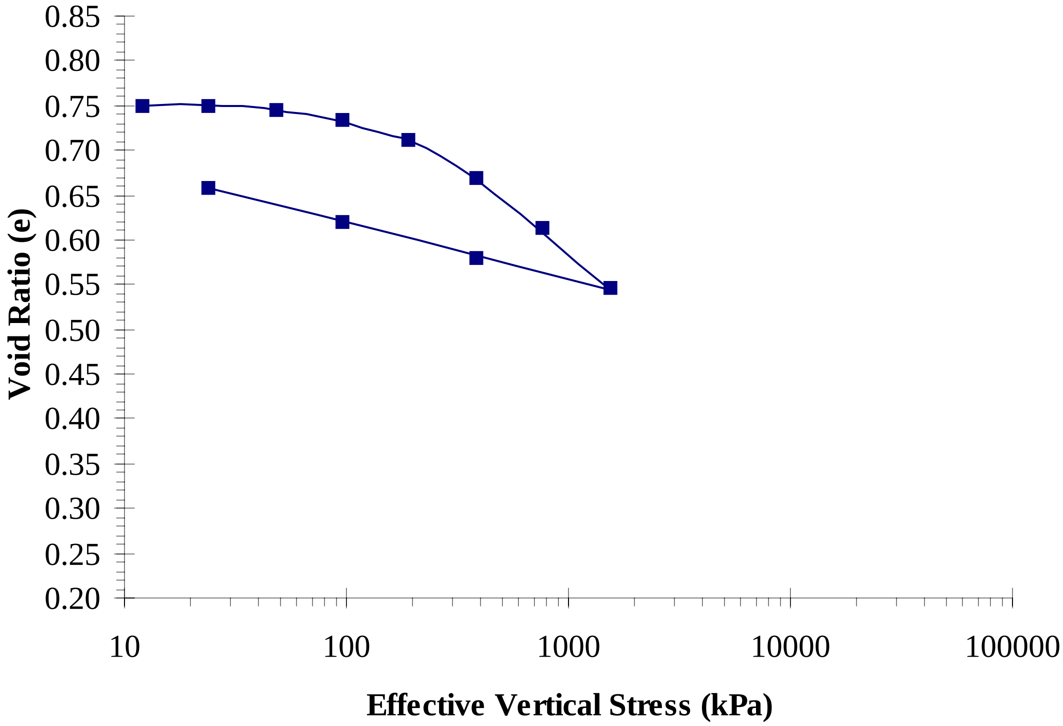

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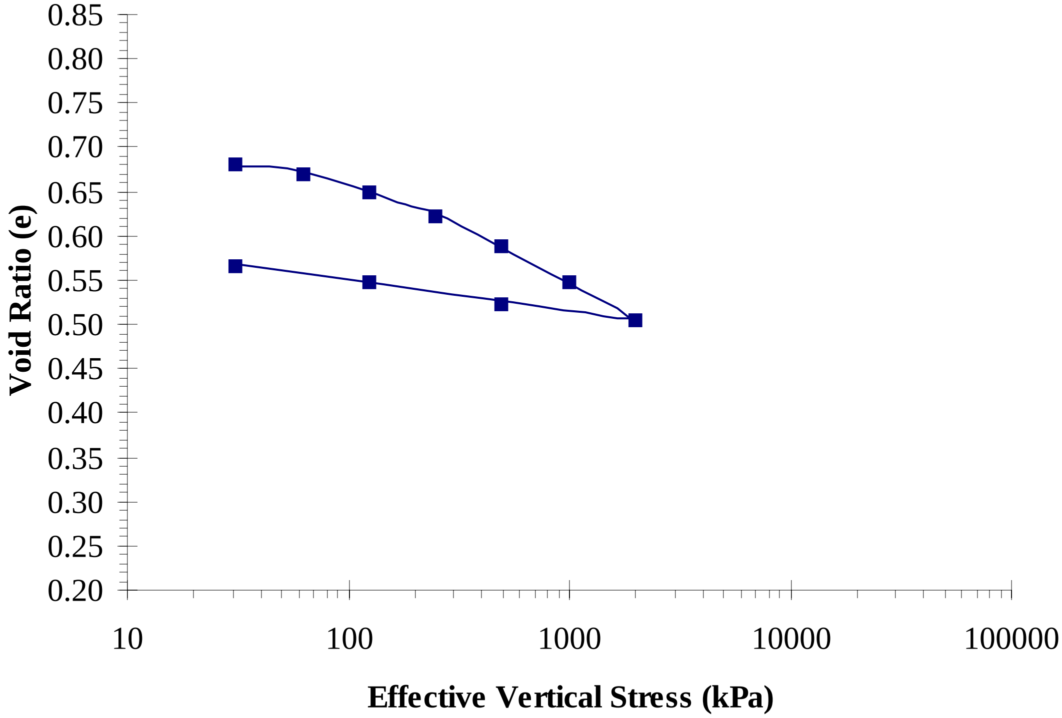

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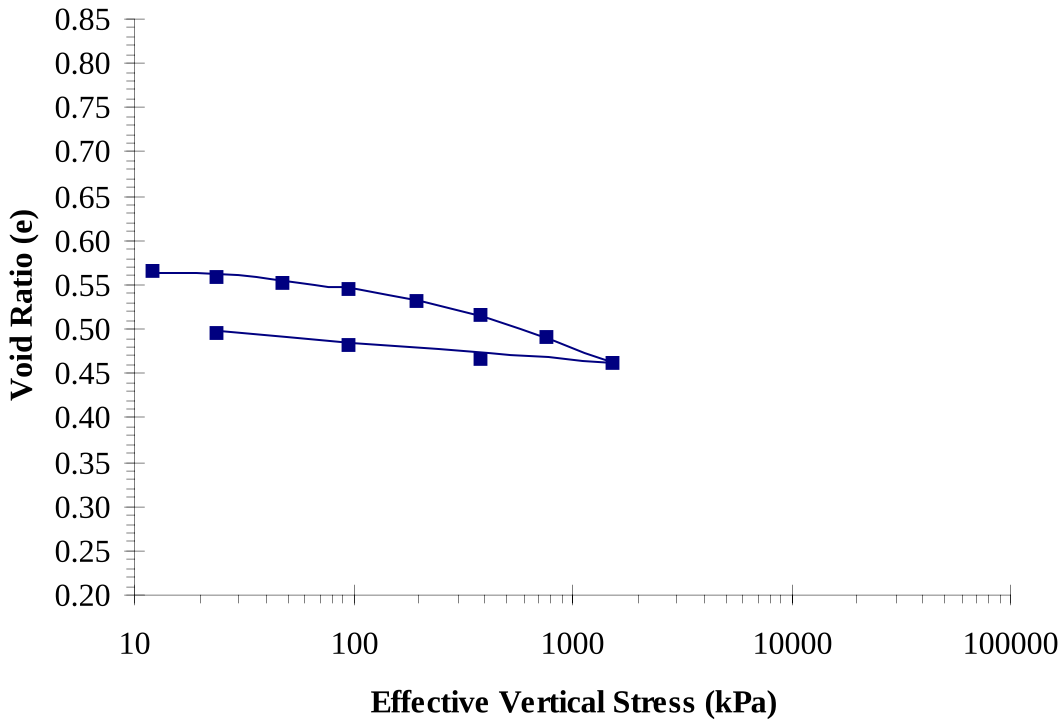

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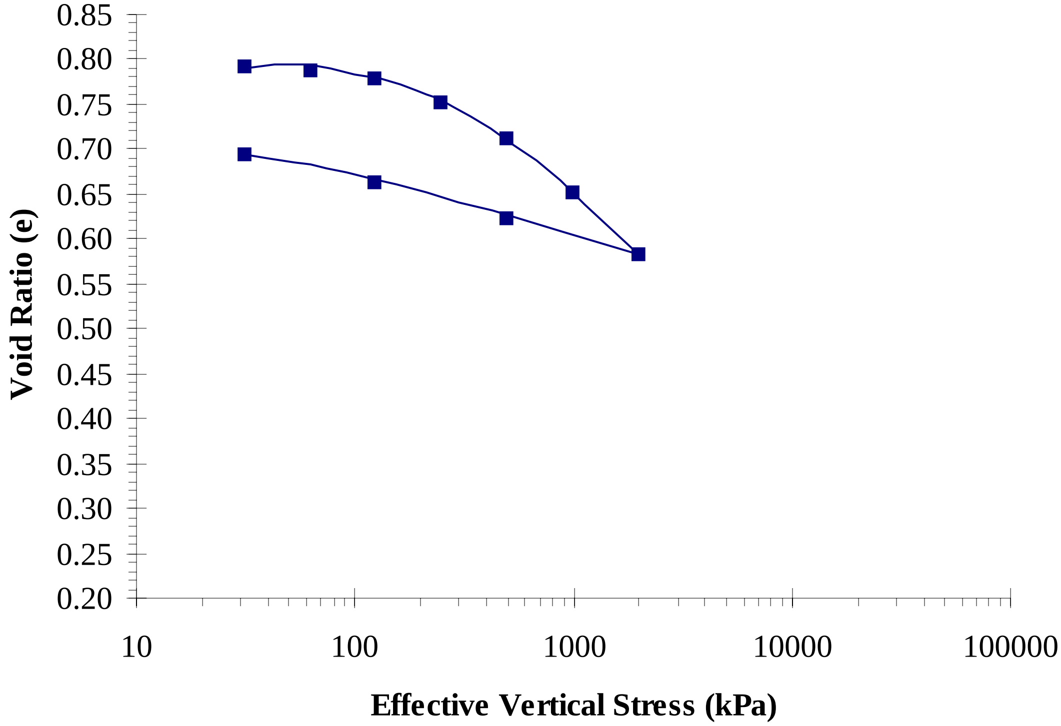

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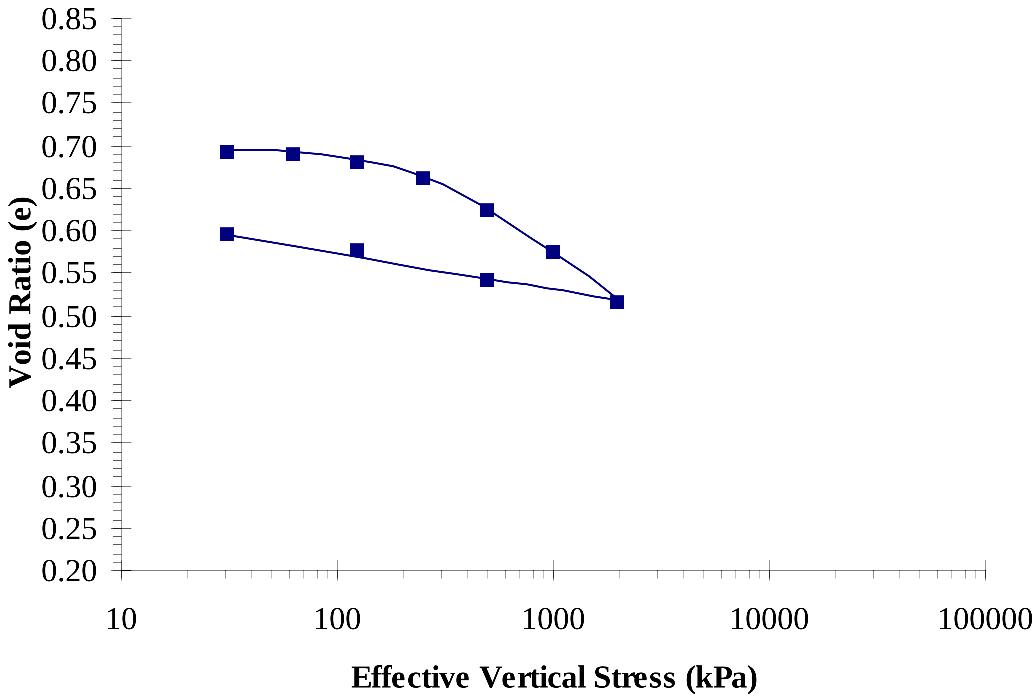

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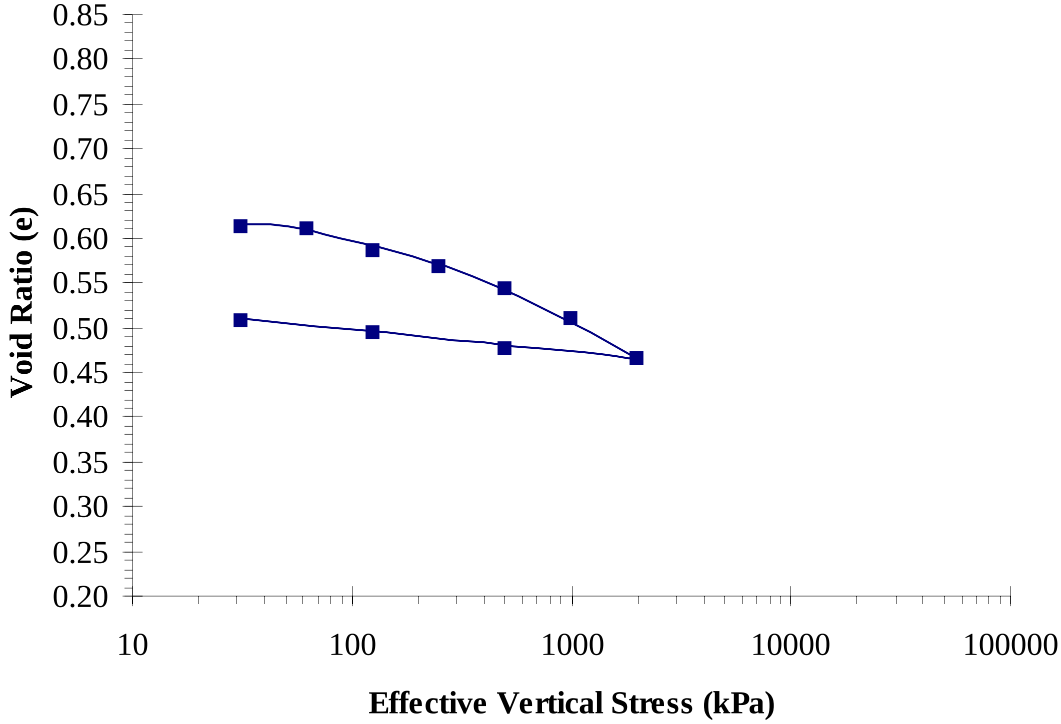
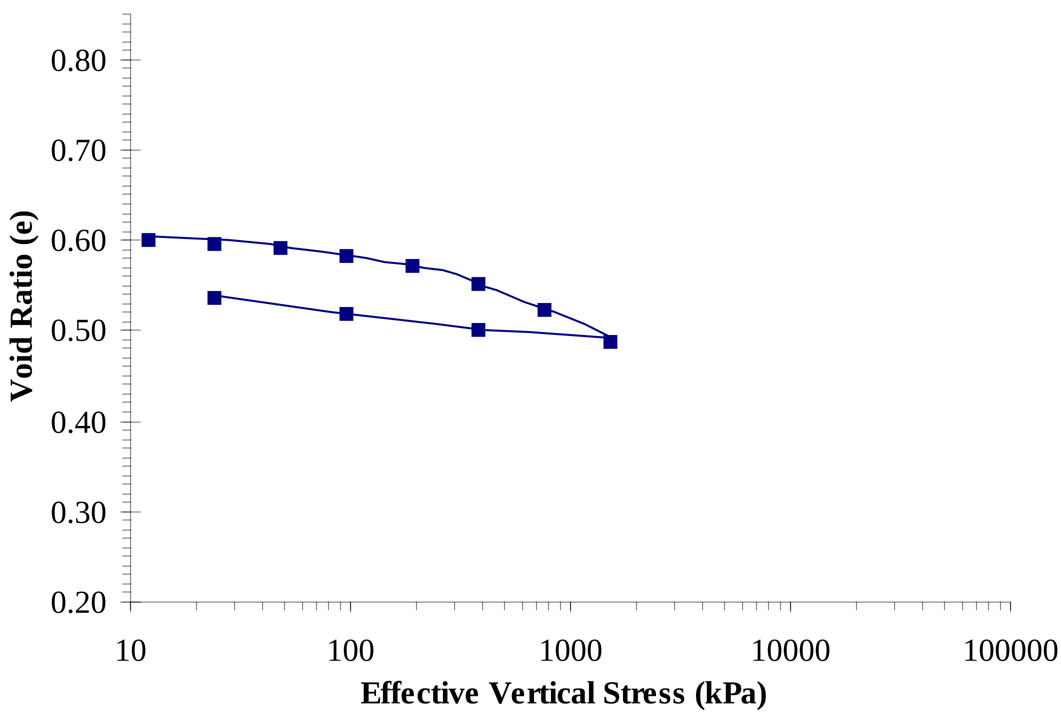


US-59 & K-10 2-1



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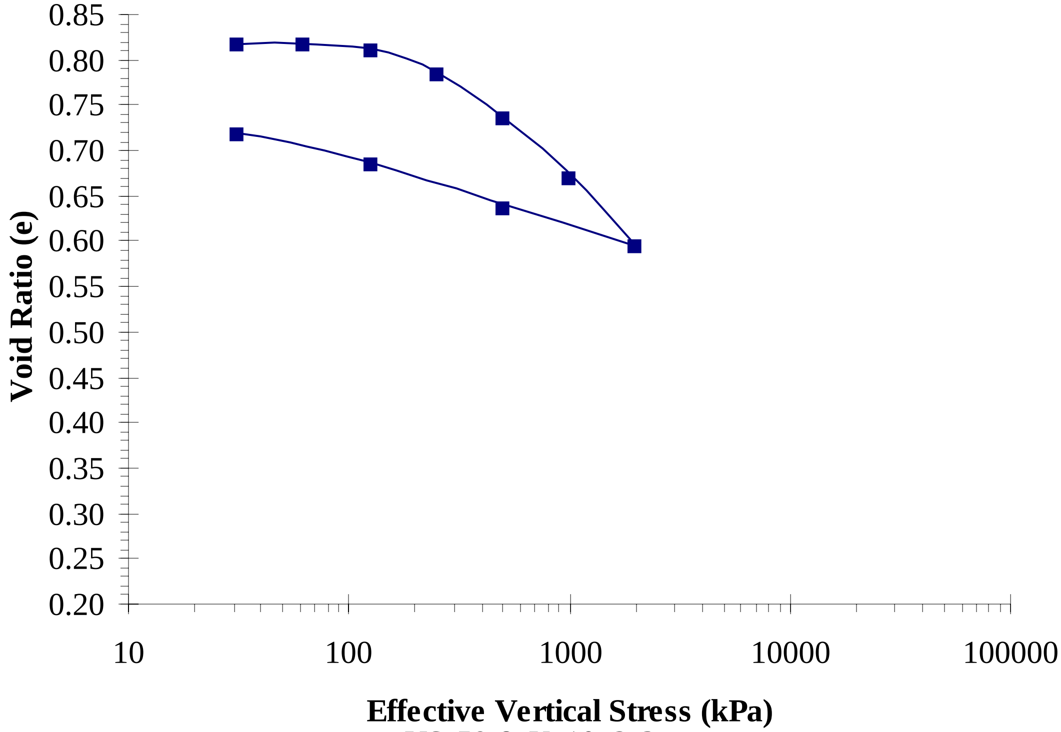

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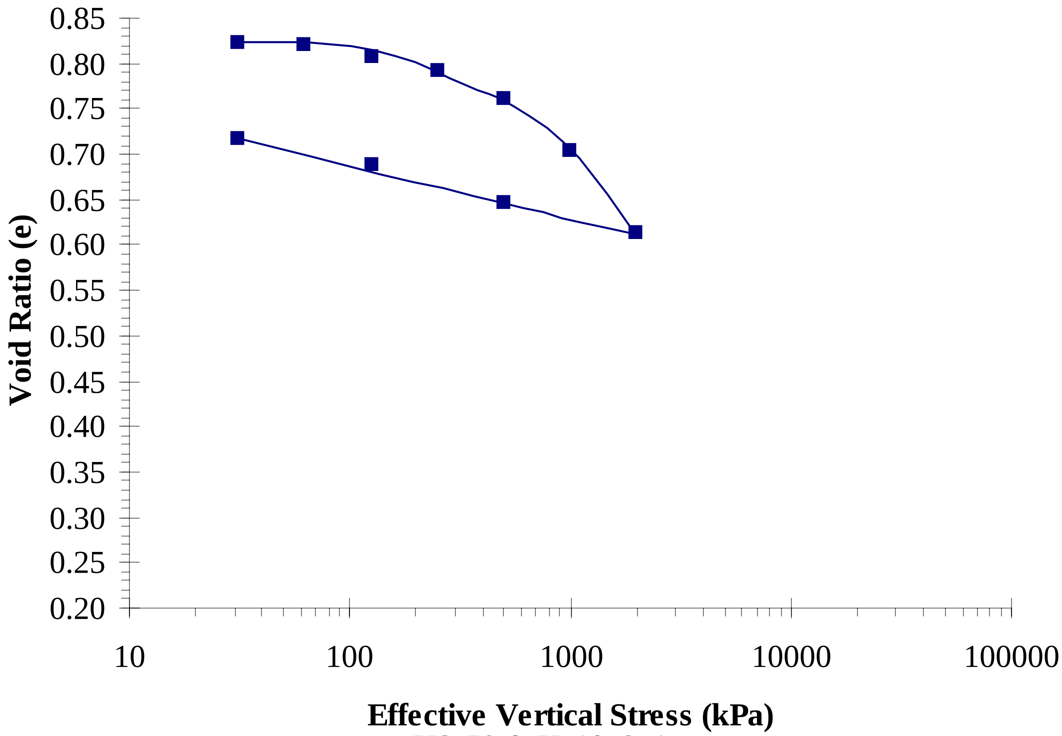

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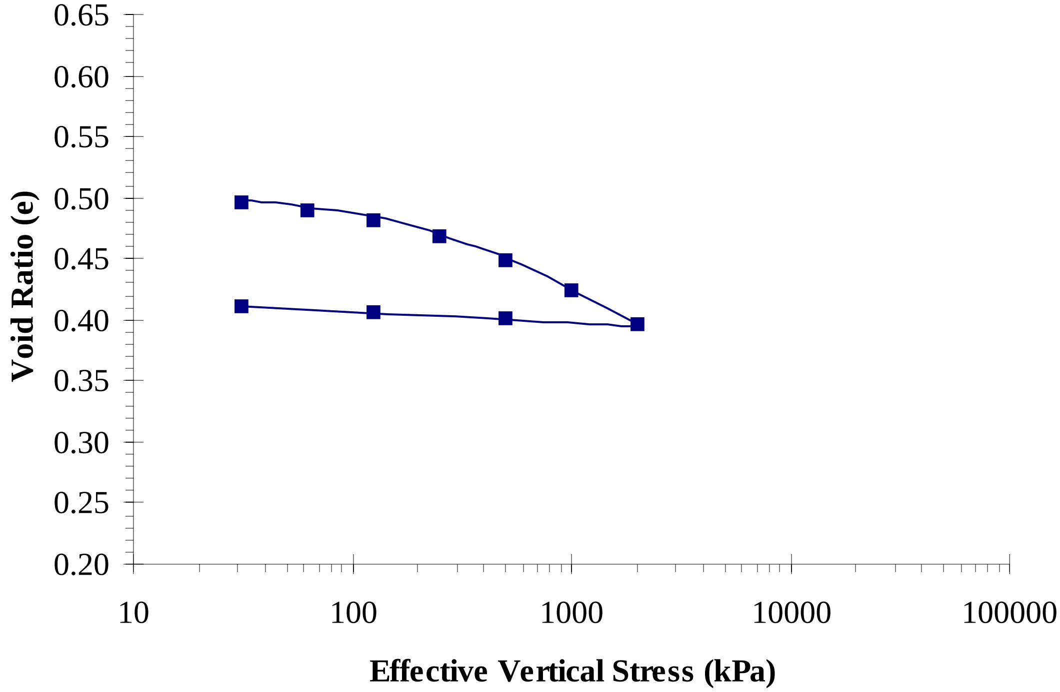
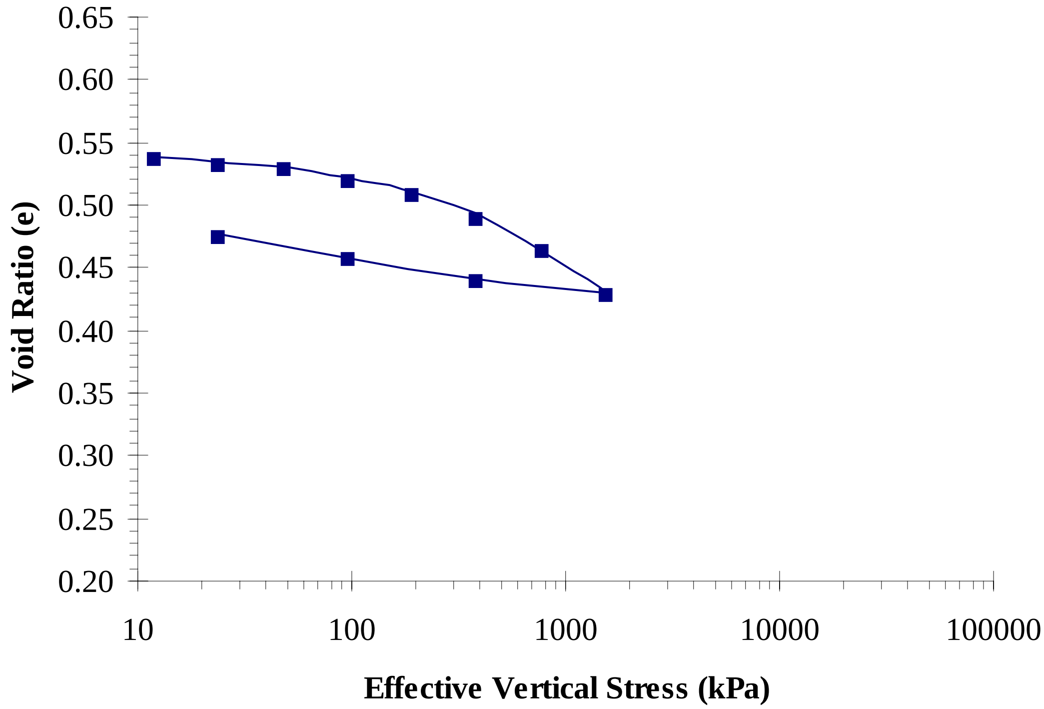


I-35 & Antioch 1-1



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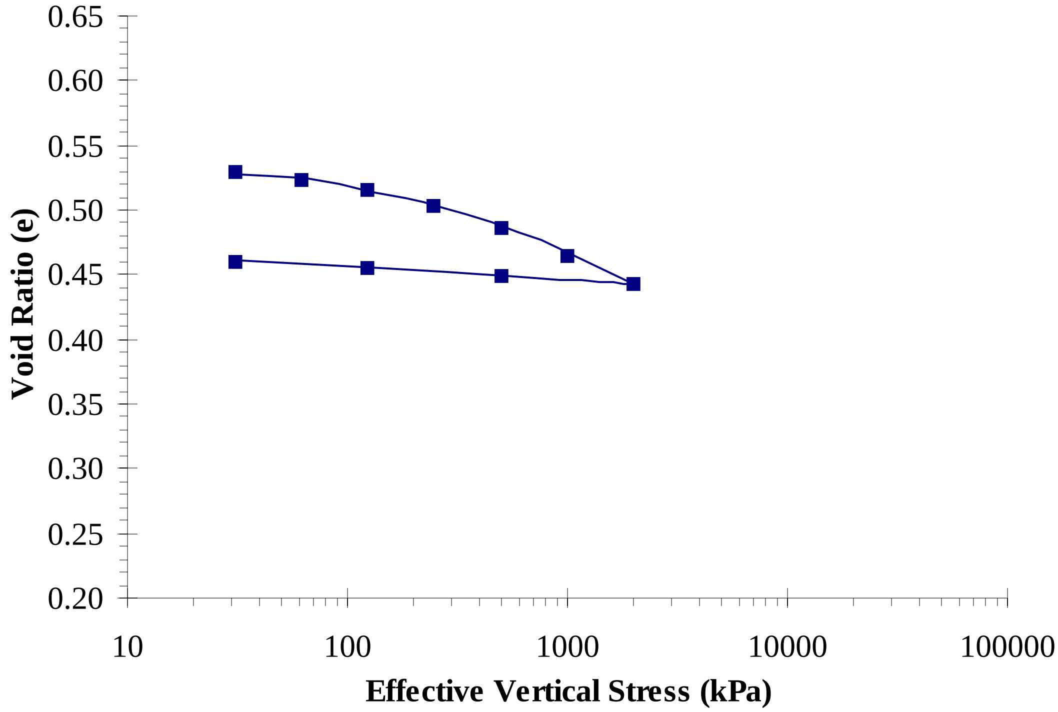

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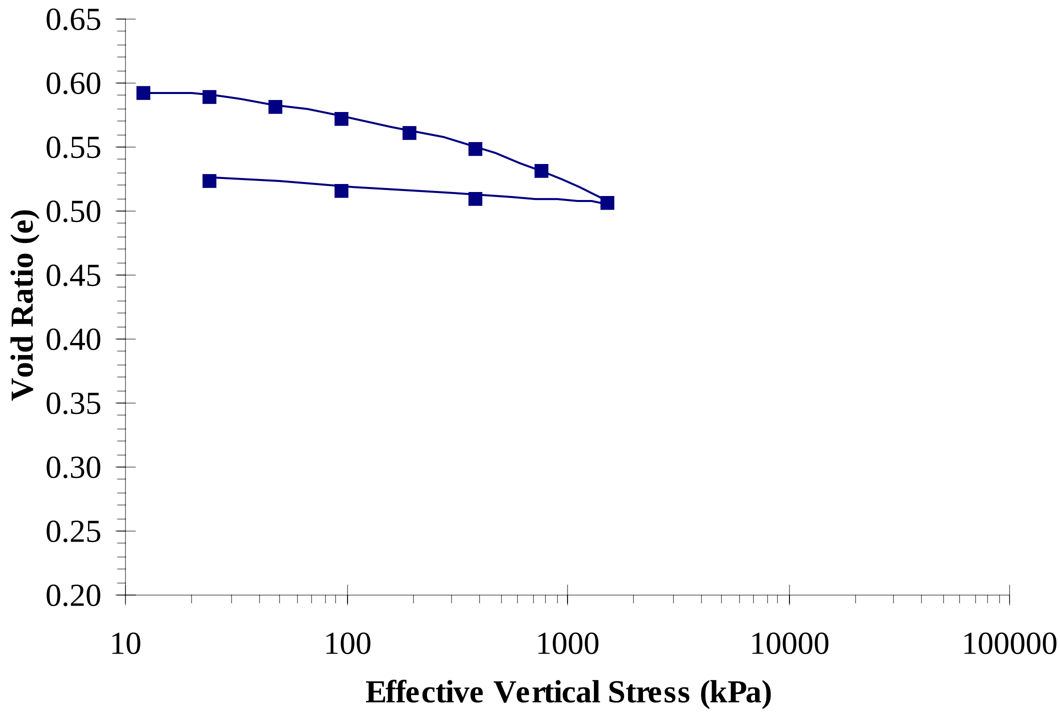

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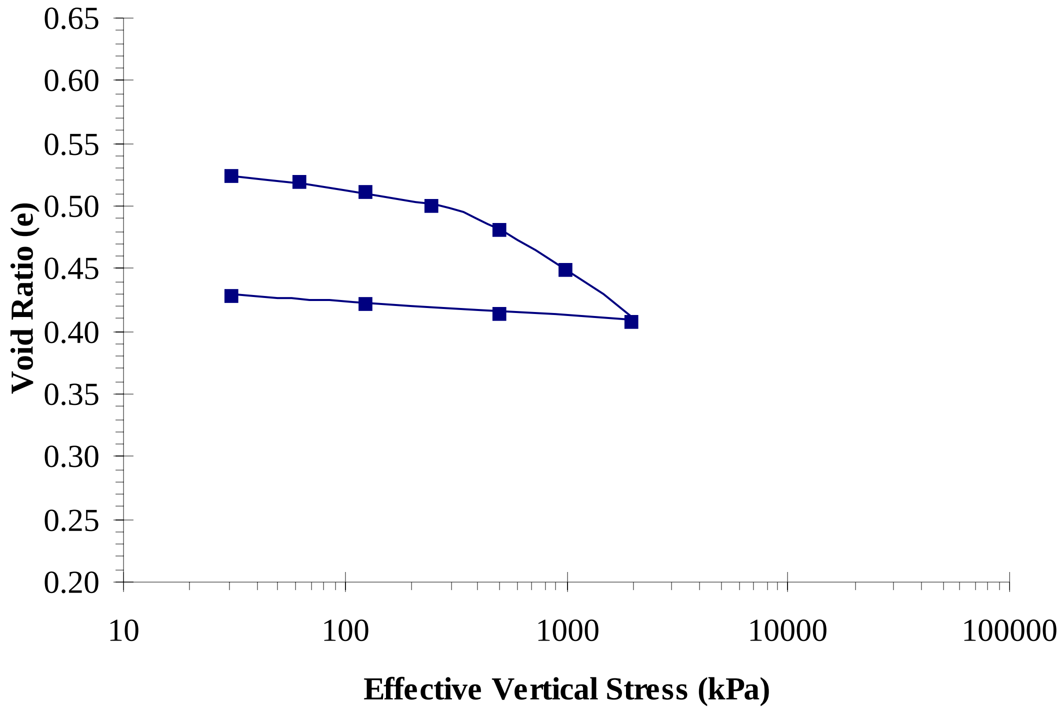

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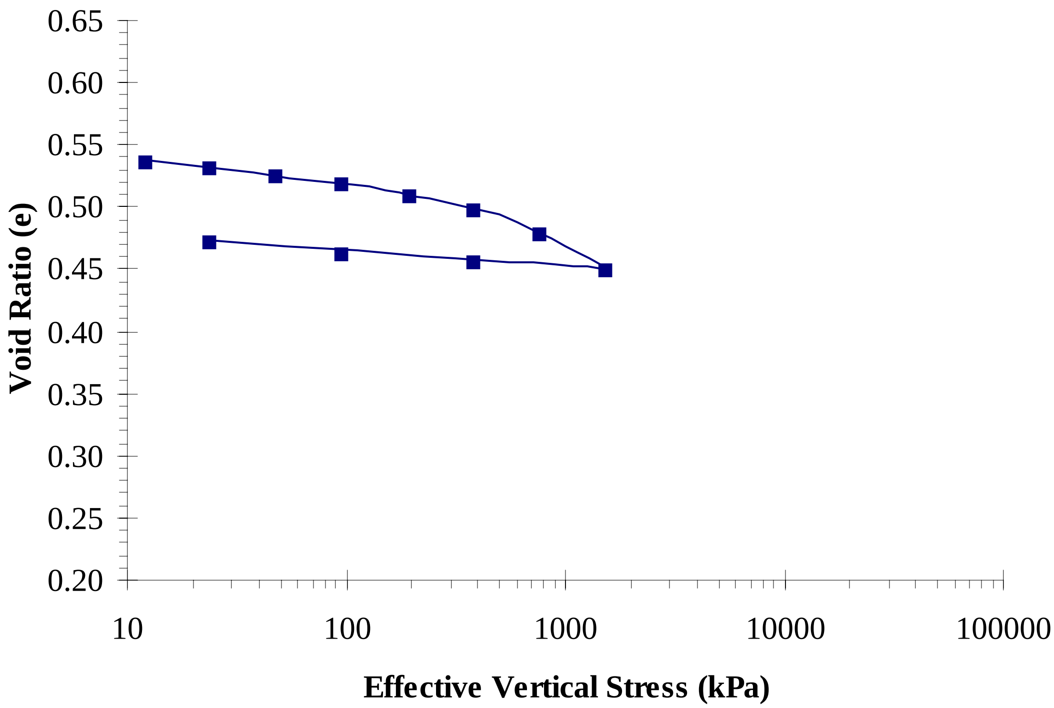

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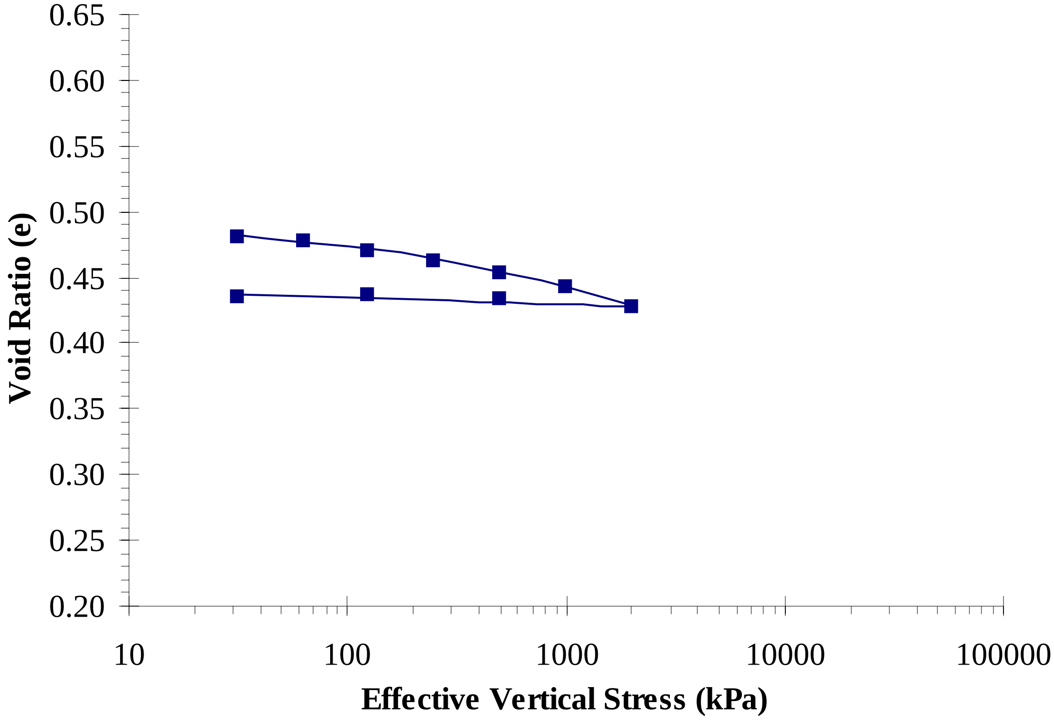

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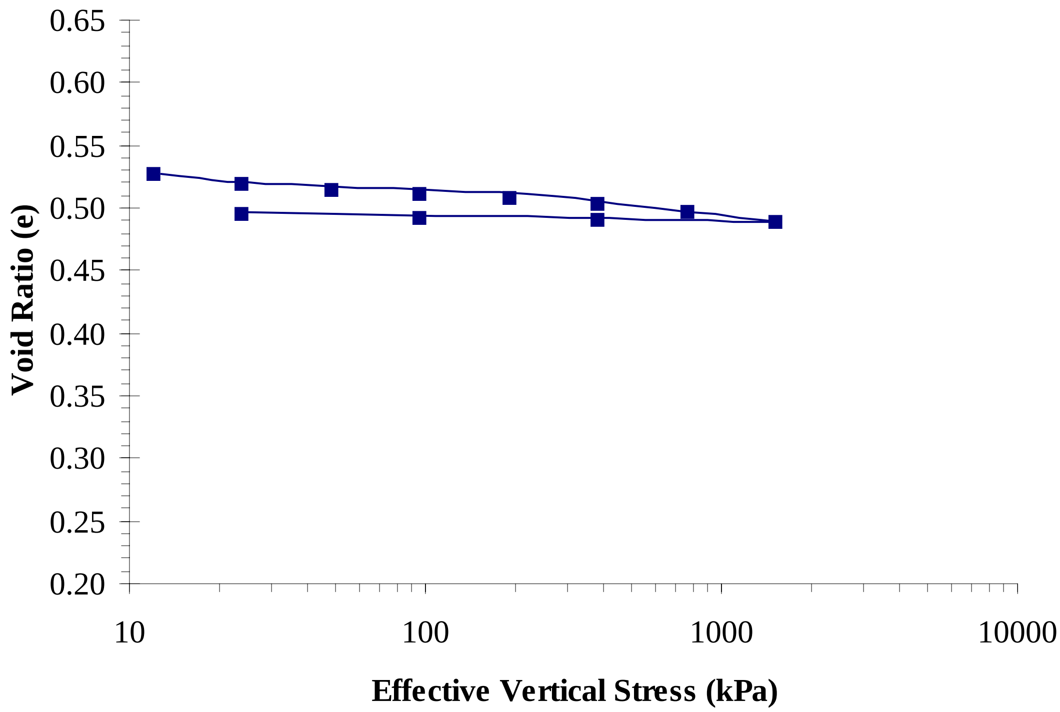

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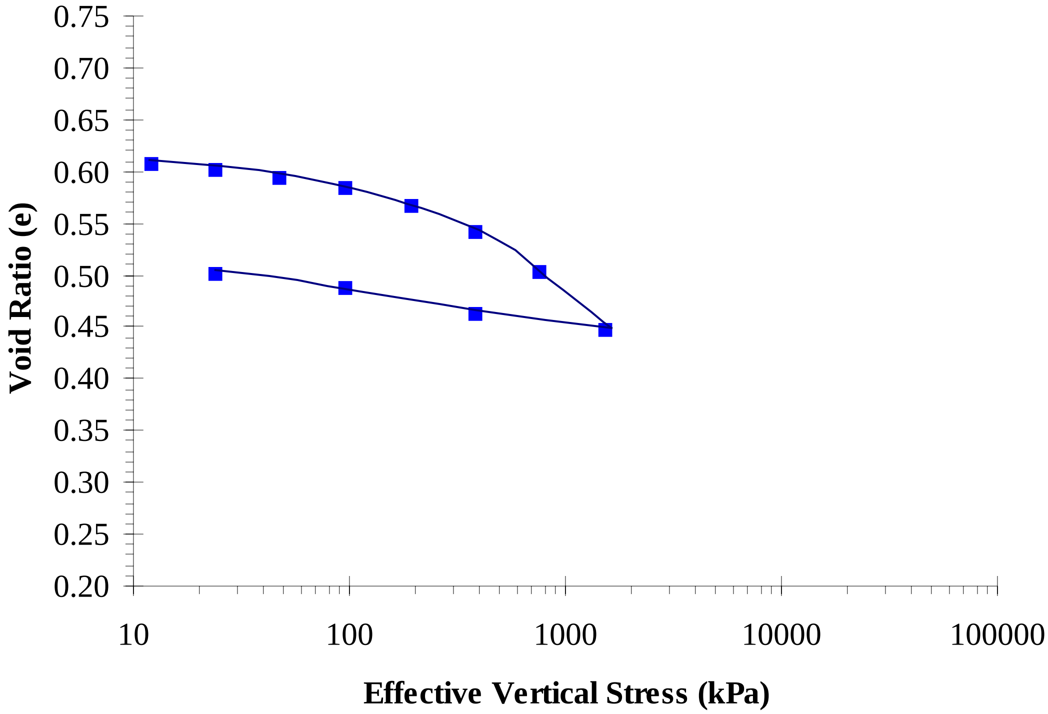
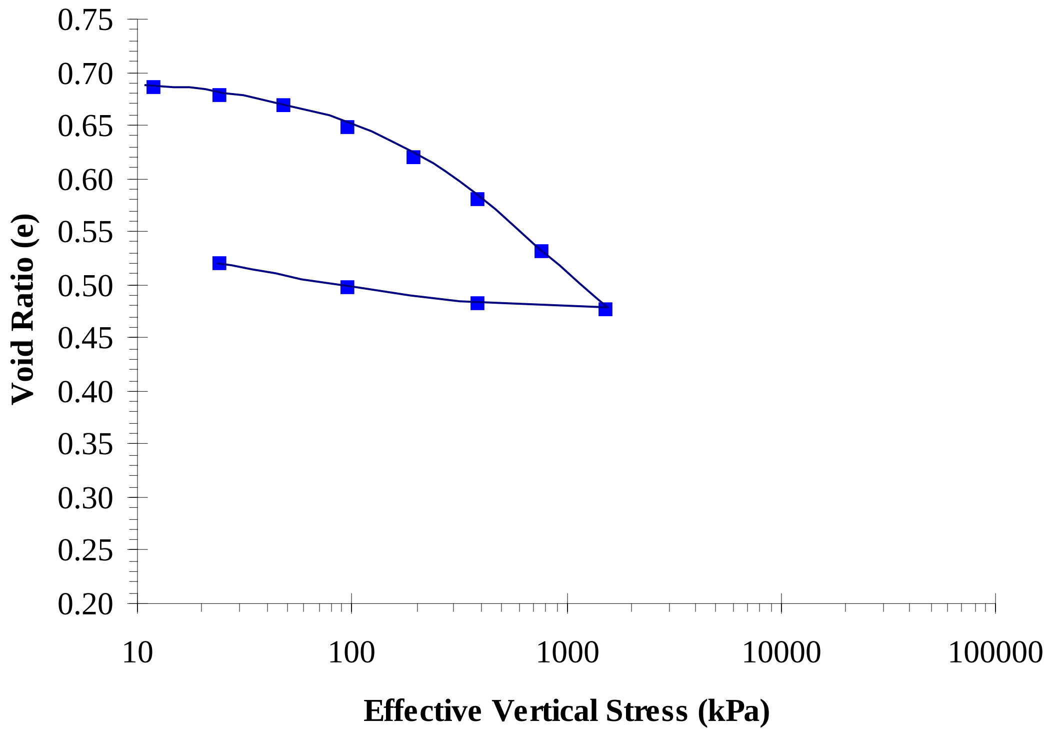


US-24 & Hog Creek 1-1



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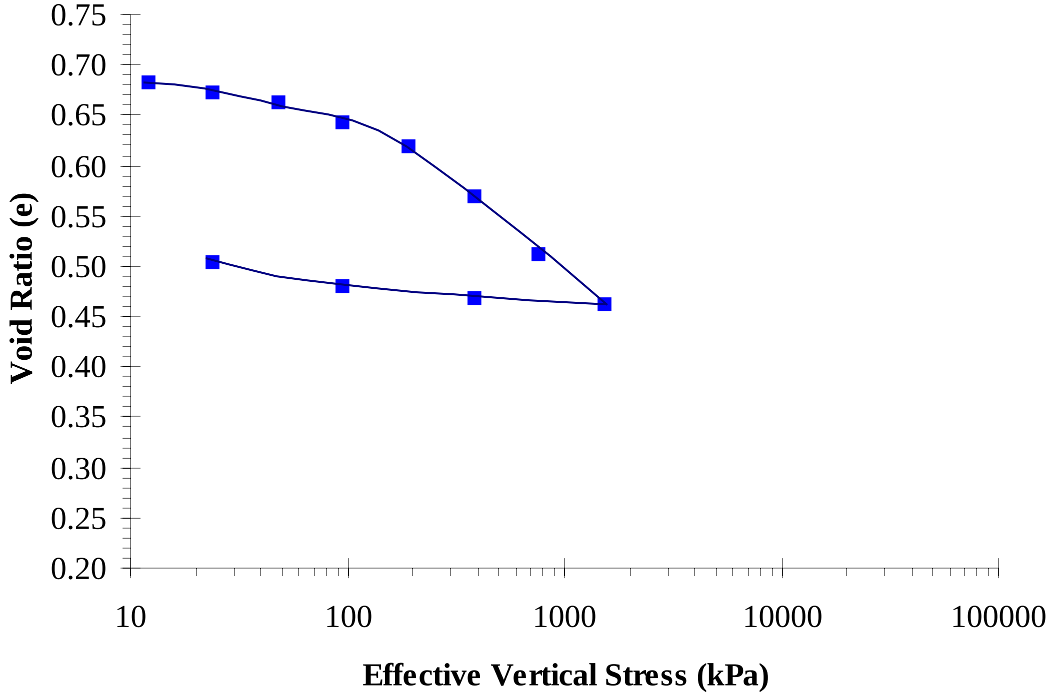

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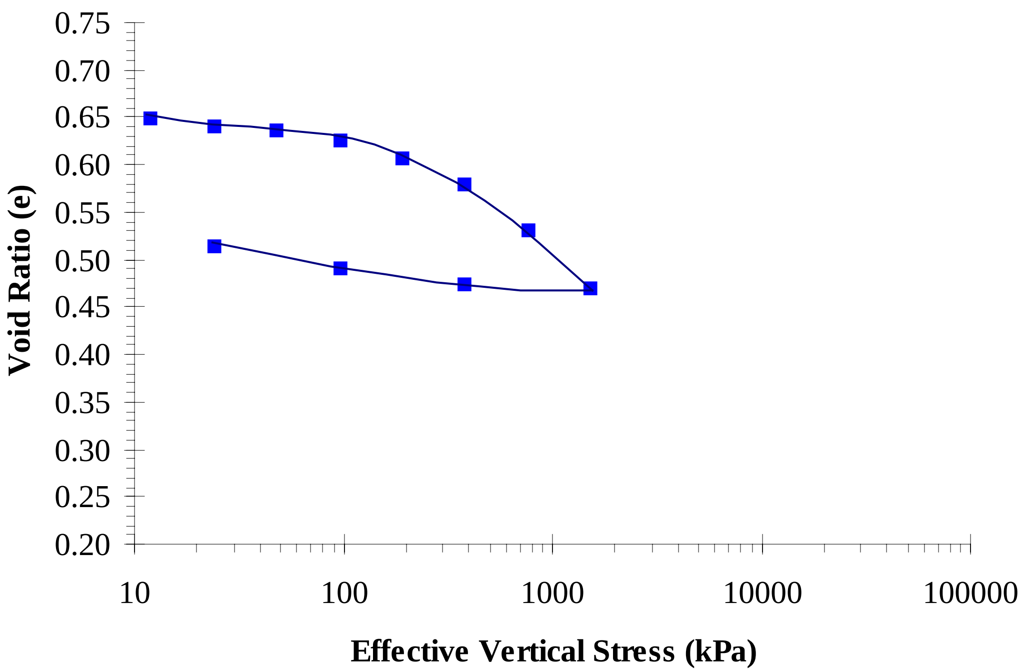

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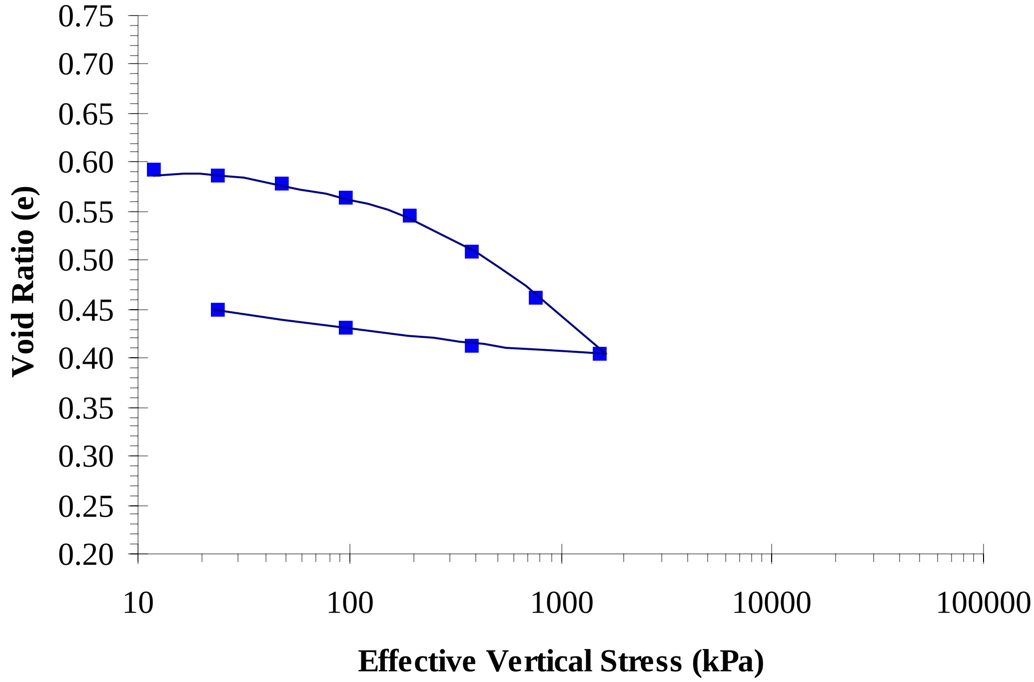

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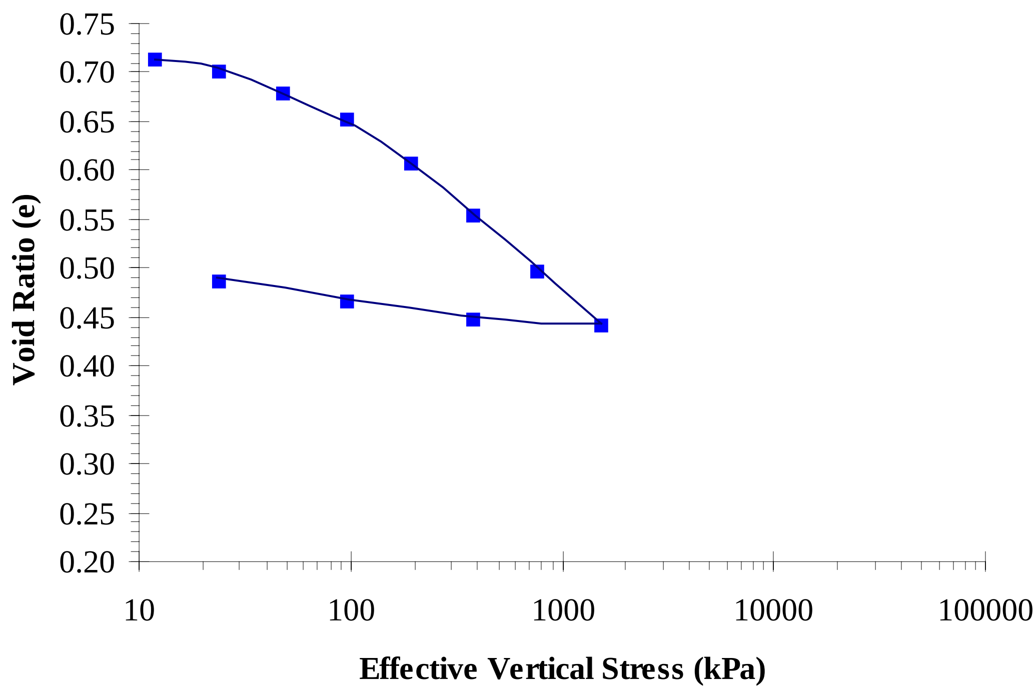

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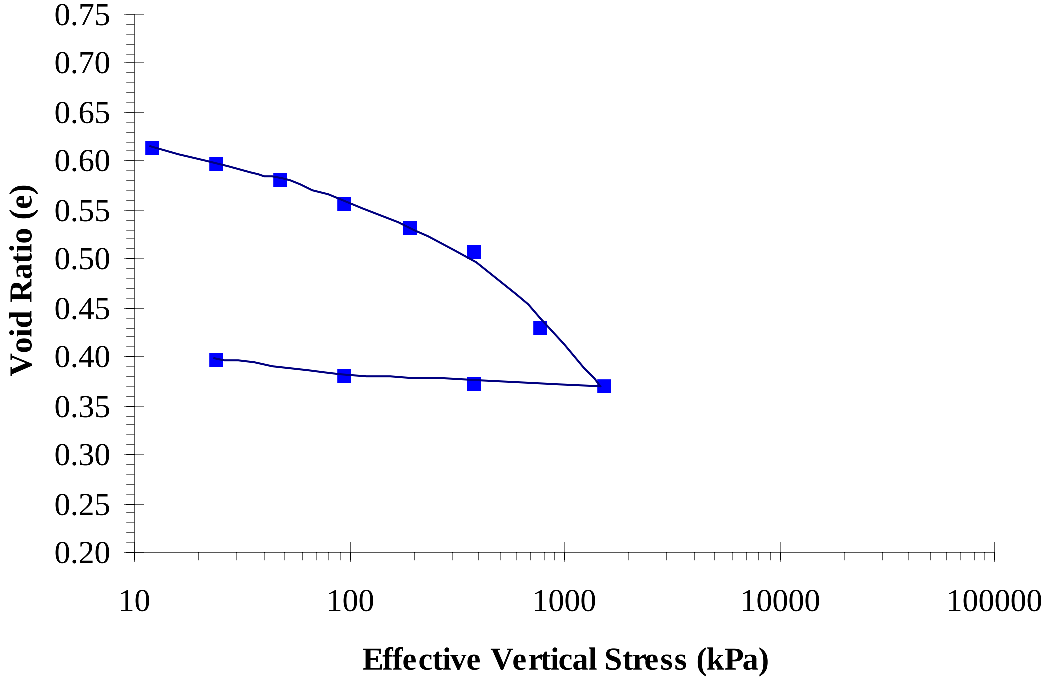

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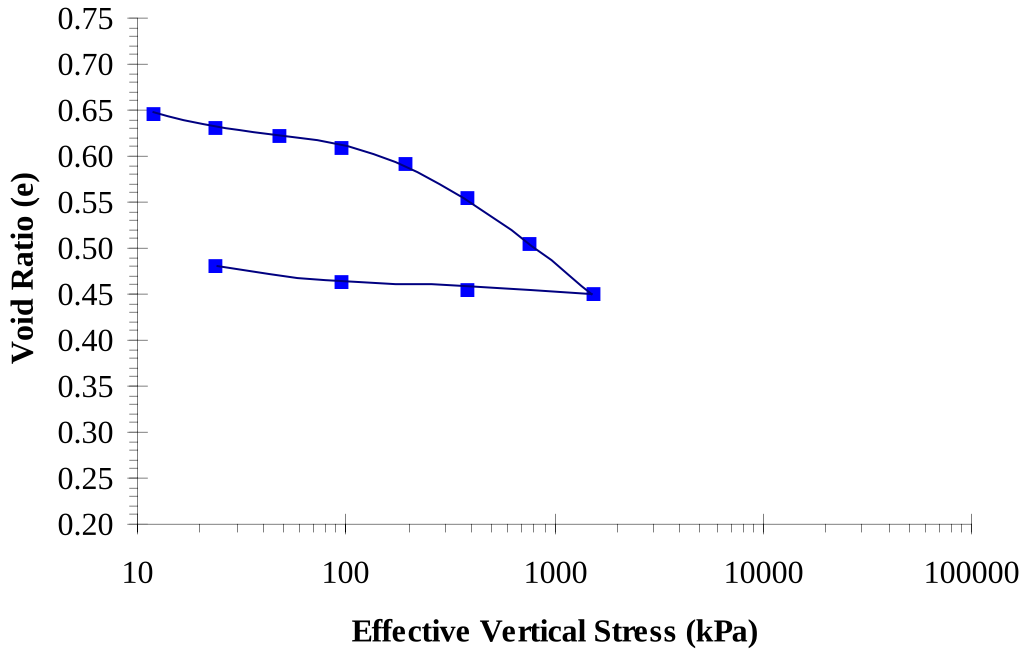
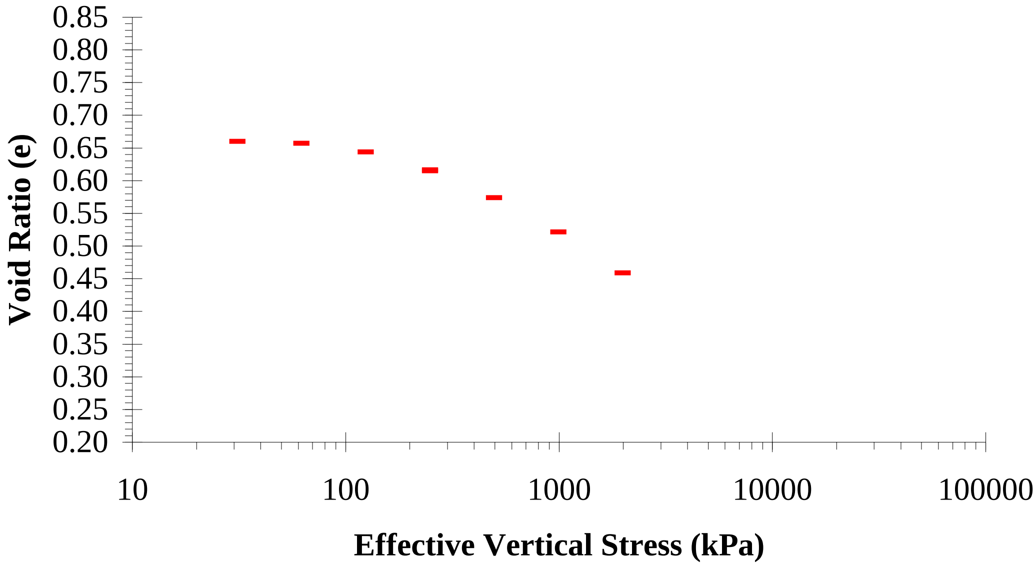


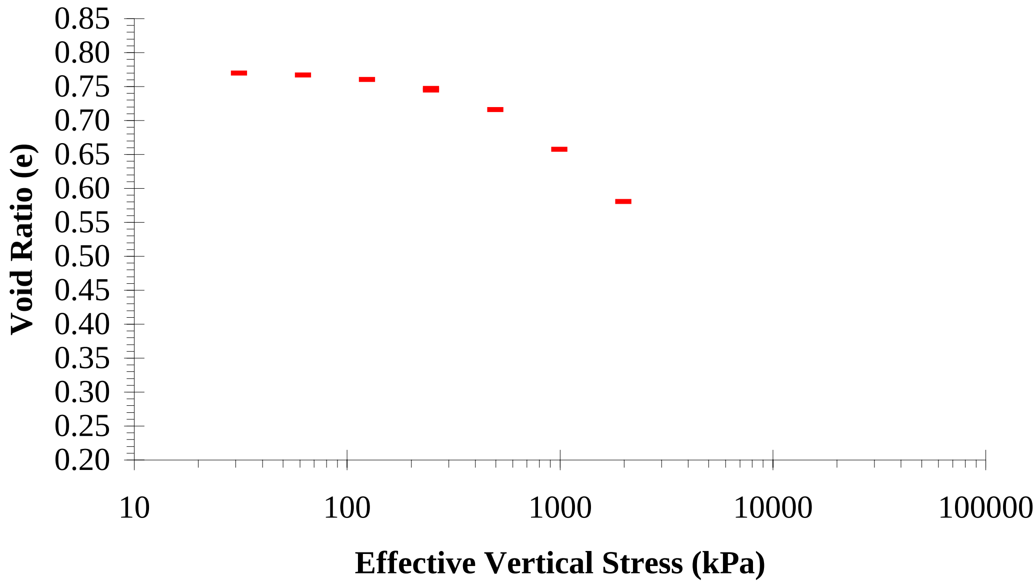
US-24 & Hog Creek 2-4

Appendix C

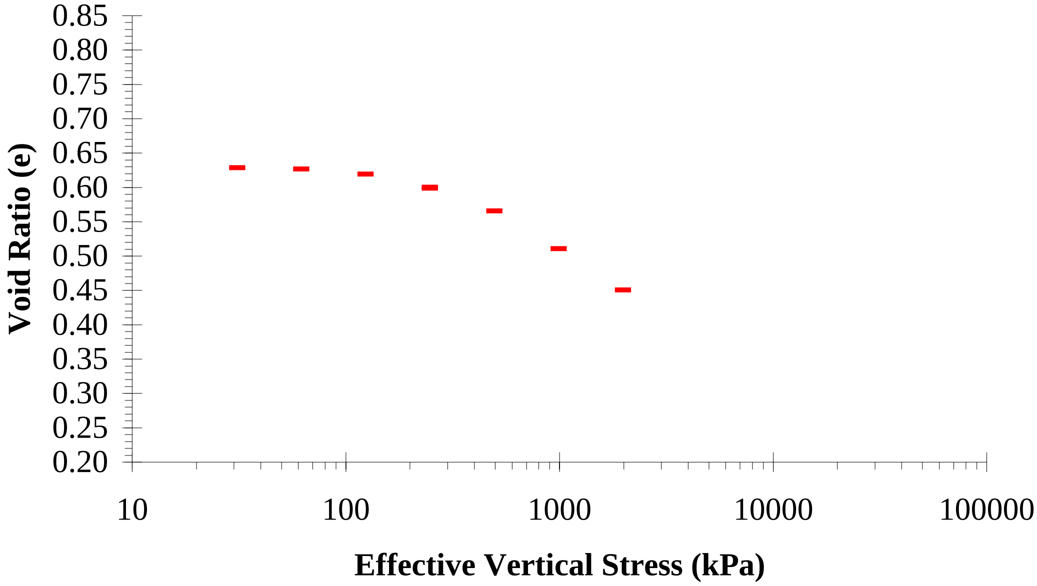
Collapse Graphs



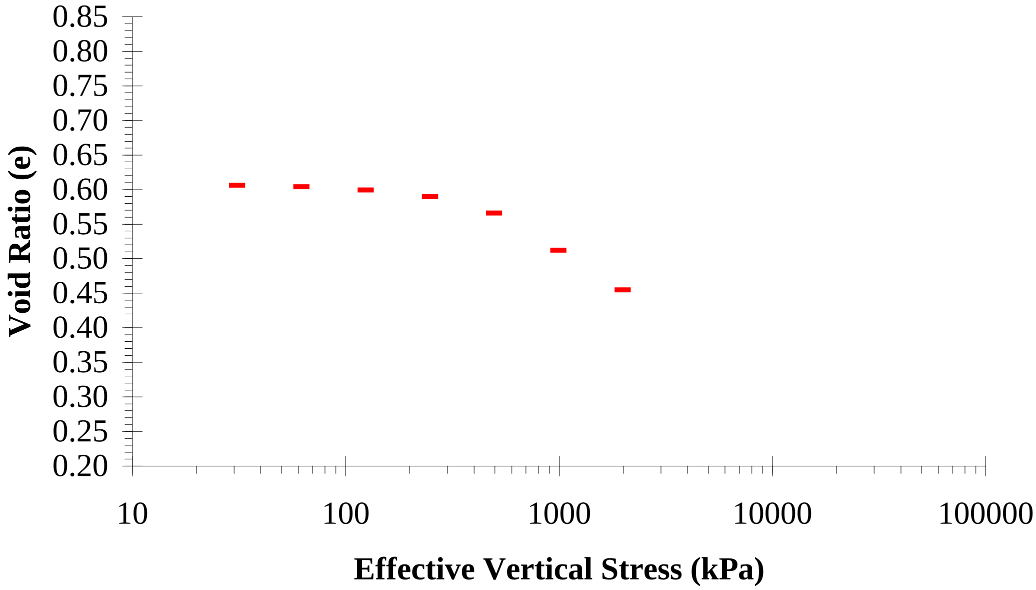
US 40 & Oakland 1-3



US 40 & Oakland 2-2



US 40 & Oakland 1-Dry



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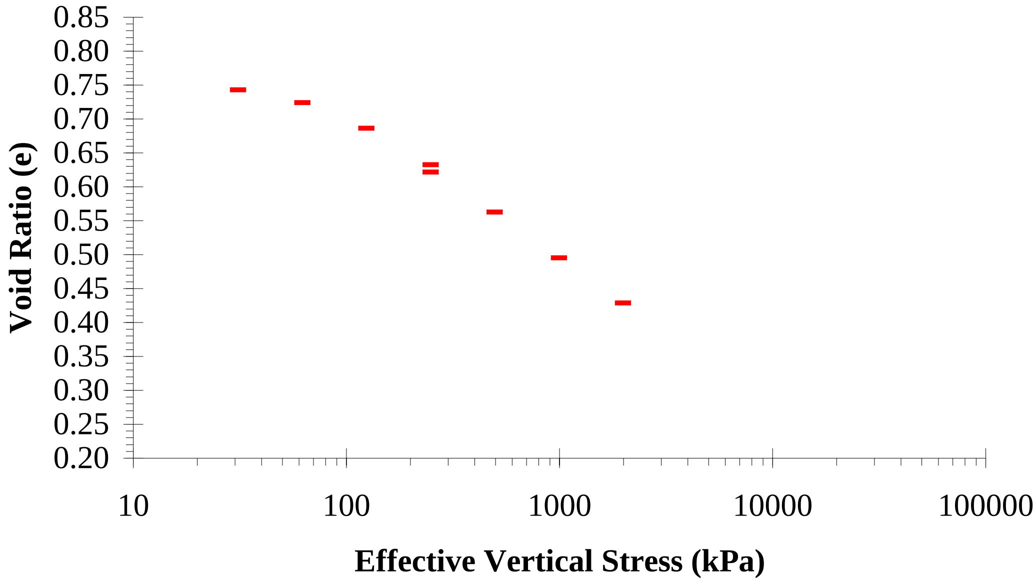
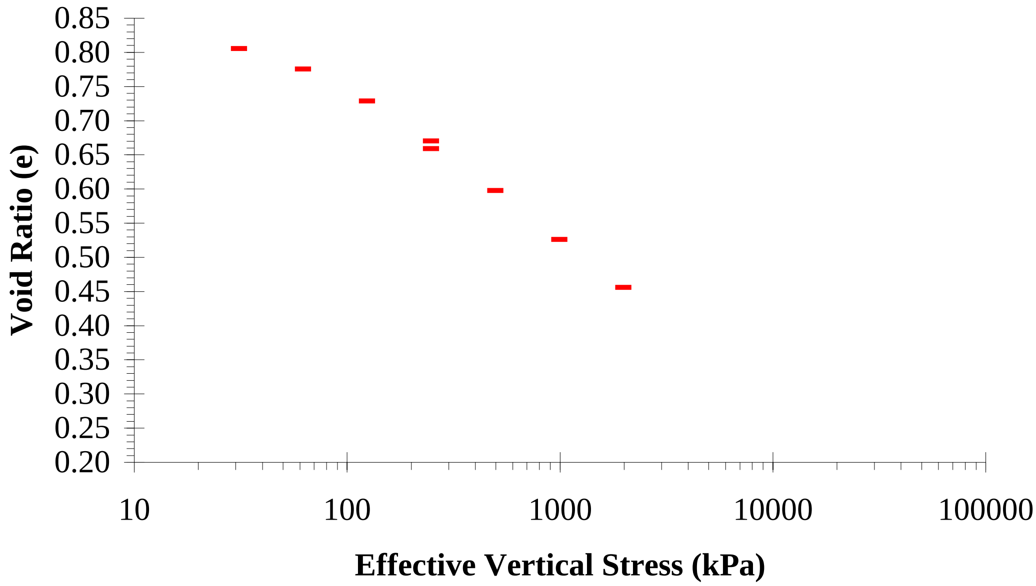
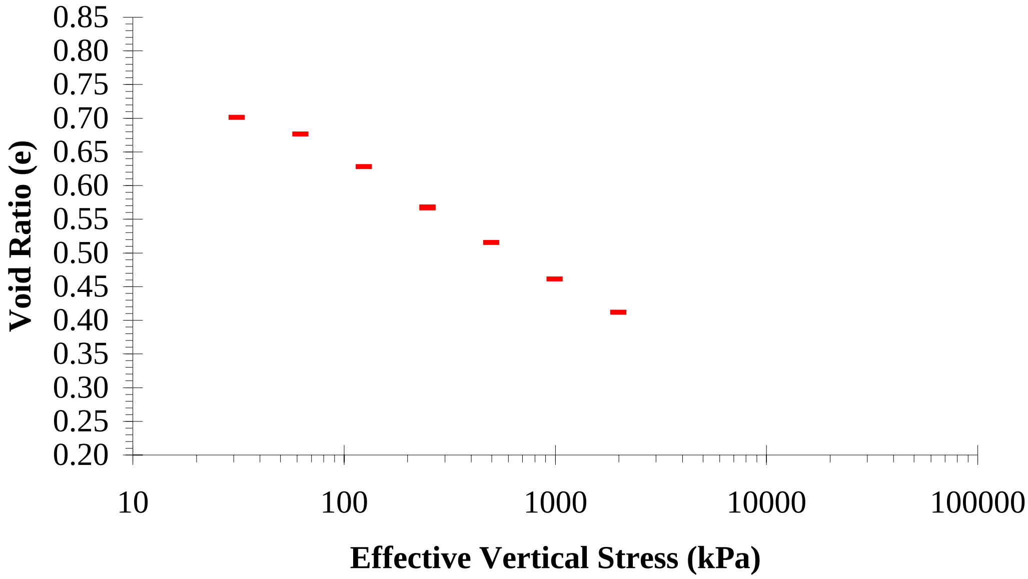


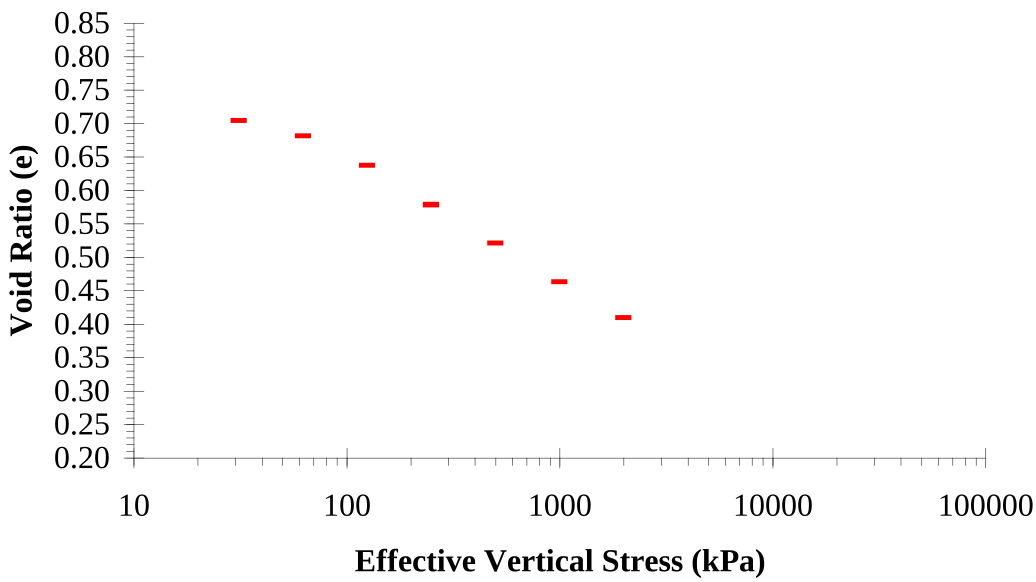
US 40 & Oakland 1-Wet



US 40 & Oakland 2-Wet



I 35 & Antioch 1-Dry



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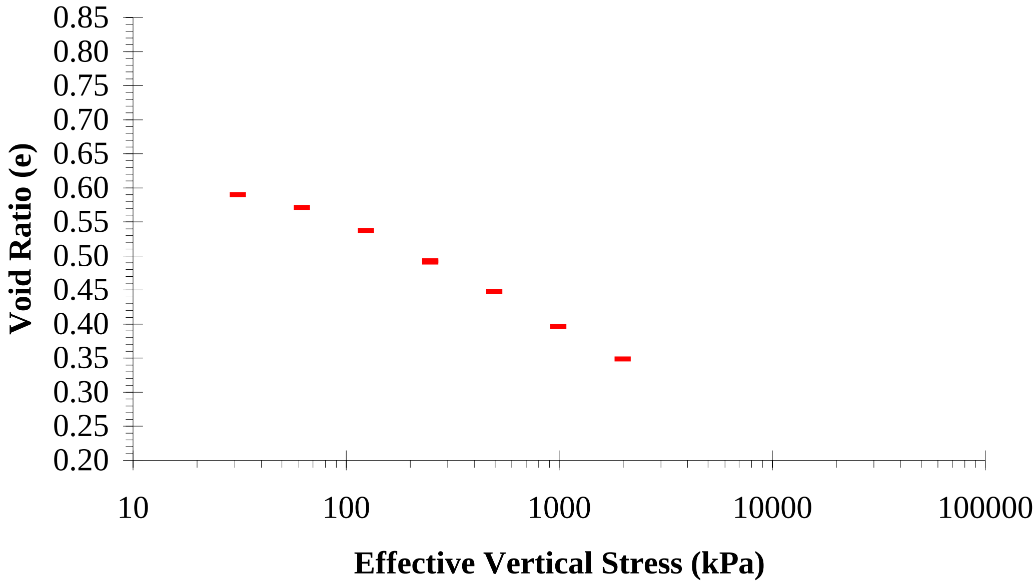

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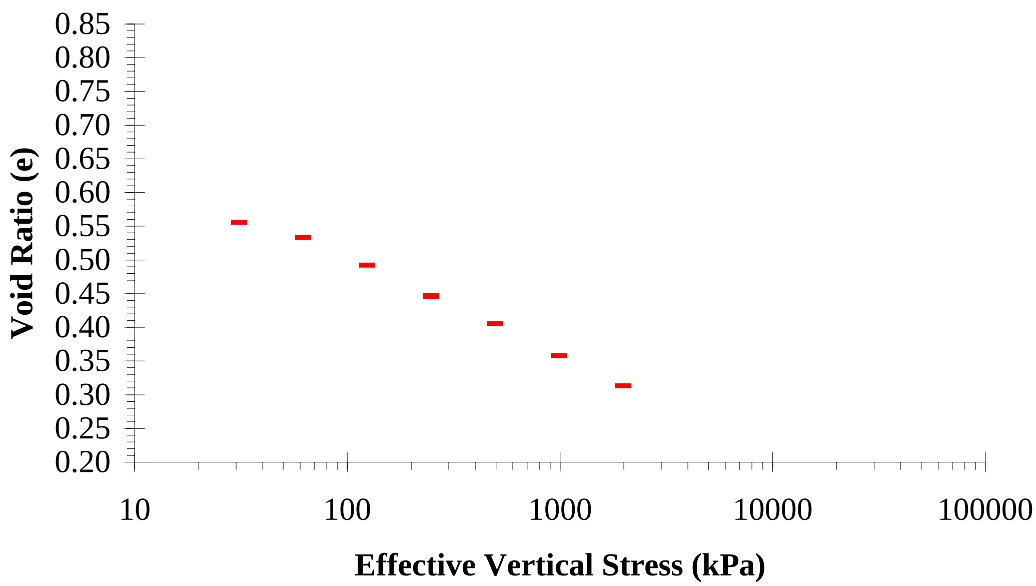

I 35 & Antioch 2-Wet