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16. Abstract

The objective of this study was to evaluate the engineering properties of two local unbound granular materials, with a focus on the gradation and especially the percentage of fines (aggregate passing the No. 200 sieve). Two materials were obtained from two local quarries for this purpose. The percentage of fines was varied in these mixtures. Each mixture was then put through a regime of tests to determine the effects that the gradation and fines content had on their properties. The tests used included moisture-density relationships, moisture susceptibility, triaxial tests, permanent deformation, and resilient modulus tests. The research approach, testing procedure, and results are presented and compared. From the tests performed it was found that increases in the amount of fines have a large impact on the engineering properties of the base materials. However, the results also showed that the percentage of fines used in the base material mixtures had a limit. This limit was found to be approximately 10%. In the range of 5 to 10% fines the base is less moisture susceptible, has higher compressive strength and a higher resilient modulus value.

Texas is the only DOT in the United States which does not control the -200 fraction of its flexible bases. It is not uncommon to find bases with between 20 and 25% passing the -200 sieve. The laboratory data presented in this study would suggest that TxDOT should conduct field investigations to determine if the low fines bases lead to improved field performance.

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Impact of Aggregate Gradation on Base Material Performance

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Abstract

The objective of this study was to evaluate the engineering properties and performance of two local unbound granular materials, with a focus on the gradation and especially the percentage of fines (aggregate passing the No. 200 sieve). Two materials were obtained form two local quarries for this purpose. The percentage of fines was varied in these mixtures. Each mixture was then put through a regime of tests to determine the effects that the gradation and fines content had on their performance. The tests used included moisture-density relationships, moisture susceptibility, triaxial tests, permanent deformation, and resilient modulus tets. The research approach, testing procedure, and results are presented and compared. From the tests performed it was found that increases in the amount of fines often times improved the performance of the base materials. However, the results also showed that the percentage of fines used in the base material mixtures had a limit. This limit was found to be approximately 10%.

Implementation Statement

This study is aimed at addressing the impact that the fines content of a base material will have on the performance and lifespan of flexible bases, especially under increased loading. Through the research the approximate limit of 10% has been found to be adequate enough to prevent permanent deformation and resist moisture susceptibility. However, further analysis may determine a more exact percentage.

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CHAPTER 1

INTRODUCTION

The quality of the material used as base for road construction can have a significant impact on the performance and life of the pavement that it is intended to support. This is especially true in cases where flexible pavements such as asphalt are used. The material selected as well as the construction methods used to place the material can also affect the performance of the unbound granular base. It has been found that although current testing procedures provide for durable pavements, a relationship between material performance in the laboratory and in the field does not exist. Among the problems is the processing and compaction of the base material during construction. Although all base materials will be exposed to continuous traffic loading, the performance of the base material becomes even more important when heavy-duty flexible pavements are used for greater volumes of traffic.

Also of concern is the selection process of base materials. It has been found that it is not uncommon for a district to have several bases classified as Class 1 materials even though they have varying properties. The need exists to develop a clearer relationship between laboratory results and field performance. At the present time density is used to determine the quality of base layers. However, there are many other factors that need to be considered. Among these are moisture variation with depth, exposure time prior to sealing, and thickness of the layers. All of these, of course deal with the construction of the base layer.

One of the topics currently being studied includes the impact of gradation and fines content of the base material on its performance. The general contention has been that the engineering properties of the material can be modified by altering its gradation. Specifically, the highway community is in agreement that the amount of fines (material passing No. 200 sieve) found in a base material can dramatically impact the behavior of a base layer. It has been determined that the fines content found in a base material mixture is linked to pavement distress and failure. Among the distresses that have links to the fines content are fatigue cracking, or alligator cracking, and rutting. Studies have also shown that the type of fines found in the base material also affect the performance of base materials. Among the findings are the effects that plastic fines have on the performance of base materials. More specifically, plastic fines impact the moisture susceptibility and the stiffness of the base material.

Under this project, various tests were used to determine the properties of base materials varying in gradation and fines content. The properties that were analyzed include strength, stiffness, moisture susceptibility, and permanent deformation. A combination of tests was performed to further understand the effects of gradation and fines content on the base materials. These tests included sieve analysis, moisture-density curves, Atterberg limits, triaxial tests, moisture susceptibility, permanent deformation, and resilient modulus.

This project was developed and simultaneously worked on with the Texas Transportation Institute (TTI) at Texas A&M University. Research approaches, testing procedures and results from both universities are presented and compared.

Chapter 2 of this report provides a brief background on previous research concerning the impact of aggregation and fines content on base material performance. Also included is a review of the European specifications on base materials focusing on gradation and fines content as well. In Chapter 3, the procedures followed to select the varying base gradations, testing procedures, and results obtained at UTEP are presented. Chapter 4 contains a summary of the tests performed and results obtained at TTI, with whom we partnered for this project. Chapter 5, the last chapter, includes a summary of the findings and the conclusions drawn.

CHAPTER 2

BACKGROUND

As previously stated, the quality of a base material can significantly alter the lifespan of the pavement that it supports. This is even truer for flexible pavements. Previous research has found that much of the distress that flexible pavements experience can be traced to problems encountered in the base material. Table 2.1 is a summary of these distresses originally found in a report produced by the NCHRP (NCHRP, 2000).

Although various tests exist that are used to classify base materials it is generally agreed that the correlation between the field and laboratory performance is not well-defined. Currently, in Texas it is not uncommon for districts to classify different materials as Class 1 though they have varying field modulus and moisture retention properties. Also of concern is the fact that other tests, such as strength-related tests, are often times not performed due to time and personnel constraints. Tests that provide a much clearer relationship to performance in the field are still needed. The need to develop such tests is emphasized by the current design by the Texas Department of Transportation for heavy-duty flexible pavements for large volumes of traffic.

Once more the techniques used during construction of a base layer also play an important role in how well a base layer performs. Currently density is used as the determining factor for the quality of a base layer. However, as Table 2.1 summarizes, density is not the sole factor in how a base material performs in the field. Among the factors that need to be considered are the moisture retention capabilities and moisture variation of the base layer, as well as its stiffness and moisture susceptibility. Construction quality control including base thickness and number of lifts are also factors in the ability of a base material to perform well in the field. It should be noted that of the four categories shown the first two are noted as being correlated to the fines content. However, frost heave has also been known to be connected to the fines content.

Table 2.1 Summary of Flexible Pavement Distress, Contributing Factors and Related Test Parameters (NCHRP, 2000)

Type of Distress	Description of Distress	Base Failure Manifestation	Contributing Factors	Possible Related Test Parameter
Fatigue Cracking (Alligator Cracking)	Appears as fine, longitudinal hairline cracks parallel to one another in the wheel path in the direction of traffic. Progression of distress is signaled by interconnection of cracks forming many-sided, sharp angled pieces. As cracks become wider spalling may occur	High deflection/strain in the asphalt concrete surface due to lack of base stiffness. Alligator cracking only occurs in areas where repeated wheel loads are applied. High flexibility in the base or inadequate thickness of base allows for excessive bending strains in the asphalt concrete surface. Changes in base properties with time can render the base inadequate to support loads.	Low modulus Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture. Degradation under repeated loads and freeze-thaw cycling.	Resilient Modulus Gradation & fines content Frost susceptibility Density
Rutting/ Corrugations	Long surface depressions in the wheel path that may not be noticeable except during and following rains. Pavement uplift may occur along the sides of the rut. Resulting from permanent deformation in one or more pavement layers or subgrade, usually caused by consolidation and/or lateral movement of the materials due to load	Lateral displacement of particles with applications of wheel loads due to inadequate shear strength resulting in a decrease in the base layer thickness. Consolidation of the base due to inadequate initial density or changes in base properties with time due to poor durability or frost effects may also cause rutting.	Low shear strength Low density of base material Improper gradation High fines content High moisture level Lack of adequate particle angularity and surface texture. Degradation under repeated loads and freeze-thaw cycling.	Triaxial Testing – angle of internal friction, cohesion Gradation Fines content
Depressions	Depressions are localized low areas in the pavement surface caused by settlement of the foundation soil or consolidation in the subgrade or base/ subbase layers due to improper compaction.	Inadequate initial compaction or non-uniform material conditions results in additional reduction in volume with load applications. Changes in material conditions due to poor durability or frost effects may also result in localized densification with eventual fatigue failure.	Low density of base material	Density
Frost Heave	Frost heave appears as an upward bulge in the pavement surface and may be accompanied by surface cracking resulting in potholes. Freezing of underlying layers resulting in an increased volume of material causes the upheaval. An advanced stage of distortion mode of distress resulting from differential heave is surface cracking with random orientation and spacing.	Ice lenses are created within the base/subbase during freezing temperatures, particularly when freezing occurs slowly, as moisture is pulled from below by capillary action. During spring thaw large quantities of water are released from the frozen zone, which can include all unbound materials.	Freezing temperatures Source of water Permeability of material high enough to allow free moisture movement to the freezing zone.	Gradation Fines Content Fines Type

TTI LITERATURE REVIEW

In a literature review performed by Hefer and Scullion (2003) for Phase I for this project the current specifications in Texas as well as various other states and two overseas countries were compared. The states and countries that were chosen exhibited climatic conditions that are similar to Texas; as well as their practices in building pavement structures that are similar to Texas'. Random states were also chosen to compliment the search for innovative concepts and practices relating to high quality aggregate bases. From this literature review, there were six categories under which base materials specifications were classified. These categories were strength, gradation, percentage of fines, degradation and soundness, crushed particle characteristics, moisture susceptibility, and compaction.

Hefer and Scullion indicated that the manner in which different state highway agencies classified materials varied according to their respective geological group, the state of weathering, or the test parameters that have been specified. Among their findings were three aspects for which Texas' standards stood out. First, Texas was the only state that did not control the amount of fines permitted in its base material. Second, Texas' plasticity index was found to be higher than most other state DOT's. Last, the majority of state DOTs had a requirement for shape, angularity, and surface texture while Texas did not.

Among the issues that have been considered in past research on bases include gradation, maximum aggregate size, the percentage of fines and the plasticity of the fines found in base materials. In NCHRP (2000), it was found that gradation of base materials could influence their performance in the field. More specifically the areas discussed in that study were concerned with shear strength, stiffness, and moisture susceptibility.

Gray (1962) studied the effects of maximum aggregate size and percentage of fines. Through the use of the Texas Triaxial tests, Gray found that increases in maximum aggregate size resulted in increases in cohesion and resistance to shear. It was also found that the ultimate strength of the base material also increased when the maximum particle size was increased. Gray also found that increases in fines content also resulted in higher densities; however, as the percentage of fines increases the bearing ratio decreased. The amount of fines was also affected by the maximum aggregate size; with the fines content decreasing as the maximum aggregate size increased.

Also of interest was the amount of permanent deformation a base material could be expected to undergo as well as the stiffness of the material. NCHRP (2000) indicated that the fines content affected the rutting of flexible pavements. Studies performed by Barksdale and Itani (1989) indicated that increases in the fines content could result in higher permanent deformation and lower resilient modulus values. These findings reinforced the previous findings of Jorenby and Hicks (1986) in their study of the design and performance of flexible bases.

The plasticity of the fines content is also an important factor to take into consideration when designing a base layer. The plasticity of a soil can affect the amount of water retained as well as the amount of water that is attracted to a base material – which can alter the desired design properties. Gray (1962) showed that the presence of plastic fines had a negative effect on the performance of base materials. Rapid increases in percent strain were also found to be a result of the increased plasticity of fines found in the base material. Gray stated that the best solution to alleviate the negative effects of plastic fines was to produce fines that come from the parent rock.

In a joint research project between TTI and the Finnish National Road Administration (Guthrie *et. al.*, 2000) a means to determine the susceptibility of a soil to moisture was developed. The test itself measures the dielectric values of compacted specimens as they are allowed to soak over a ten-day period. In a project conducted by the Tempere University of Technology (Scullion, 1997) it was found that the suction action was a result of the fines content and the chemical properties of the material used. With more emphasis placed on the presence and content of fines found in the base material, it was also determined that exceeding 5% fines caused the dielectric constant to reach the preset limit of 9. The previously designated ranges for the dielectric constant were less than 10 for good material, 10 through 16 for fair material, and dielectric constants greater than 16 as poor material.

The research performed at the Tempere University of Technology, unlike the research of Barksdale an Itani, showed that the resilient modulus increased with increases in fines content for dry specimens. However, the specimens that were conditioned to experience moist and post-freeze- thaw conditions had resilient modulus values that decreased with increases in the fines content. The addition of fines in this study generally showed that increases in the fines content resulted in higher permanent deformation.

EUROPEAN CODES

As in the United States, countries in Europe often use aggregates produced in quarries or naturally occurring gravel as base materials. Again, as in the United States emphasis has been placed on the gradation of unbound granular bases used as pavement materials. In a report published by the European Cooperation in the Field of Scientific and Technical Research (COST 337) the sources, production methods, in situ and laboratory methods and their specifications were looked at for the use with unbound granular materials.

As with any road pavement construction, there are many unique aspects and challenges from an engineering point of view. As pointed out in the COST 337 these include the fact that the material is naturally occurring with a wide array of properties. The loading on road pavement is applied in a cyclic fashion thousands to millions of times. The response to the loading and the material type combine to produce the engineering challenge. The COST report points out that the mechanical behavior of unbound granular material can be placed into three categories. These are strength, stiffness, and permanent deformation.

Strength is affected by the mineral type, particle shape, roughness, strength and packing; which can be affected by the particle sizes found within the bases gradation. The percentage of fines and the type of fines also affect the strength of bases. The increase in fines has been found to increase the angle of internal friction. However, it was also seen that the addition of clay fines to an unbound granular base mix would lower the angle of internal friction due to reduced interparticle contact.

COST 337 defines stiffness as the ratio of repeated stress to resilient strain. Much like their American counterparts, many European nations use resilient modulus to describe the stiffness of base materials.

COST 337 found that the permanent deformation behavior occurred in three forms. The first was that due to "shakedown" where voids in the material are filled as the base is loaded. Under this type of behavior, the material behaves resiliently in the end, becoming asymptotic to a constant value. In the second form, the material deforms continuously over time. The final form is described as non-stabilizing which produces large deformations.

Strength was also stated as being related to the stiffness and permanent deformation of the base material. With this statement in mind, the items that affect the strength of a base material also would affect the stiffness and permanent deformation of the material.

COST 337 also looked at the testing of unbound granular materials. Laboratory tests that are performed throughout Europe included particle size, shape and strength, moisture sensitivity (susceptibility), durability, and density and moisture content. Also included were repeated load triaxial tests which allowed for the simulation of pavement loading. Resilient behavior tests and permanent strain test were also evaluated. The manner in which the report described these tests is summarized below.

- Particle Size – Determination through gradation and fines content
- Particle Shape – Determination of particle shape through the shape index, flakiness index, and percentage of crushed particles
- Particle Strength – Resistance to crushing through the Los Angels Test, various abrasion tests, and country-specific tests
- Moisture Sensitivity – Estimates through plastic fines content through sand equivalency test, methylene blue test, and country-specific tests
- Durability – Under this category was included resistance to frost heave and magnesium sulphate soundness
- Density and Moisture Content – Most commonly used was the Proctor test
- Resilient Behavior – Not specified as a standard in most countries, procedure varied country to country as well
- Permanent Strain – Repeat Triaxial loading with either a constant or variable confining pressure

Of special interest were the specifications for grading the base material. The maximum aggregate size varied throughout the continent, with Nordic countries having large maximum aggregate sizes due to the use of thick subbases for frost protection. The percentages of fines ranged from 0% - 4% at the low end, and up to 5% - 15% at the high end. Due to frost susceptibility, some countries had set the fines content at or below 3%. This was mostly in the Nordic countries where open-graded materials are also used to minimize moisture and frost problems. One of the complicating factors in the COST 337 report was that fines were defined differently with particle diameters ranging from 0.00248 in to 0.00315 in.

Even with varying definitions for the term "fines" the underlying thought shared among the participating countries was that the percentage of fines found in a base material needed to be controlled. It was also stated that although some fines in the material are desirable, too large of a percentage can decrease the stiffness and permeability of a base material; as well as increase its susceptibility to frost. However, COST does not set a limit on the percentage of fines that should be allowed in base materials.

The particle shape was mostly determined using the flakiness index, with a smaller number of countries using the shape index. The proportion of crushed particles had not yet been specified in many countries, only France was mentioned of having a means to evaluate the percentage of crushed particles in a material.

There were also varying views on how to evaluate the cleanliness and quality of fines present in base materials. The methods include the Sand Equivalent test, the Methylene Blue test and the Atterberg Limits. The latter was discussed further in the report and could be summarized as placing the participating countries into two categories: those that only allowed for the use of non-plastic fines, and those that placed a limit on the plasticity index of the fines found in the base materials. In either category the Atterberg requirements varied. There were also specifications in some countries pertaining to the sand equivalency tests as well as a combination of the two when the sand equivalency reached a certain limit.

Particle strength was mostly determined using the Los Angeles Abrasion test with the thought that most all countries participating in the study would adopt it as the norm. However, at the time of publication there were varying limits on the LA abrasion values, as well as, varied use of other tests to determine particle strength.

Frost sensitivity or susceptibility was one of the areas that did not seem to receive much attention even though many of the European countries involved were susceptible to extreme climatic conditions. Those countries that did have specifications varied in their testing methods. These methods included freeze/thaw tests, frost heave tests, and varied indirect methods or reliance on other criteria such as classification criteria.

The review of specifications is summarized below.

- Grading and Particle Shape
  Criteria are similar within the countries surveyed; there exists a wide variation in the maximum particle size and specification of fines content.
- Cleanliness of Fines
  Three different tests (Sand Equivalent, Methylene Blue and Atterberg Limits) are currently used with no agreement on the most appropriate one.
- Mechanical Resistance of Aggregate
  Main test is the LA abrasion test with a vast range of maximum values (25 – 50).

The need to determine what effects the gradation, and specifically the percentage of fines, has on base material is an item that seemed to be of importance in both the United States and Europe. This is the underlying theme of the research that was conducted for this project.

CHAPTER 3

UTEP APPROACH

As stated previously, the amount of fines found in base materials is not currently specified in Texas. However, past research has shown that the amount of fines affects the performance of base material in the field. The effects of fines content and the gradation of two base materials used in Texas were investigated in this study.

The procedures followed at UTEP to investigate the impact of gradation on the performance and strength of several base materials are presented in this chapter. Under the UTEP approach, the gradation of base materials was varied according to fines content to optimize the mixtures. In addition, the impact of low- and high-plasticity fines was also evaluated. In total, seven mixes were studied.

MATERIAL DESCRIPTION

One common base used in the El Paso District was selected. The local producer typically combines three types of materials to produce the base. Prior to mixing the materials, large samples of the three materials were obtained from a local quarry. The materials were oven-dried and sieved to determine their original gradation. The gradation of each bin is shown in Figure 3.1. The first material, called Bin A, consisted of approximately 20% of the aggregates passing the $^3/_8$-in. sieve, 10% retained on the No. 4 sieve and 70% passing the No. 4 sieve. This material is dolamatic in nature. The second material, Bin B, consisted of material that was approximately 95% finer than the No. 4 sieve. Finally, Bin C consisted of aggregates of which 95% were finer than $^3/_8$-inch and about 6% passed the No. 4 sieve. The commercial product delivered to TxDOT consists of approximately: 60% Bin A, 10% Bin B and 30% Bin C.

Seven different mixtures were blended and tested to determine various engineering properties of the materials. Of these seven mixes, three contained different percentages of low-plasticity fines naturally mixed with the material. The second set of three mixes were similar to the first three mixtures in gradation, however, the low-plasticity fines were replaced with clay (plastic fines). The seventh mix contained no clay or fines and was used as a control mix.

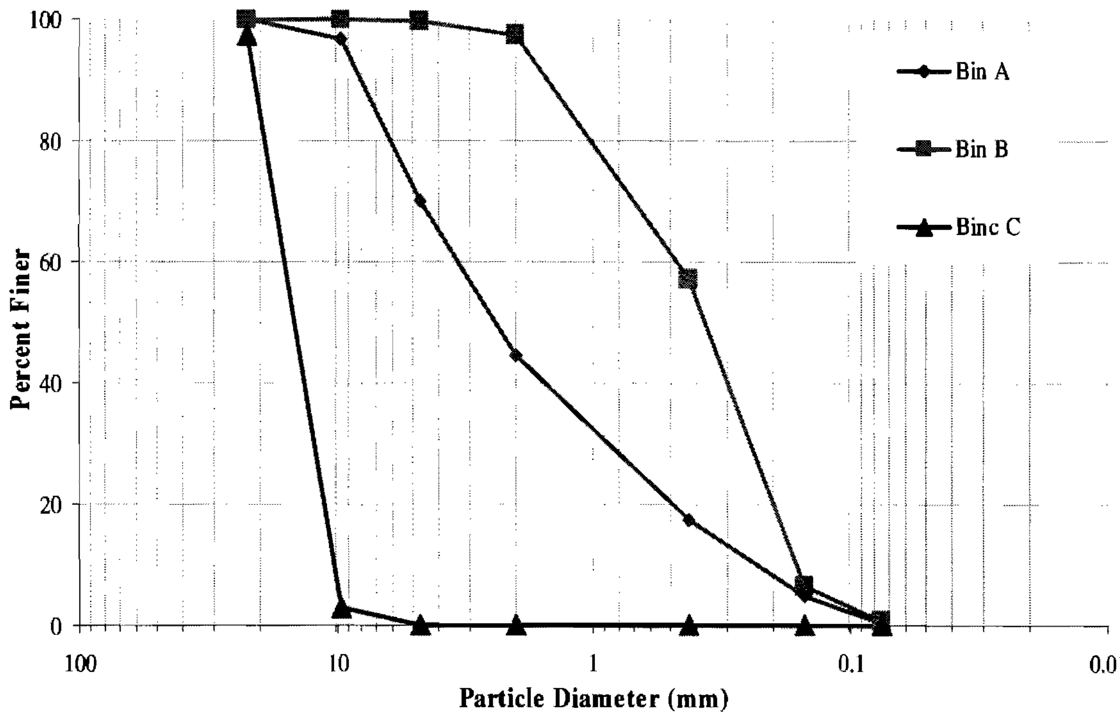


Figure 3.1 - Gradations of Bin A, Bin B, and Bin C

Table 3.1 provides a summary of test performed on each mixture. All seven mixtures were mixed, wetted and tested for optimum moisture-density, moisture susceptibility, strength, modulus, and permanent deformation. Aside from classification, and index tests, strength and deformation tests were used to study the behavior of these materials. Seismic tests were performed to measure the modulus of the material and its susceptibility to moisture. The Texas Triaxial Tests (Test Method Tex-143-E and Tex-117-E) were performed to classify the materials. The resilient modulus and permanent deformation tests were used to determine the stiffness and rutting performance of the base material, respectively.

GRADATION OF MIXES

Cooper et al. (1985) provided a relationship for maintaining structural stability of a given material as the fines content is changed. Cooper et al.'s relationship, which was used in this study, is in the form of

$$P = \frac{(100 - F)(d^n - 0.075^n)}{(D^n - 0.075^n)} + F \tag{3.1}$$

where P = percentage passing a sieve of size d in mm, F = percentage of material passing through a 0.075 mm sieve (i.e., fines content), d = sieve size (mm), D = maximum particle size (mm), and n = power relationship (typically 0.45).

Table 3.1 - Tests Performed on Base Material Mixtures

Test	TxDOT Specification	Property Measured
Particle size analysis	AASHTO Designation M 145-91	Gradation
Determination of Liquid limit	Tex-104-E	Atterberg Limits
Determination of Plastic Limit	Tex-105-E	
Determination of Plasticity Index	Tex-106-E	
Laboratory Compaction Characteristics and Moisture-Density Relationship of Base Materials	Tex-113-E	Moisture Density Relationship
Triaxial Compression for Disturbed Soils and Base Materials	Tex-117-E	Triaxial Strength
Determination of Shear Strength Parameters of Laboratory Compacted Soil and Base Material	Tex-143-E	
Moisture Susceptibility	Tex-145-E	Variations in Modulus with Moisture
Permanent Deformation	--	Permanent Deformation
Resilient Modulus	--	Resilient Modulus

Equation 3.1 ensures that for a predetermined fine content, F, the material can obtain the densest state. Using this relationship, four different gradations were created. The mixes had a fines content F = 0%, 5%, 10%, and 20%. The mixes containing 5%, 10%, and 20% low-plasticity fines were altered to create the last three of the seven mixes by replacing the fines content with plastic fines while maintaining the same gradation. The final gradations for the seven mixes are given in Table 3.2. The selected mixtures and the extreme limits of current specifications according to Items 247 and 245, as proposed, are shown in Figure 3.2. All mixtures meet the Item 245 specifications.

Table 3.2 - Gradation of Blends

Sieve	Particle Diameter (mm)	Acceptable Limits (% Passing) from Item 245		Acceptable Limits (% Passing) from Item 247		%Passing (P)			
		Low	High	Low	High				
#200	0.075	10	5	--	--	0	5	10	20
#100	0.150	--	--	--	--	3	7	12	22
#40	0.425	30	5	30	15	8	13	17	27
#4	4.7510	55	25	55	35	38	42	45	51
$^3/_8$ in	9.525	70	45	70	50	55	57	60	64
1.75 in	31.750	100	100	100	100	100	100	100	100

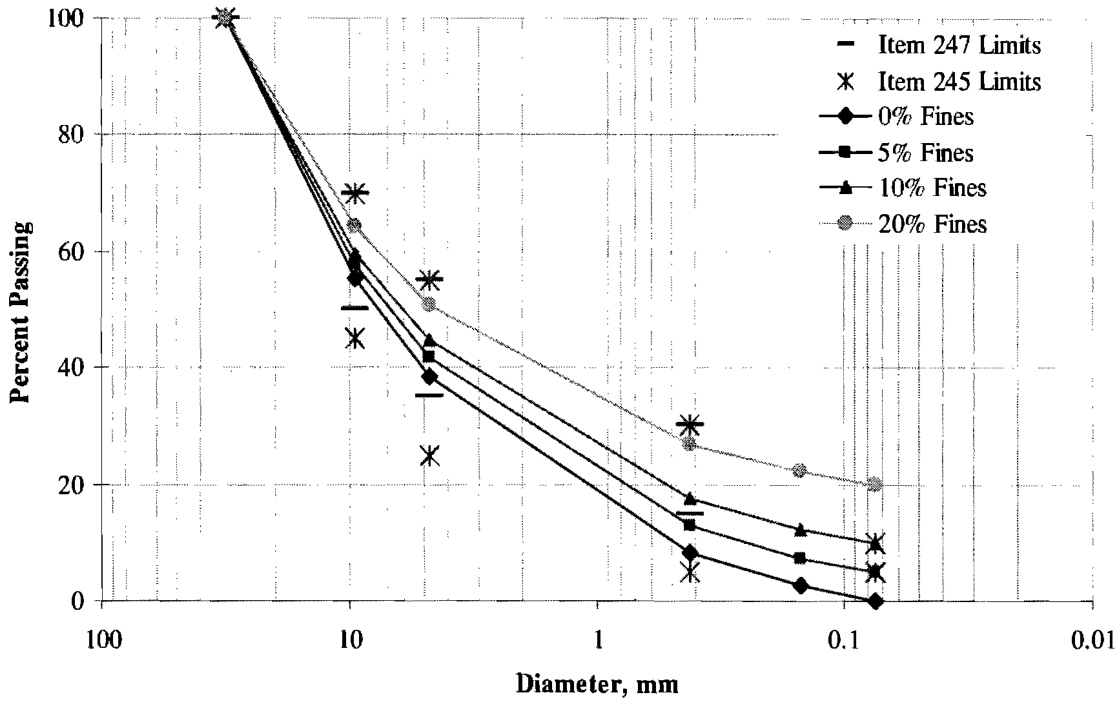


Figure 3.2 - Gradation of Control Blend and Selected Mixtures

The clay used was also crushed so that it would pass the #4 sieve. This ensured that the clay was of a fine enough consistency so that it could be distributed evenly when the mixture was wetted. Once all the required materials had been sieved and the clay crushed the seven mixtures were created by mixing the predetermined proportions.

TEST PROCEDURES

Moisture-Density and Moisture-Modulus Relationship

The maximum dry density and optimum moisture content were determined for each of the seven mixtures as per Test Method Tex-113-E. According to this method the relationship between water content and the dry unit mass (density) of base materials was determined. A typical result from the mixture containing 0% fines is shown in Figure 3.3. In this case, the optimum moisture content and the maximum dry unit weight were 3.6% and 139.1 pcf, respectively.

The specimens used to determine the moisture-density relationship were also used to determine the seismic modulus with the Free-Free Resonant Column (FFRC) device according to Test Method Tex-145-E. Originally developed for Portland cement concrete specimens, the method has been adapted for base and subgrade materials through hardware and software modifications.

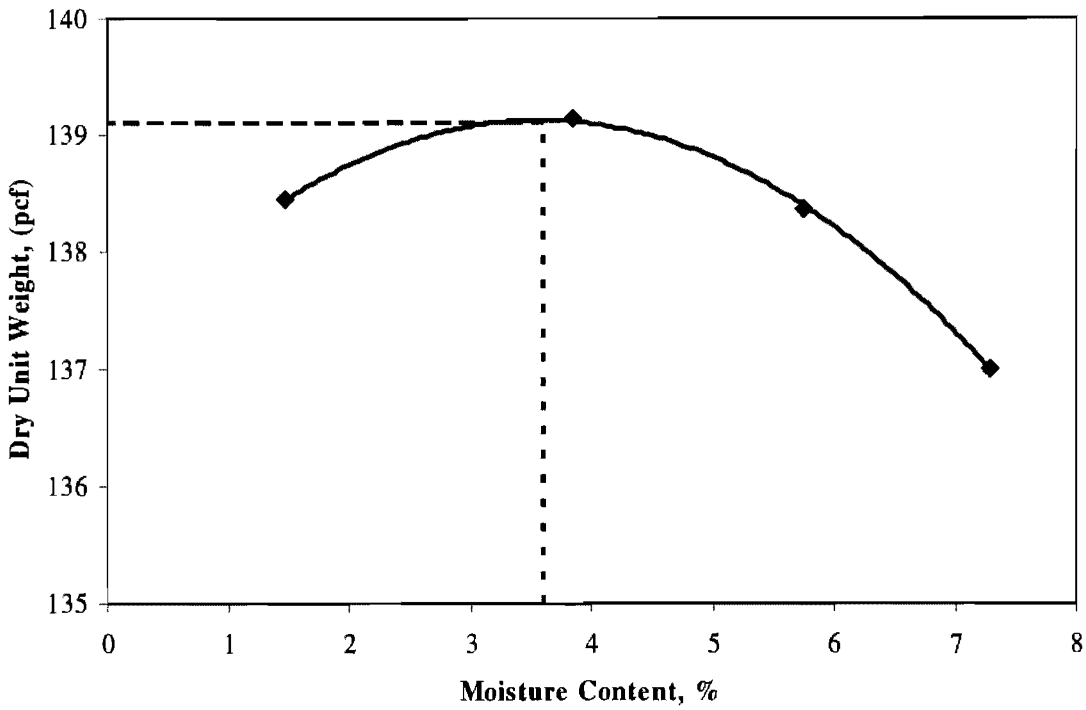


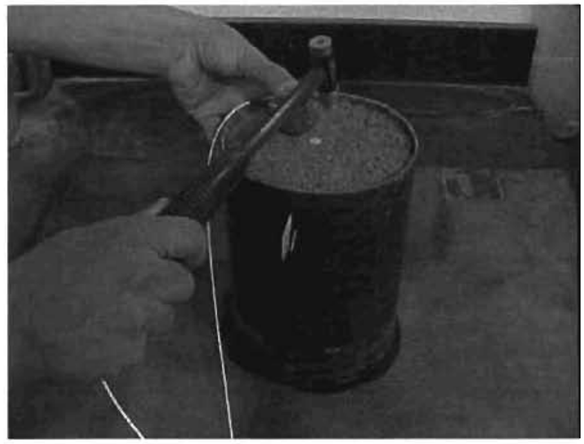
Figure 3.3 - Typical Variation in Dry Density with Moisture Content

In FFRC tests seismic energy propagates over a large range of frequencies when an impulse load is applied to the specimen. The energy associated with one or more frequencies are trapped and magnified (resonate), which depends on the dimensions and stiffness of the specimen. The specimen dimensions can easily be measured, and when combined with the resonant frequencies, the modulus of the specimen can be determined using the principles of wave propagation in a solid rod (Richart et al. 1970).

The schematic of the test set up is shown in Figure 3.4. Performing the test is simple. An accelerometer is placed securely on top of the specimen, and the top is tapped with a hammer that has a load cell attached to it. Both sensors are connected to a data acquisition system that is located in a laptop computer. Software has been developed to acquire and manipulate the time records from the accelerometer and the load cell. Figure 3.5 shows a typical time record for the load cell and accelerometer. The load consists of a short-duration half-sine pulse. The response measured with the accelerometer contains an oscillation that corresponds to the standing wave energy trapped within the specimen.

The frequency of oscillation can be determined by transforming the two signals into the frequency-domain using a fast-Fourier transform and then normalizing the acceleration amplitude with the load amplitude. The variation of normalized amplitude as a function of frequency, which is called a transfer function, contains peaks that correspond to the oscillation of

(a)
Testing & Data Acquisition Equipment



(b)
Testing of Specimen

Figure 3.4 - Free-Free Resonant Column Device and Testing

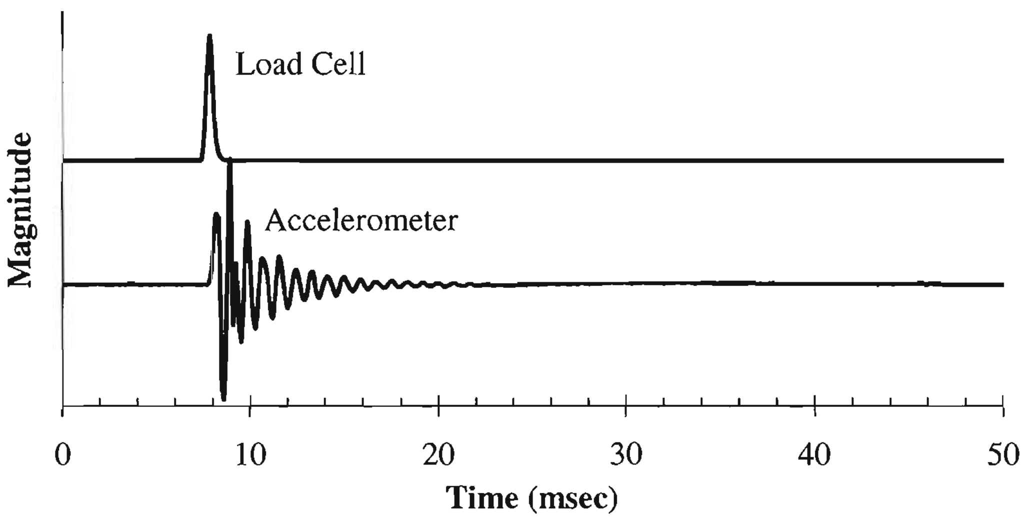


Figure 3.5 - Typical Load Cell and Accelerometer Response

the standing waves. A typical transfer function is shown in Figure 3.6 with the peak frequency clearly marked. Knowing the resonant frequency, f_P, mass density, ρ, and the length of the specimen, L, Young's modulus, E, can be found using:

$$E = \rho(2f_P L)^2 = \rho(V_P)^2 \tag{3.2}$$

where V_P is the compression wave velocity.

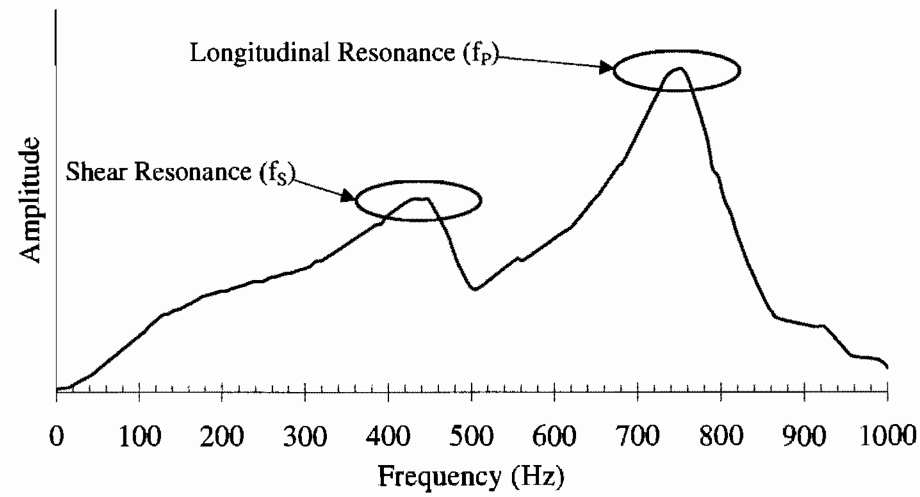


Figure 3.6 - Typical Transfer Function

In much the same way that the moisture-density curve is developed a moisture-modulus relationship can also be developed. In this manner, the water content at which the maximum seismic modulus is obtained can also be determined. Alternatively, the seismic modulus at the traditional optimum moisture content can be estimated. Figure 3.7 is an example of the moisture-modulus curve. The maximum modulus, which occurs at a moisture content of 3.2%, is approximately 55 ksi whereas the modulus at the traditional optimum moisture content of 5.4% is approximately 46 ksi.

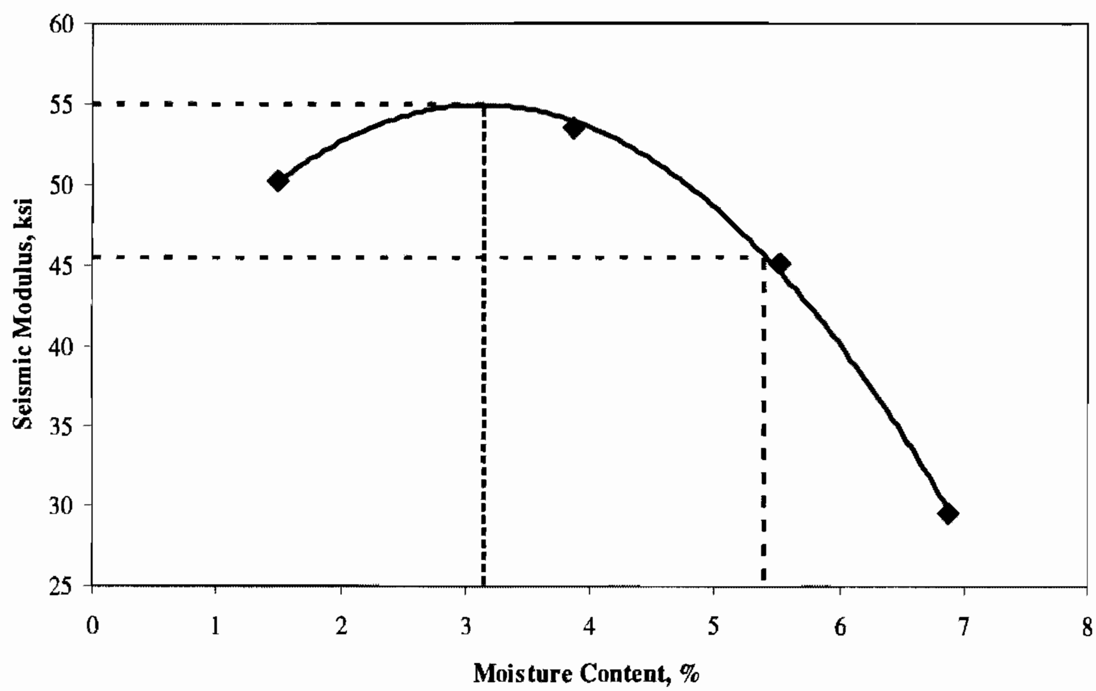


Figure 3.7 - Typical Variation in Modulus with Moisture Content of Control Blend

Moisture Susceptibility

TxDOT has begun to use the Tube Suction Test (TST) to evaluate the moisture susceptibility of granular bases. As previously described in Chapter 1, the amount of moisture found in a base as well as the amount of moisture that it attracts to itself can have detrimental effects on the base material and the overall pavement system. Unfortunately, the TST device was not available at UTEP. As part of this study, similar to tube suction tests, the moisture susceptibility of the base material was evaluated by using a series of measurements from the FFRC tests.

To establish the moisture susceptibility of the base with FFRC, one specimen of each blend was compacted in a concrete cylinder mold at optimum moisture content according to Tex-113-E. The concrete mold assists in the reduction of the loss of material during testing, and also reduces the probability that a specimen could break during its handling throughout the test.

Figure 3.8 illustrates the procedure. After the seismic modulus of a specimen is obtained on the day of compaction, it is placed in an oven at 60°C for four days to allow the specimen to dry. After the fourth day, the specimen is placed in a water bath to soak moisture through capillary action for six days. While drying or soaking, the specimen is weighed every twenty-four hours to determine the bulk moisture loss or gain, and then is tested with the FFRC device.

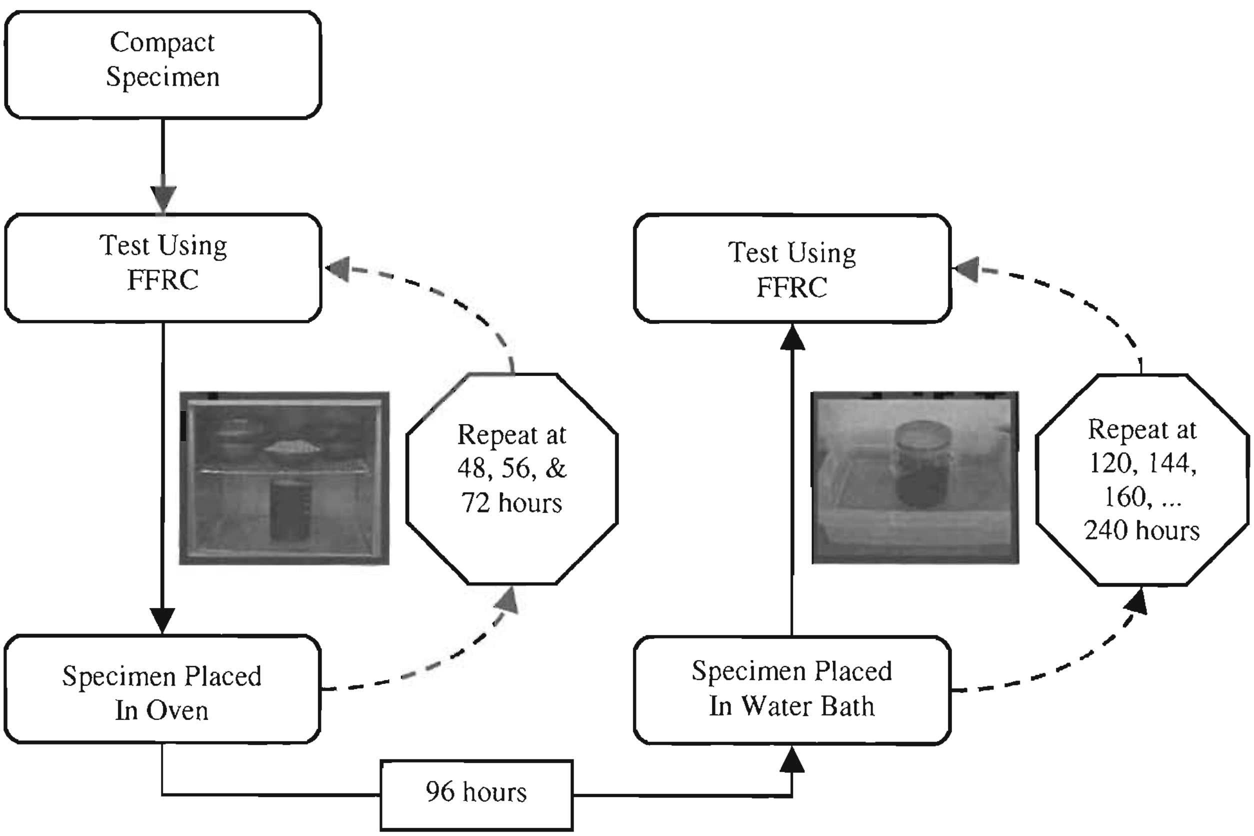


Figure 3.8 - Schematic of Protocol for Moisture Susceptibility with FFRC Device

Typical responses from the FFRC and the variations in moisture content over the 10-day time span are shown for the specimen containing 5% fines in Figure 3.9. During the drying period, the modulus increases as the moisture content decreases. However, a sudden drop in modulus occurs after the first day of soaking in the water bath. Within only a few days in the water bath the base material has absorbed enough water such that the modulus is very close to the residual modulus of the specimen. The residual modulus is considered the average of the modulus values during the test that are near-constant, most commonly the last three readings taken during the moisture susceptibility. Nazarian et al. (2003) in Project 1735 demonstrate that the ratio of the peak modulus to the residual modulus is the best indicator of the performance of a base material.

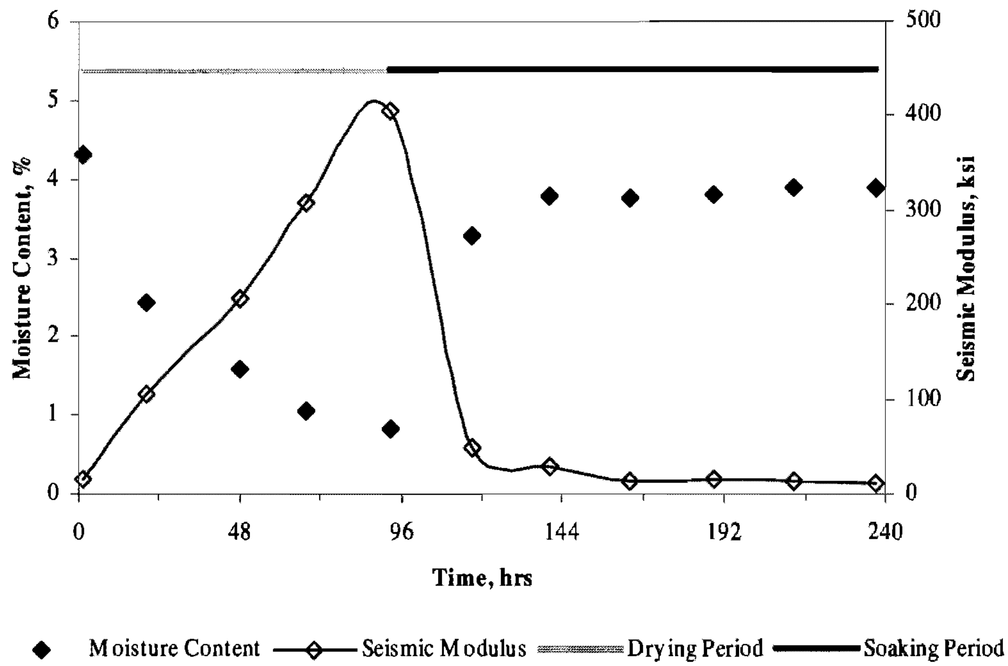


Figure 3.9 - Variations in Modulus and Moisture Content with Time

Texas Triaxial Tests

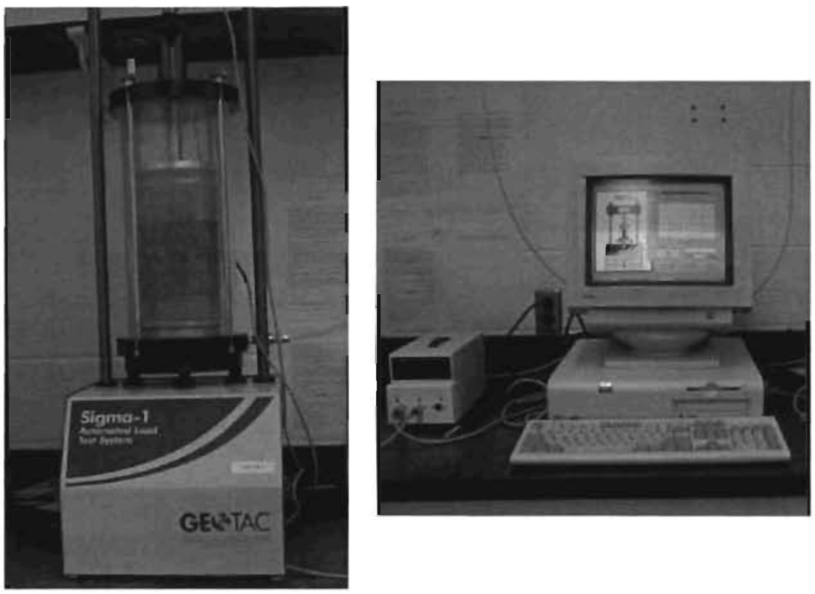
TxDOT currently categorizes base materials according to the results obtained from the Texas Triaxial test. In this test, the shear strength of the soil at several confining pressures is measured. Under the Texas Triaxial tests, a base material can be classified as Class 1 to Class 6 with Class 1 being of the highest quality. Two methods are currently available to obtain the shear strength of soils Tex-143-E and Tex-117-E. All seven base blends were tested according to both specifications.

Tex-143-E Tests

Three specimens of each blend are compacted at the optimum moisture content. Each specimen nominally measures six inches in diameter and approximately eight inches in height. All specimens are moistened, mixed, molded and finished so that their properties would be as nearly identical as possible. All specimens are then encased in a rubber membrane and allowed to mature for at least 24 hours prior to testing.

One specimen is tested at confining pressure of 3 psi, 7 psi or 10 psi. Specimens are tested under an increasing load of 1% strain per minute while the stress-strain diagram of material is recorded up to failure. With this information the Mohr diagram is constructed and the failure envelope (common tangent line to all Mohr circles) is found. The end results of the test are the angle of internal friction, cohesion and the classification.

The setup for this test is shown in Figure 3.10. A GEOTAC Sigma-1 Automated Load Test System was used, and relevant data during the test was acquired using the software provided by the manufacturer. Air was used as the confining medium. The system is outfitted with a load cell as well as internal LVDT to measure the compression load and deformation. Typical results are shown in Figure 3.11.



(a)
Testing Equipment

(b)
Data Aquisition

Figure 3.10 - Test Setup for Triaxial Tests

Tex-117-E Tests

Under this specification, seven specimens have to be prepared for testing. However, for this project the specification was modified so that only four specimens of each base mixture were prepared and tested. Once more, the specimens are prepared at the optimum moisture content according to Tex-113-E similar to Tex-143-E. The four specimens are allowed to dry at room temperature for twenty-four hours, and then subjected to ten days of capillary wetting before they are tested. During these ten days the specimens are exposed to a constant pressure of 1 psi and loaded with a surcharge load.

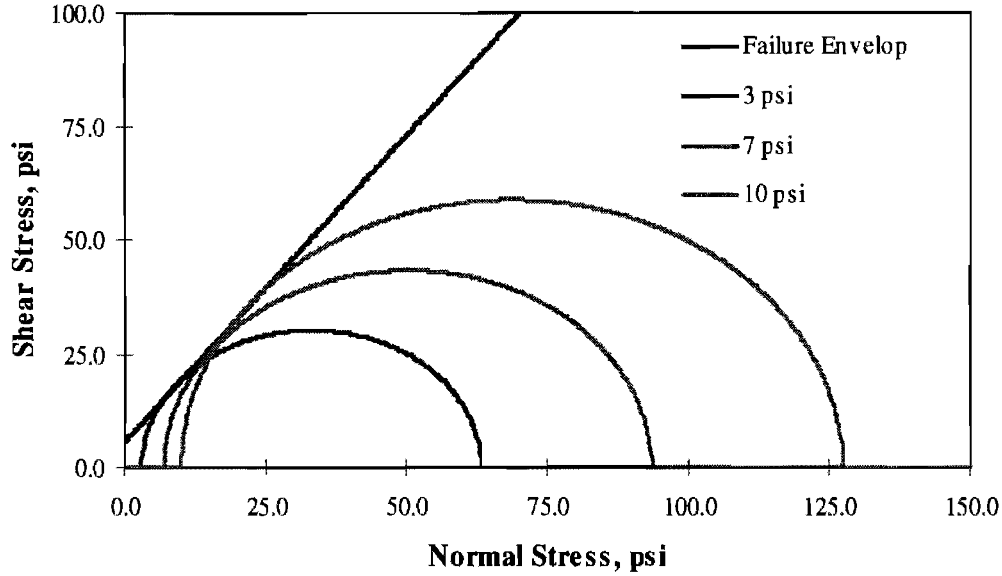


Figure 3.11 - Typical Mohr Diagram for Tex-143-E Triaxial Tests

After the soaking period, the specimens are tested at four confining pressures; 0 psi, 5 psi, 10 psi and 15 psi. Specimens are compressed under an increasing load set at 2% strain per minute while the load and deformation of the material were recorded up to failure. Once more, the Mohr circles for the four tests are plotted and the angle of internal friction, cohesion and the Texas Triaxial classification of the material are determined. Typical results from this method are shown in Figure 3.12.

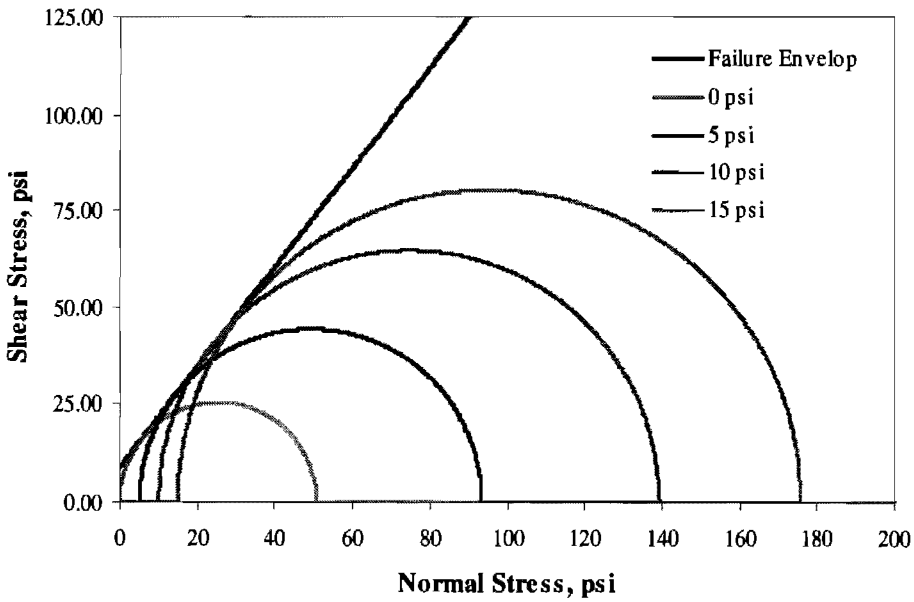


Figure 3.12 - Typical Mohr Diagram for Tex-117-E Triaxial Tests

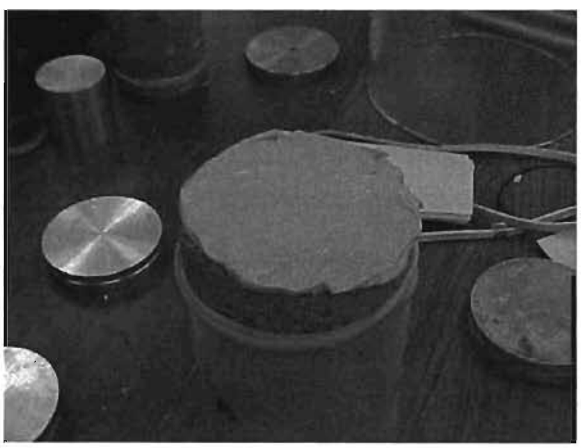
Resilient Modulus Tests

The resilient properties of the base materials are determined using a repeated load triaxial test. Repeated loading properties such as those determined through the resilient modulus test and permanent deformation of base materials are major factors that influence the response and performance of flexible pavements to the dynamic loadings they experience. The results of the resilient modulus test provide a relationship between stiffness and stress of the base material tested. The manner in which this test is carried out provides a modulus that corresponds to a loading cycle that is similar to that which the pavement system will experience in the field.

As with the previous tests, several specimens of each blend are prepared at the optimum moisture content. The specimens, which are six inches in diameter and approximately twelve inches in height, are compacted in six layers. After they are extruded from the mold, each specimen is encased in a rubber membrane and allowed to mature for a minimum of 24 hours prior to testing.

Prior to testing each specimen, two platens (top and bottom) are adhered to the specimen with a thin layer of grout as shown in Figure 3.13. The grout ensures that the bottom and top of the specimen are level. It also ensures that any deformations in the surface of the specimen that would otherwise cause an eccentric loading are eliminated. The specimen is then encased in a second rubber membrane to ensure no moisture loss or air leakage occurs during testing. Finally, the membranes are secured to the platens by sealing them with vacuum grease and placing o-rings over the membranes.

The loading sequence is provided in Table 3.3. After the specimen is placed in the test apparatus and centered, a series of confining pressures are applied. At each confining pressure, a 0.1-second haversine deviatoric load is repeatedly applied to the specimen. A 0.9-sec rest period is allowed between each load cycle. The test sequence was followed until the specimen failed or the specimen experienced 5% total permanent strain.



(a)
Grout for top plate

(b)
Leveling top platen

Figure 3.13 - Grouting of Resilient Modulus and Permanent Deformation Specimens

Table 3.3 - Loading Sequence for Resilient Modulus

Sequence	Confining Pressure		Contact Stress		Cyclic Stress		Maximum Stress		N_{rep}
	kPa	psi	kPa	psi	kPa	psi	kPa	psi	
Conditioning	103.5	15	10.4	1.5	93.1	13.5	103.5	15.0	1000
01					18.6	2.7	20.7	3.0	
02	20.7	3	2.1	0.3	41.4	6.0	41.4	6.0	25
03					62.1	9.0	62.1	9.0	
04					31.1	4.5	34.5	5.0	
05	34.5	5	3.4	0.5	69.0	10.0	69.0	10.0	25
06					103.5	15.0	103.5	15.0	
07					62.1	9.0	69.0	10.0	
08	69	10	6.9	1.0	138.0	20.0	138.0	20.0	25
09					207.0	30.0	207.0	30.0	
10					58.6	8.5	69.0	10.0	
11	103.5	15	10.3	1.5	103.5	15.0	103.5	15.0	25
12					207.0	30.0	207.0	30.0	
13					89.7	13.0	103.5	15.0	
14	138	20	13.8	2.0	138.0	20.0	138.0	20.0	25
15					276.0	40.0	276.0	40.0	

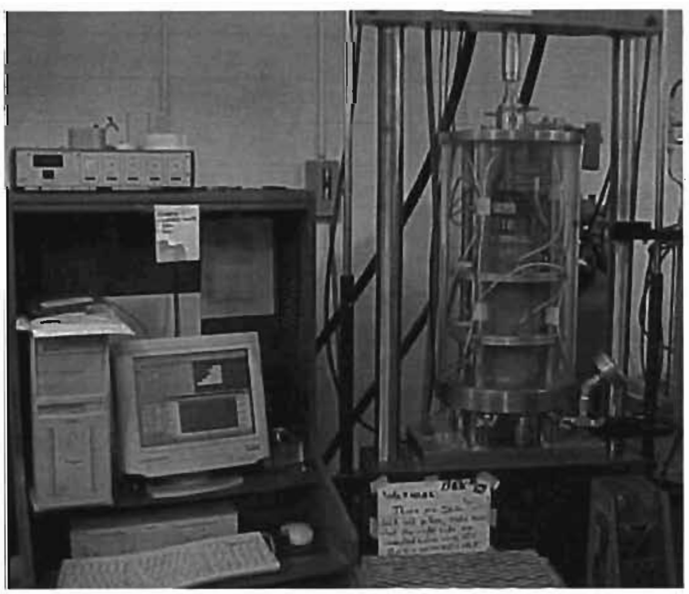


Figure 3.14 - Resilient Modulus and Permanent Deformation Test Set-up

The test setup used for resilient modulus tests is shown in Figure 3.14. An MTS Load Test System is used, and the required data during the test is acquired using the ATS data acquisition software. Air was used as the confining medium. The applied load and permanent and resilient deformations are recorded during testing. A 2000-lb load cell records the applied load. Six proximeters and/or two LVDTs measure the deformation of the specimen. From the measured loads and deformations, the applied stresses, applied strains as well as the resilient moduli are obtained.

Typical variation in resilient modulus with deviatoric stress and confining pressure for one specimen is shown in Figure 3.15. To obtain the nonlinear parameters for the specimen, a curve fitting routine is used to obtain parameters k_1, k_2 and k_3 associated with the following equation

$$M_r = k_1 \times \sigma_c^{k_2} \times \sigma_d^{k_3} \tag{3.2}$$

where M_r = resilient modulus, k_i = regression constants, σ_c = confining pressure, and σ_d = deviatoric stress. Once parameters k_1, k_2 and k_3 are obtained, the resilient modulus of the material at any state of stress can be conveniently determined.

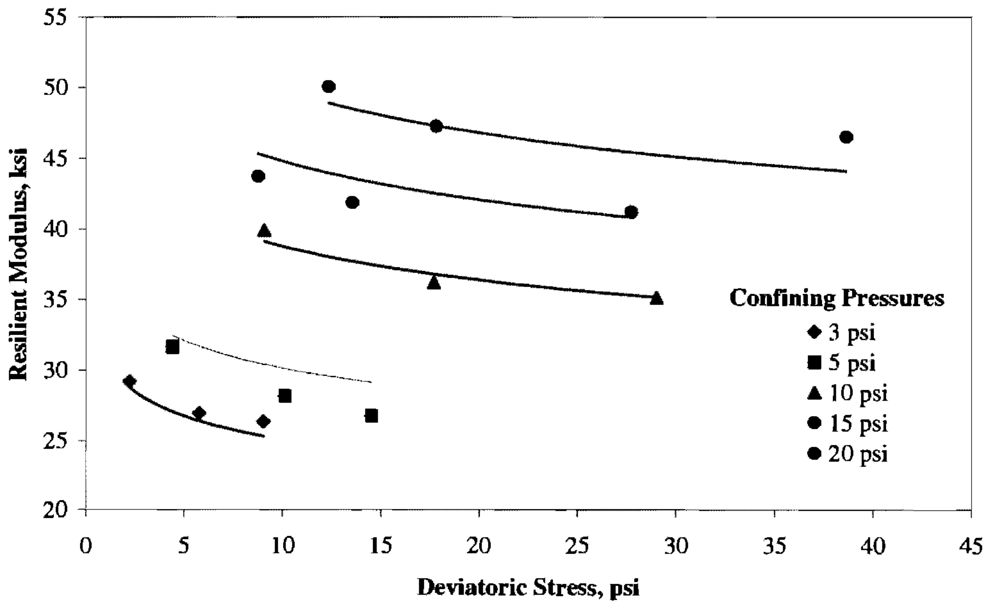


Figure 3.15 - Typical Resilient Modulus Test Results

Permanent Deformation

The permanent deformation properties of the base material were also determined from a repeated load test. The information obtained from the permanent deformation test is important in pavement design due to its capability to predict the rutting performance of the pavement system. The permanent deformation tests just as the resilient modulus tests, simulates the moving wheel loadings that a pavement would experience.

The specimen preparation for this test is identical to the one described for the resilient modulus tests. Once the specimen is placed in the testing chamber, 100 conditioning cycles followed by 10,000 repetition cycles are applied to the specimen (see Table 3.4). The deformation is recorded by two LVDTs. Table 3.5 contains the intervals at which the deformations are recorded. The final deformation at each interval is the average of the two LVDTs' readings. With this information the rutting parameters α and μ are determined. Although the standard number of cycles is 10,000 the test is stopped if the specimen failed or the permanent strain reached 5% during testing.

Table 3.4 - Loading Sequence for Permanent Deformation

Sequence	Confining Pressure		Contact Stress		Cyclic Stress		Maximum Stress		$N_{rep.}$
	kPa	Psi	kPa	psi	kPa	psi	kPa	psi	
Conditioning	103.5	15	20.7	3.0	20.7	3.0	41.4	6.0	100
Permanent Deformation	103.5	15	20.7	3.0	93.2	13.5	113.9	16.5	10,000

Table 3.5 - Data Acquisition Intervals for Permanent Deformation

Data Collection During Cycles			
1-15	450	1,300	4,000
20	500	1,400	4,500
30	550	1,500	5,000
40	600	1,600	5,500
60	650	1,700	6,000
80	700	1,800	6,500
100	750	1,900	7,000
130	800	2,000	7,500
160	850	2,200	8,000
200	900	2,400	8,500
250	950	2,600	9,000
300	1,000	2,800	9,500
350	1,100	3,000	10,000
400	1,200	3,500	

With the recorded deformation information, permanent deformation and rutting parameters are determined following the steps below.

1. The cumulative axial permanent strain and resilient strain, ε_r, are determined at the 200th repetition cycle.
2. A log-log plot is created of the cumulative axial strain versus the number of cycles and the permanent deformation parameters (i.e., intercept, a, and slope, b) are determined as shown in Figure 3.16.

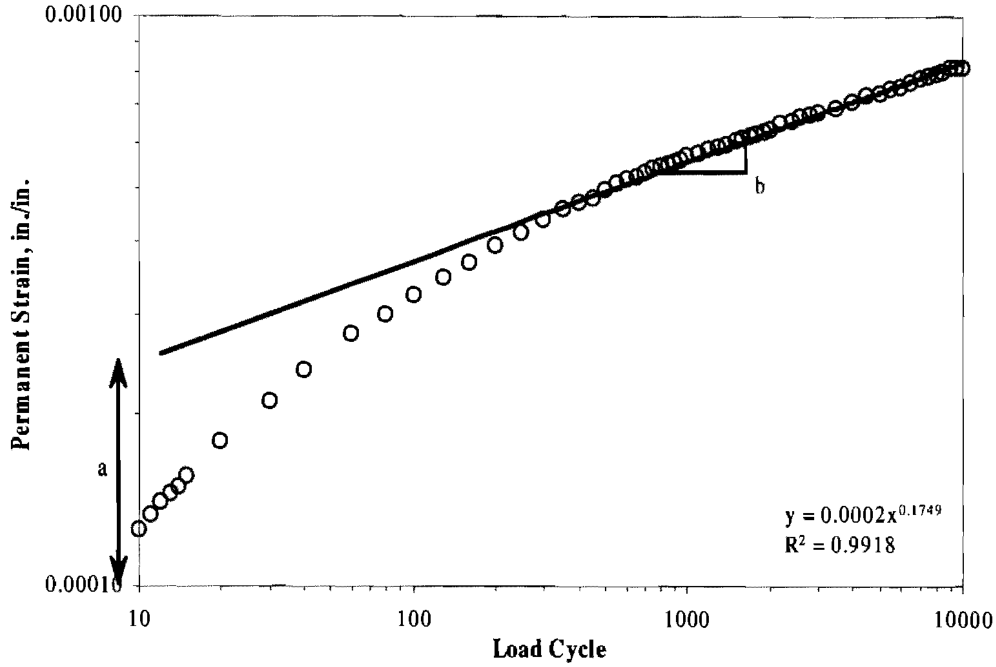


Figure 3.16 - Typical Determination of Permanent Deformation Parameters, a and b

3. Using the permanent deformation parameters, a and b, the rutting parameters α and μ were determined using the following equations

$$\alpha = 1 - b \tag{3.3}$$

$$\mu = \frac{a \times b}{\varepsilon_r} \tag{3.4}$$

RESULTS

Moisture-Density and Moisture-Modulus Relationship

A summary of the results for the moisture-density and moisture-modulus relationships is included in Table 3.6. Up to a fines content of 10%, the optimum moisture content increases for both low-plasticity and high-plasticity fines. However, when 20% fines are added, the optimum moisture content is lower than when 10% fines are added. This perhaps occurs because less lubricating agent is needed for a dense pack when the fines contents are high. The maximum dry densities of all specimens containing fines are more or less the same but significantly higher than the control mix (with no fines). Once again, the maximum dry density is greatest at 10% fines content for both low and high plasticity fines.

The moduli at the optimum moisture content measured 24 hours after the compaction of the specimens are also reported in Table 3.6. As the fines content increases, the modulus typically increases as well. The maximum modulus is achieved for specimens with 20% fines content for both low and high plasticity fines. The highest modulus was measured for the specimen with the

Table 3.6 - Maximum Dry Density and Modulus of the Blends

Percent Fines in Blend	Moisture Density		Moisture-Modulus		
	Optimum Moisture Content (%)	Maximum Dry Unit Weight (pcf)	Modulus at Optimum Moisture Content (ksi)	Moisture Content @ Maximum Modulus (%)	Maximum Modulus (ksi)
0%	3.6	139.1	40.1	4.0	40.8
5%	4.3	143.2	40.8	3.8	41.4
10%	6.0	144.0	37.0	3.2	43.6
20%	5.4	143.8	44.7	2.6	55.3
5% Clay	4.9	144.8	37.2	3.5	37.4
10% Clay	5.1	145.6	46.8	4.0	48.7
20% Clay	4.6	143.0	53.3	3.6	55.1

high-plasticity fines. As anticipated, the maximum modulus occurs at a moisture dry of optimum moisture content.

Moisture Susceptibility

As explained previously, the moisture susceptibility of each mixture was evaluated using the FFRC tests. The procedure included exposing the specimen to both extreme wet and dry conditions through placement in a water bath and oven. Each specimen was weighed and tested daily for seismic modulus. As reflected in Figure 3.17, all specimens followed a similar trend. The maximum modulus occurred on the last day of drying followed by a dramatic drop in modulus after soaking.

Table 3.7 summarizes the moisture susceptibility test for each blend. This table provides the initial modulus (modulus after 24 hours air drying), peak modulus and residual modulus. The lowest peak modulus was obtained from the blend that contained no fines. The maximum peak modulus was obtained for the specimens with 10% low-plasticity fines and 5% high-plasticity fines.

For a well performing base, Project 0-1735 recommends an initial modulus of greater than 100 ksi. Based on these criteria, the blends with 5% to 10% low-plasticity fine are acceptable. The initial moduli from other cases are way below 100 ksi. In addition, the low maximum-to-initial modulus ratios for these two blends ensure that excessive drying will not occur during construction.

For the high-plasticity clays, the drop from the peak to residual strength occurs at shorter time, indicating the adverse impact of high-plasticity fines. The peak-to-residual modulus ratio is in all cases greater than 10. Project 0-1735 recommends a peak-to-residual modulus ratio of less than 5. In that sense, the long-term performance of this particular base may be of concern.

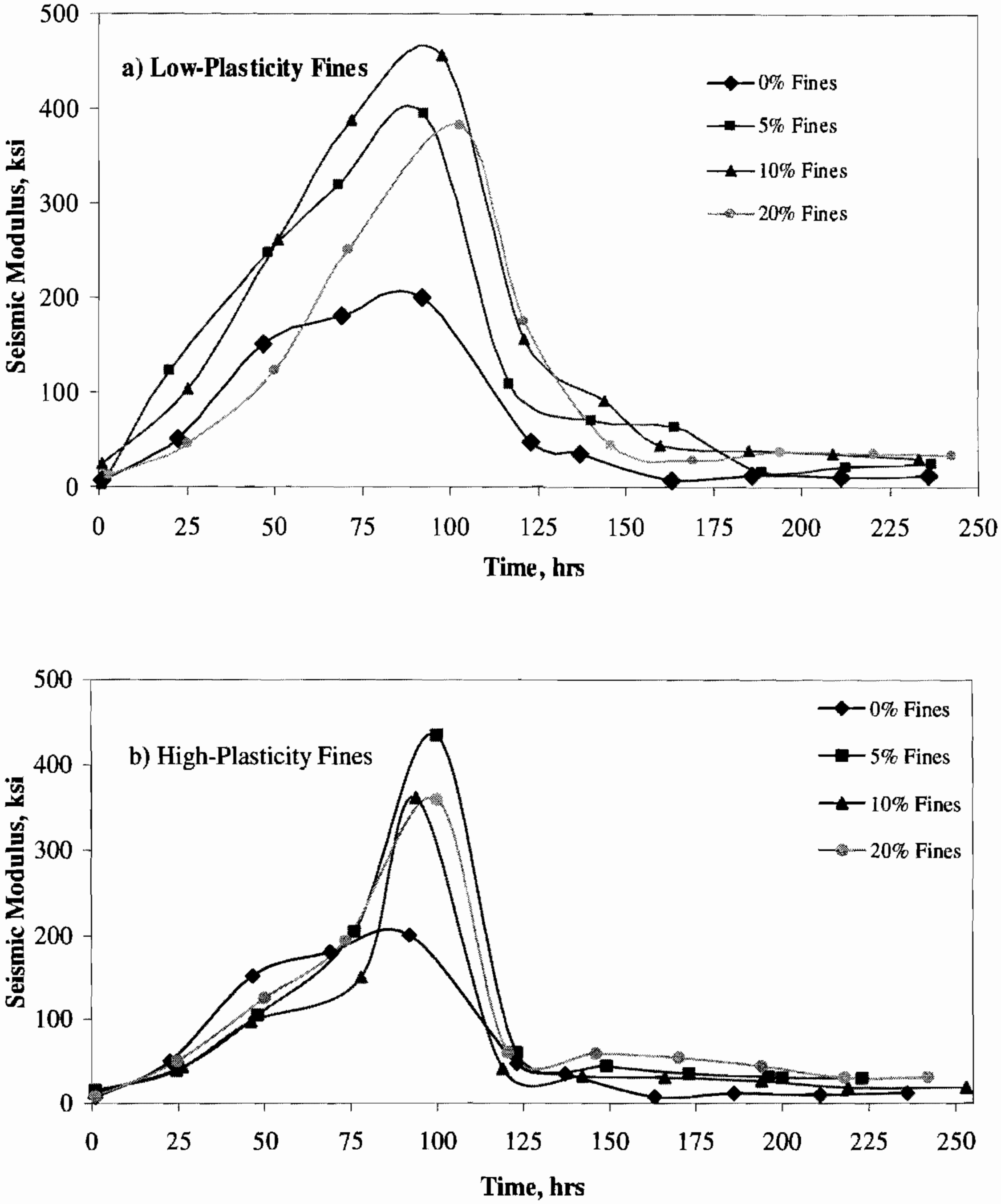


Figure 3.17 - Variations in Modulus with Time for Different Blends

Table 3.7 - Moisture Susceptibility Results of Blends

Percent Fines in Blend	Initial Modulus (ksi)	Peak Modulus (ksi)	Peak to Initial Modulus Ratio	Residual Modulus ksi	Peak to Residual Modulus Ratio
0%	50	360	7	35	10
5%	123	459	4	21	22
10%	104	456	4	35	13
20%	45	383	9	35	11
5% Clay	39	435	11	31	14
10% Clay	43	362	8	21	17
20% Clay	50	360	7	35	10

The water retention of the bases can be inspected in Figure 3.18. For the blends with 0% and 5% low-plasticity fines, the moisture content is practically equal to zero. This means that all the free-water has evaporated after 4 days of oven drying. However, the blends with 10% and 20% low-plasticity fines retained about 1% of the moisture. For the blends with high-plasticity fines, the pattern is similar. For a well-performing material, the ability of losing all the free-water is desirable. In that sense all blends seem to be reasonably pervious except perhaps the blend with 20% high-plasticity fines. After ten days, all specimens soak up to 4% moisture through capillary rise.

Triaxial Compression Tests

Table 3.8 contains the results from the two triaxial test methods. The results from Tex-143-E are more representative of the condition of the base as constructed, and those from Tex-117-E correspond to the long-term behavior of the material.

According to Tex-143-E (see Figure 3.8a), the angles of internal frictions from all blends are around 55 degrees, except for the control blend (0% fines) and the blend with 20% high-plasticity fines. The cohesion on the other hand is variable, perhaps because of the curve fitting process. The best indicator of the quality of the base is perhaps the strength at 10 psi, as presented in Table 3.8a. For both low-plasticity and high-plasticity blends, the maximum strength are achieved when about 10% fines are added. The blends with the low-plasticity fines exhibit greater strength as well.

As shown in Table 3.8b, the blends containing 10% or less fines exhibit similar angles of internal friction of about 52 degrees. The blends with 20% fines exhibit substantially lower angles of internal friction as compared with the other blends. As anticipated, the reported cohesions generally increase as the fine contents increase. Once again, as reflected in the table, the highest strengths at 10 psi confining pressure are obtained at fines contents of about 10%, followed by those at 5%. Once again, indicating that fines on the order of 5% to 10% provide better-performing bases.

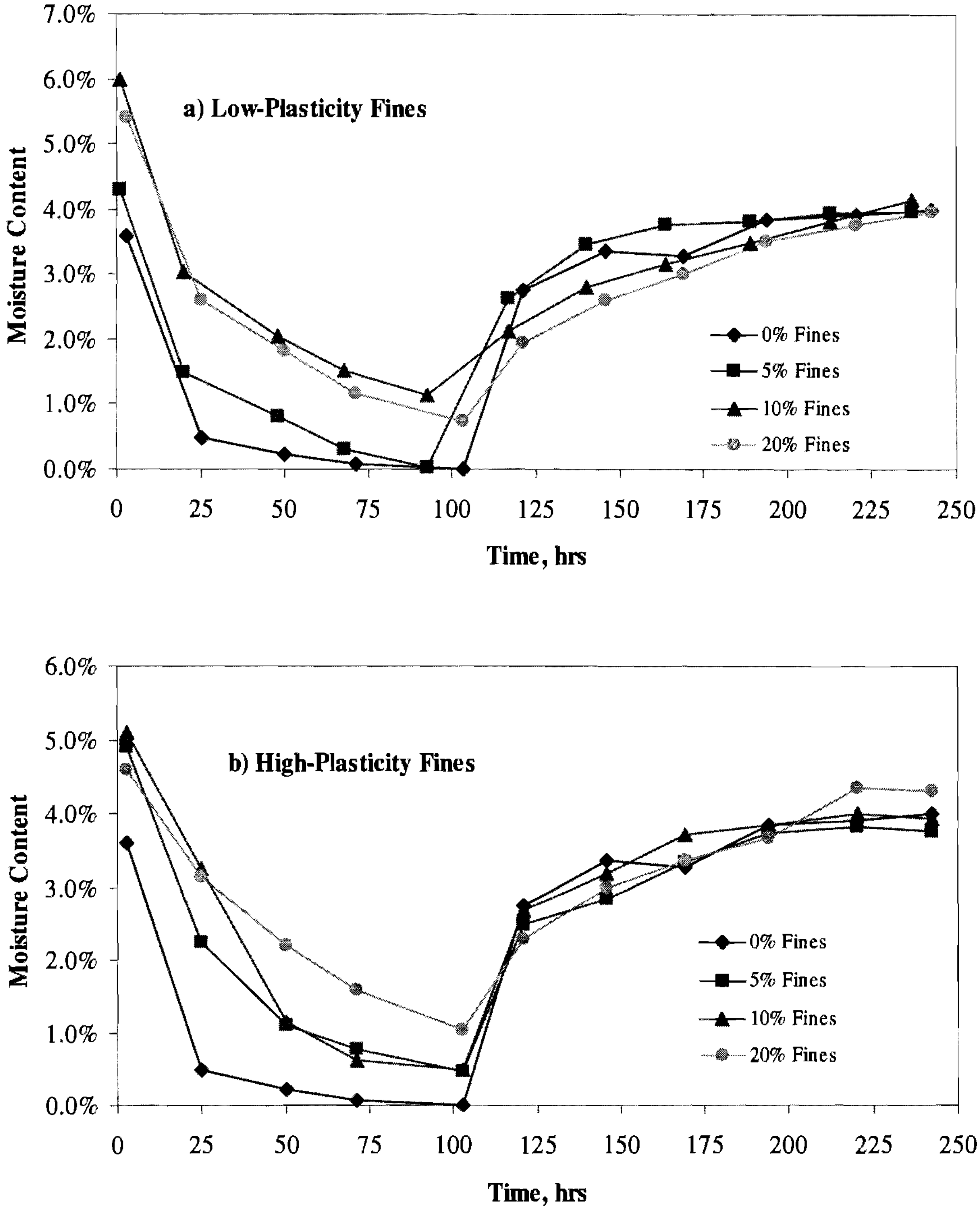


Figure 3.18 - Variations in Moisture Content with Time for Different Blends

Table 3.8 - Triaxial Results of Different Blends

a) According to Tex-143-E

Percent Fines in Blend	Angle of Internal Friction, degrees	Cohesion (psi)	Strength at 10 psi Confining Pressure (psi)
0%	41.4	10.2	40.9
5%	53.3	6.3	60.3
10%	54.1	9.1	71.2
20%	57.6	1.2	61.5
5% Clay	54.2	3.4	56.8
10% Clay	53.2	7.8	62.0
20% Clay	49.1	9.2	54.6

b) According to Tex-117-E

Percent Fines in Blend	Internal Angle of Friction	Cohesion (psi)	Strength at 10 psi Confining Pressure (psi)
0%	51.8	5.6	55.3
5%	52.2	8.8	64.7
10%	54.5	12.2	78.2
20%	33.2	20.8	50.6
5% Clay	52.7	7.6	61.1
10% Clay	51.0	12.8	73.2
20% Clay	42.5	12.3	52.4

Resilient Modulus Tests

The results from the resilient modulus tests are reported in Table 3.9. In general, the quality of the data collected was good, and the fitted models described the collected data well as judged by the R^2 values in excess of 0.94. The values for parameter k_2, which are indicators of the stress-hardening of base correspond well with typical base materials. As anticipated for typical base materials, the values of parameter k_3, which correspond to the strain softening of the material, are generally small.

Table 3.9 - Resilient Modulus Results from Different Blends

Percent Fines in Blend	Model Parameters			R^2	Resilient Modulus ksi
	k_1	k_2	k_3		
0%	22	0.42	-0.03	0.98	39.9
5%	25	0.36	-0.03	0.97	41.1
10%	22	0.44	-0.22	0.96	24.6
20%	15	0.54	-0.08	0.97	28.8
5% Clay	23	0.46	-0.07	0.98	39.9
10% Clay	18	0.27	-0.06	0.94	23.6
20% Clay	20	0.38	-0.09	0.97	28.9

To consistently rank order the blends, the resilient moduli at a confining pressure of 5 psi and a deviatoric stress of 15 psi were determined using Equation 3.2. These stresses are representative of a given base layer in Texas. For both low-plasticity and highly-plastic fines, the greatest moduli are obtained from the blends with 5% fines, and the lowest with 10% fines. The moduli from the blends with 20% fines are slightly higher than those of 10% fines. It seems that for the material tested, maintaining fines at a level close to 5% is desirable.

Permanent Deformation Tests

The results from the permanent deformation tests are included in Figure 3.19 and the relevant parameters are summarized in Table 3.10. The blend containing no fines exhibits minimal permanent deformation. This trend is anticipated because of intimate grain-to-grain contact between the aggregates with low fines content. The blends with 10% fines, both containing low-plasticity and high-plasticity fines, initially (for the first 2000 to 3000 load cycles) perform well. However, for higher load cycle repetitions, they exhibit large tertiary deformations, indicative of poor performance. Both blends containing 10% fines exhibited the largest permanent deformation. Further testing also provided similar results. The two blends that contained 20% fines exhibited large initial permanent deformations, but became stable passed 1000 cycles.

The resilient strain, which can be considered as the resistance of the mixture to permanent deformation is shown in Table 3.10 for each blend. The larger the resilient strain is, the more recoverable the deformations are. For the low-plasticity fines, the blend with 5% fines demonstrates the highest resiliency. However, for the highly-plastic fines the mixtures with 10% and 20% exhibit the highest resiliency.

In general, when all the parameters are considered, the blends with 5% to 10% fines perform better. Also the mixtures with low-plasticity fines seem to exhibit better performance.

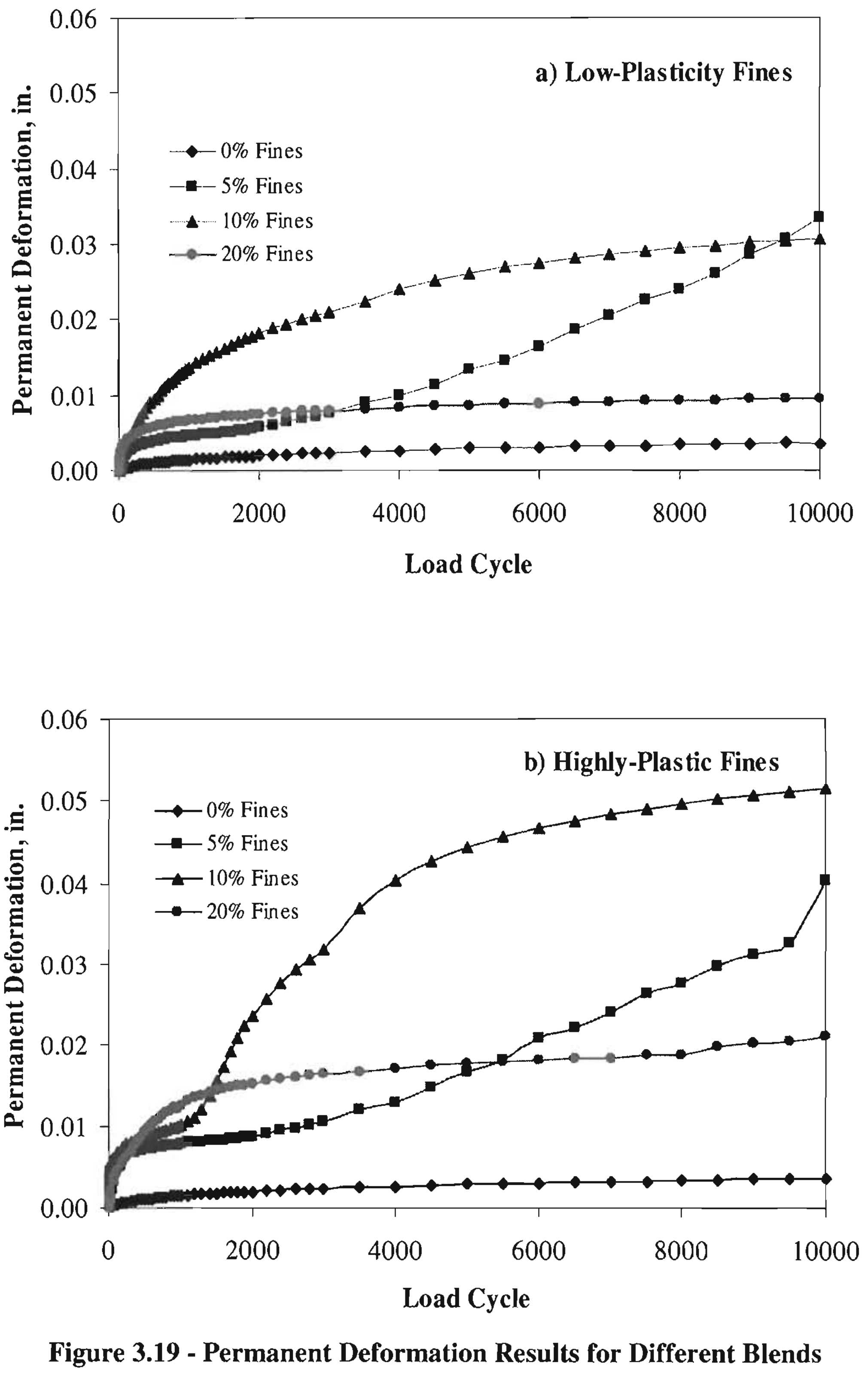


Figure 3.19 - Permanent Deformation Results for Different Blends

Table 3.10 - Permanent Deformation and Rutting Parameters for Different Blends

Percent Fines In Blend	Resilient Strain, Micro-strain	μ	α
0%	272	0.01	0.57
5%	398	0.04	0.73
10%	325	0.08	0.57
20%	300	0.07	0.81
5% Clay	294	0.10	0.87
10% Clay	379	0.10	0.81
20% Clay	375	0.07	0.61

CHAPTER FOUR

TEXAS A&M APPROACH

BACKGROUND

In the study conducted at Texas Transportation Institute, Texas A&M University, the influence of fines on the engineering properties is evaluated by an extensive laboratory testing of the Texas crushed stone, a typical Texas base material at varying fines content. This evaluation is based on material properties that contribute in preventing premature failure of granular bases and further increasing the performance of pavement. Laboratory tests were conducted at three different fines content, 5 percent, 10 percent and 17 percent while retaining the same percentages for the other sieve sizes. The laboratory tests that were conducted are the Tube Suction Test (TST) and the Permanent Deformation and Resilient Modulus Test. The significant engineering properties, which affect the performance of the flexible pavements, are moisture susceptibility, strength, resilient modulus and permanent deformation. This section provides a brief description of the historic work of the test methods used to determine these properties. Laboratory description of test procedures and their test results are presented subsequently. This is followed by an analysis of results and conclusions and recommendations.

Tube Suction Test

The TST was developed in a cooperative effort between the Finnish National Road Administration and the Texas Transportation Institute for assessing the moisture susceptibility of granular base materials (Guthrie, 2000). Moisture ingress degrades the engineering properties of aggregate base layers reducing the performance of the pavement. Research studies demonstrated that moisture susceptibility is related to both the matric and osmotic suction properties of aggregates. Matric suction is mainly responsible for the capillary phenomenon in aggregate layers, and osmotic suction is the suction potential resulting from salts present in the aggregate matrix.

Important factors for determining moisture susceptibility include soil suction, permeability, and the state of bonding of water that accumulates within the aggregate matrix. Soil suction is a measure of the affinity of a material for water, and permeability controls the rate of moisture migration within the aggregate layer. The state of bonding of water describes the structuring of the water molecules within the aggregate matrix. Water is classified as both bonded and unbonded moisture. The bound (adsorbed) water molecules are arranged in layers around aggregate particles, where the electrical attraction between water molecules is relatively strong. This moisture is very difficult to displace and generally does not have a large impact on base performance. The unbound (viscous or capillary) water is beyond the zone of electrical capture. This moisture is loosely bound to the aggregates but it can migrate within the base under the influence of environmental factors (freeze-thaw cycles) or heavy loads. It is the amount of unbound water in a base that influences the engineering properties in the field, including load carrying capability and resistance to freeze thaw cycles. The quantity and distribution of unbound water thus plays a very important role in moisture damage mechanism. The amount of unbound water that exists within an aggregate base material is directly related to the dielectric value of the base as measured in the TST (Guthrie, 2000, Guthrie, 2001).

Texas Triaxial Test

In this study the Texas Triaxial test is conducted as part of the TST. This is one of the advantages of TST wherein, the Texas triaxial test is merged within the TST enabling the determination of moisture susceptibility and strength on the same specimen. The TST should be conducted in a Texas triaxial cell and the bottom cap should be removed to determine the strength. The compressive strength of the specimen is determined in capillary soaked condition and at different lateral pressures. Thus, the estimation of strength in soaked condition gives an estimate of the property of the granular material under the worst circumstances.

Permanent Deformation and Resilient Modulus Test

The resilient properties of the base materials are determined using the repeated load triaxial test. Repeated loading properties like resilient modulus and permanent deformation accumulation are major factors that influence the structural response and performance of conventional flexible pavements. These parameters are typically determined in a resilient modulus test. This test is used to determine the permanent deformation property and the resilient modulus. It is performed by placing a specimen in a triaxial cell and applying repeated axial load. After subjecting the specimen to confining pressure, measurements are taken of the recoverable axial deformation and the applied load. Both resilient (recoverable) and permanent axial deformation responses of the specimen are recorded and used to calculate the resilient modulus and the permanent deformation, respectively. Permanent deformation is the unrecovered deformation during the testing, and resilient modulus is the ratio of the peak axial repeated deviator stress to the peak recoverable axial strain of the specimen.

The test procedure followed for the present study is adapted from the standard test methods given by the VESYS user manual, NCHRP1-28A report and AASHTO T307, TP46 (FHA, 1996, Barksdale *et al*, 2003, AASHTO, 2002, FHA).

TEST SEQUENCE

The primary mechanical properties of the material were determined as per Texas manual of testing procedures. After completion of the preliminary tests, the gradation, and optimum moisture content results are used in the preparation and compaction of samples.

The preliminary tests that were conducted and the results are provided in Table 4.1.

Table 4.1 - Preliminary Tests Results

Test	TxDOT Specification	Property measured
Particle size analysis	Tex-101-E	Gradation
Determination of Liquid limit	Tex-104-E	Liquid Limit – 19
Determination of Plastic Limit	Tex-105-E	Plastic Limit – 16
Determination of Plasticity Index	Tex-106-E	Plasticity Index – 2
Laboratory Compaction Characteristics and Moisture-Density Relationship of Base Materials	Tex-113 E	Moisture Density Relationship

Sieve Analysis

Dry and wet sieve analysis was performed on Texas crushed stone material. There was a large variation in the fines content from dry to wet sieve, a wet sieve was performed on the entire material to correctly estimate the amount of fines content in the material. The dry and wet sieve analysis is provided in Table 4.2. Figure 4.1 shows the gradation of the samples with the three fines content, 5 percent, 10 percent, 17 percent and that of the proposed new specifications.

Table 4.2 - Sieve Analysis of Texas Crushed Stone along with the proposed new Specifications

Sieve Size	% Passing Dry Sieve	% Passing Wet Sieve	% Passing new fines #1	% Passing new fines #2	% Passing new fines #3	Specifications	
30	94.29	94.63	93.82	94.15	94.60		
22	78.31	71.87	67.64	69.34	71.72	65	85
15	66.74	65.74	60.59	62.66	65.57		
9	52.96	53.12	46.07	48.91	52.88	45	65
4.75	38.75	41.12	32.27	35.83	40.82	25	45
2	24.99	32.76	22.65	26.72	32.42		
0.4	12.94	24.41	13.04	17.62	24.02	5	30
0.075	5.00	17.42	5.00	10.00	17.00	0	10
pan	0.00	0.00	0.00	0.00	0.00		

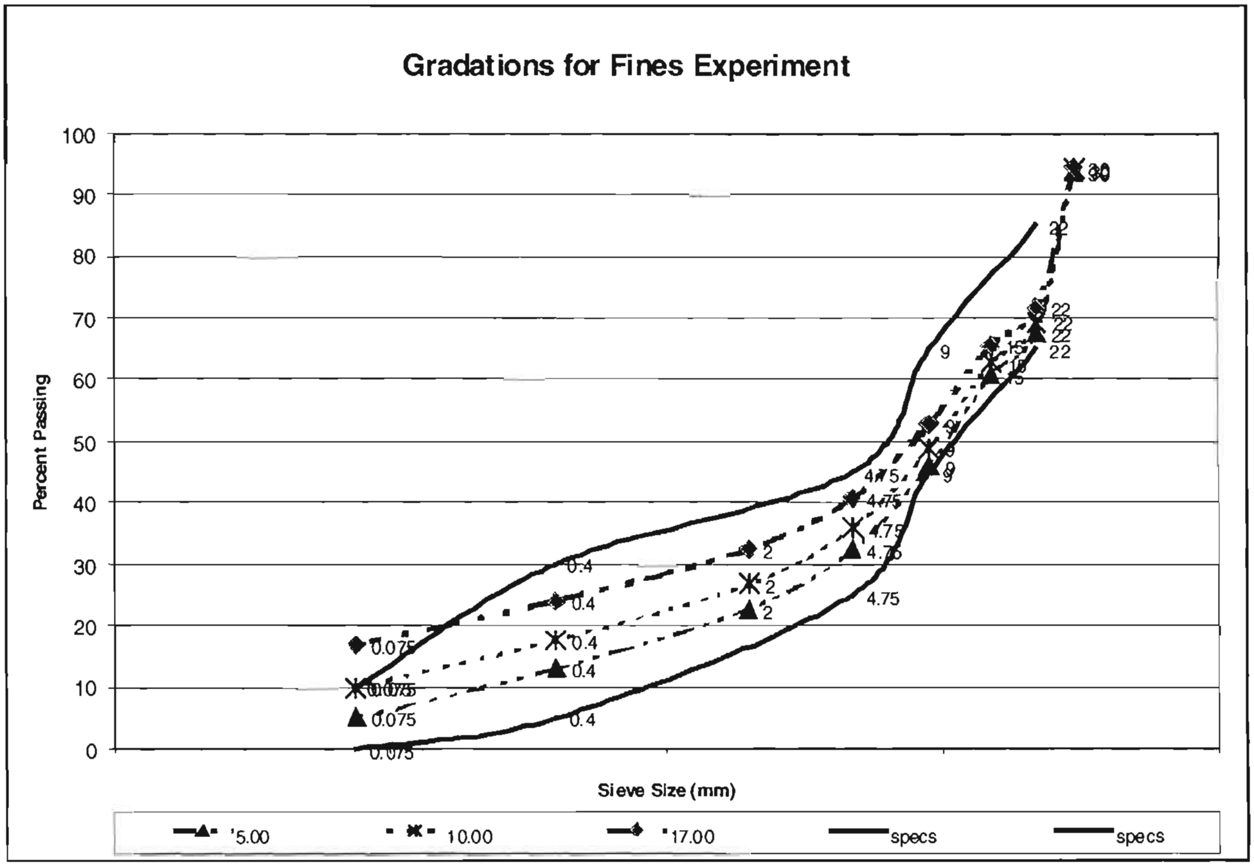


Figure 4.1 Gradation of Texas Crushed Stone with Varying Fines.

Sample Preparation

The optimum moisture content (OMC) and maximum dry density (MDD) of the material are determined from the moisture density relationships. The procedure recommended in the standard test protocol Tex-113E was used to mold the test specimens and to determine the OMC and MDD. The moisture density relationship curve is shown in Figure 4.2. Specimens are molded at the prescribed compactive effort in the respective test protocols. Figure 4.3 shows the molding and compaction equipment used in this study.

Tube Suction Test

The TST was conducted at each of the three different fines content. The optimum moisture content (OMC) and maximum dry density (MDD) of the material determined was used to mold the test specimens. Two specimens were compacted at each of the fines content to maximum dry density at optimum moisture content according to test method Tex-113 E. A 52.4 mm (6 in) diameter and 203.2 $\pm$ 6.4 mm (8 $\pm$ 0.25 in) specimens in height are wetted, mixed, molded and finished as nearly identical as possible. The surface of the specimen is made smooth after compaction. The compacted specimens were extruded from the molds and weighed. The samples after compaction and before placing in the triaxial cell are shown in Figure 4.4.

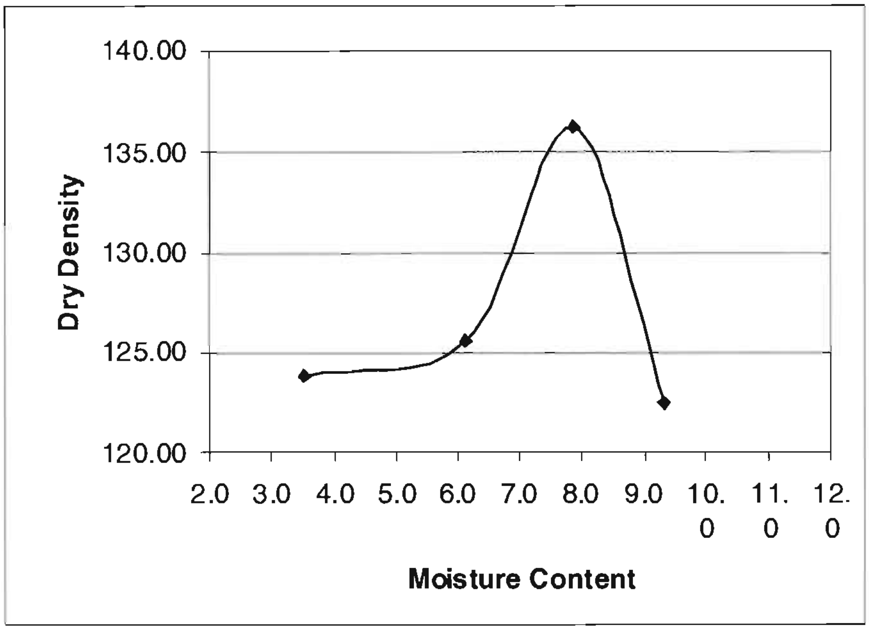


Figure 4.2 - Moisture Density Relationship

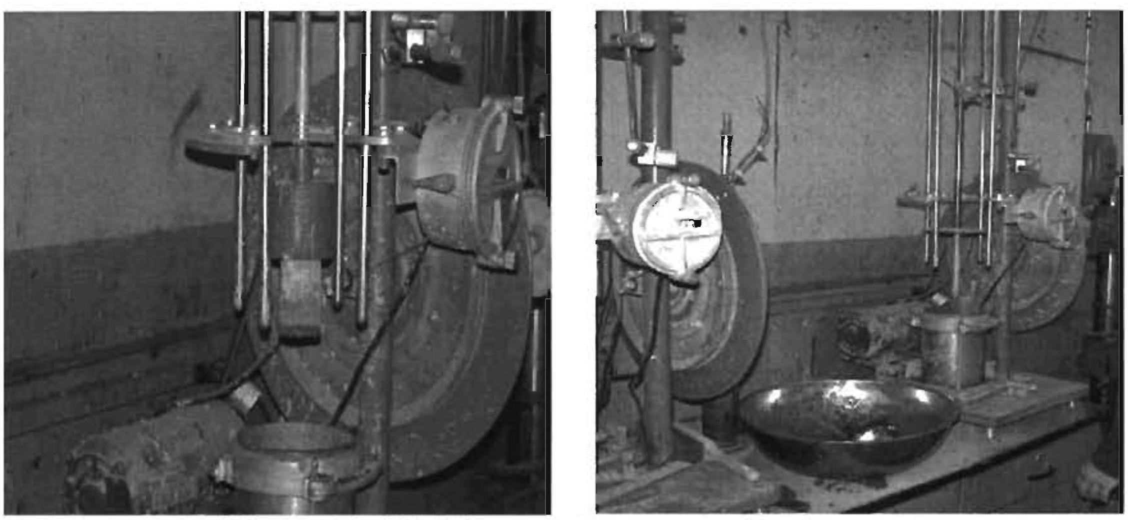


Figure 4.3 - Molding and Compaction Equipment

The specimens are then placed in an oven maintained at 60 $\pm$ 5 °C (140 $\pm$ 9 °F) for 48 $\pm$ 4 hours. The specimens are removed from the drying oven and weighed. They are then placed in a triaxial cell with a bottom base cap. Around the circumference of the base cap 1.5 mm (1/16 in) diameter holes were drilled at a horizontal spacing of 12.5 mm (1/2 in). This equates around 38 or 39 holes around the base cap. Also 1.5 mm diameter holes were drilled in each quadrant of the bottom of the base cap about 50 mm (2 in) from the center. The Adek Percometer as shown in the Figure 4.5 is used to take the six initial dielectric readings on each specimen surface. Five are equally spaced around the perimeter of the specimen, and the sixth was in the center (Guthrie, 2000, Guthrie, 2001). The probe was pressed down with a force of 9.1 $\pm$ 2.3 Kg (20 $\pm$ 5 lb.) to ensure adequate contact of the probe on the specimen surface. This was followed each time dielectric values were measured.

Figure 4.4 - Compacted Samples for TST Prior to Placing in the Triaxial Cells

Figure 4.5 Percometer

The samples were then placed in an ice chest on a level surface in a laboratory room maintained at $25 \pm 5\ ^{\circ}C$ ($77 \pm 9\ ^{\circ}F$) as shown in Figure 4.6, Figure 4.7 and Figure 4.8. The ice chest was filled with distilled water to a depth of 12.5 ± 3.2 mm ($1/2 \pm 1/8$ in). The water was maintained at this depth throughout the testing. Care was taken to avoid splashing the specimen surfaces with water during the test. The ice chest was kept closed during the ten day capillary rise, except during periods when readings are being taken. Six dielectric readings were taken on each specimen surface once a day for ten days. The sample weight was also recorded daily as W_{WET} to monitor the water content at each time interval. The bottom of the mold was wiped dry before weighing. The ice chest lid was closed after taking measurements. The test is completed when the elapsed time exceeded 240 hours. The final surface dielectric values and weights are measured and recorded. The molds are then placed in an oven at 110 $^{\circ}C$ for 24 hours. The weight of the oven dry aggregate particles was recorded as W_S.

Figure 4.6 TST at 5% Fines Content

Figure 4.7 TST at 10% Fines Content

Figure 4.8 TST at 17% Fines Content

Calculations

Gravimetric water content of each specimen just after the two day drying period,

$$WC_{dry} = \frac{100 \times (W_{dry} - W_{mold} - W_s)}{W_s} \tag{4.1}$$

where

WC_{dry} = Gravimetric water content (%),
W_{dry} = Weight of specimen and mold after two-day drying period (g or lb),
W_{mold} = Weight of mold (g or lb),
W_s = Weight of oven-dried aggregate particles (g or lb),

The gravimetric water content of the three specimens at the end of the soaking period was calculated.

Percentage of water loss for each specimen during the two day drying period,

$$P_{loss} = 10000 \times \left(\frac{((W_{omc} - W_{dry}) \div W_s)}{W_{omc}} \right) \tag{4.2}$$

where

P_{loss} = Percentage water loss (%),
W_{omc} = Weight of specimen and mold just after compaction (g or lb),
W_{dry} = Weight of specimen an dmold after two-day dryong period (g or lb),
W_s = Weight of oven-dried aggregate particles (g or lb),
WC_{omc} = Gravimetric water content just after compaction (%),

The average percentage of water loss for the three specimens is reported.

Dielectric Values

For each specimen at each time interval, the highest and the lowest dielectric readings are discarded. The average dielectric value calculated from the remaining four readings was plotted against time. The TST results for each of the six samples are shown in the Figure 4.9, Figure 4.10 and Figure 4.11. For each material the dielectric versus time and moisture-time curves are plotted for each specimen.

Texas Triaxial Test

A 6 in. by 8 in. specimen molded at optimum moisture content and maximum dry density is used in the Texas triaxial test. The specimens were under a capillary soak for 10 days during the TST. The test specimens are subjected to compression load along with their assigned constant lateral pressures of 0 and 15 psi. The base cap was removed from the test specimens. The motorized press compresses the sample at a rate of 1% strain per minute. Simultaneous readings of load and deformation at intervals of 0.5 mm (0.02 in.) deformation until specimen fails were taken. The proving ring dial is read at each 0.02 in. deformation. The readings are continued until 0.6 in. of deformation or failure occurred (*14*). The strength is determined for the different lateral pressure conditions. The test setup for 0 psi and 15 psi lateral pressure are shown in Figure 4.12, Figure 4.13 respectively.

TUBE SUCTION TEST
Data Analysis Report

SUMMARY:	Tx Crushed Stone (5%)
Batch Date:	March 8, 2003

SAMPLE TESTING

Time (hr)	0.0	28.3	50.3	73.0	97.2	140.7	170.2	213.1	237.8	259.5

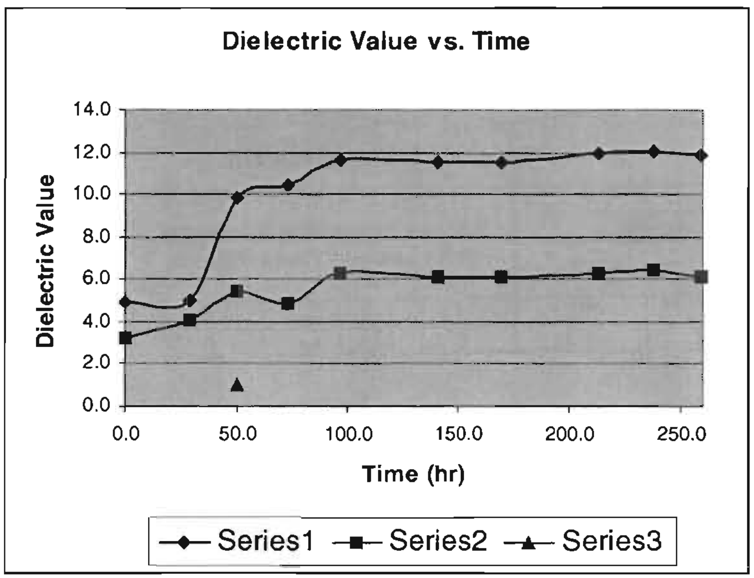
Specimen No — Average Dielectric Value

1	4.9	5.0	9.9	10.5	11.6	11.6	11.6	11.9	12.0	11.9
2	3.3	4.1	5.4	4.8	6.3	6.1	6.1	6.3	6.5	6.1

Specimen No — Average Water Content During Soaking (%)

1	0.6	5.6	6.8	7.2	7.2	7.4	7.4	7.5	7.5	7.6
2	0.5	6.2	7.2	7.7	7.5	7.5	7.5	7.7	7.6	7.7

Average Final Dielectric Value	9.0	Average Final Gravimetric Water Content (%)	7.6
Moisture Susceptibility Ranking	Good	Average Water Loss in Drying (% of OMC)	92.8



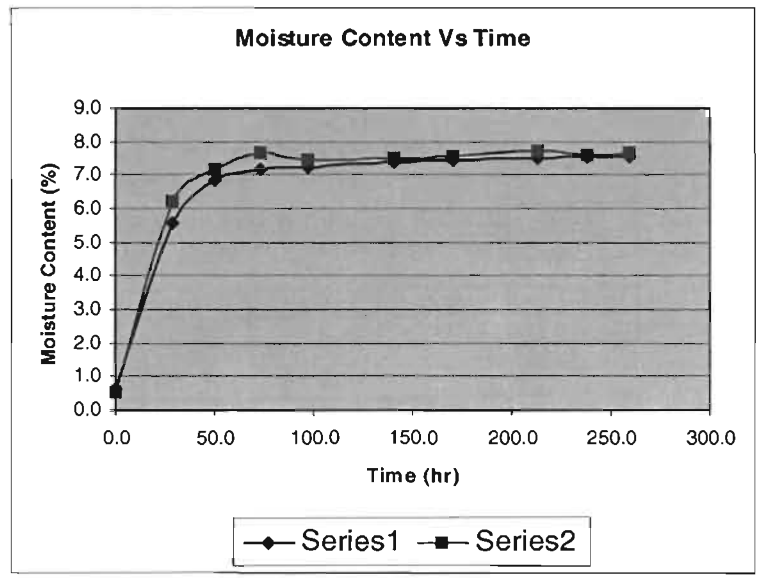


Figure 4.9 TST Results at 5% Fines Content

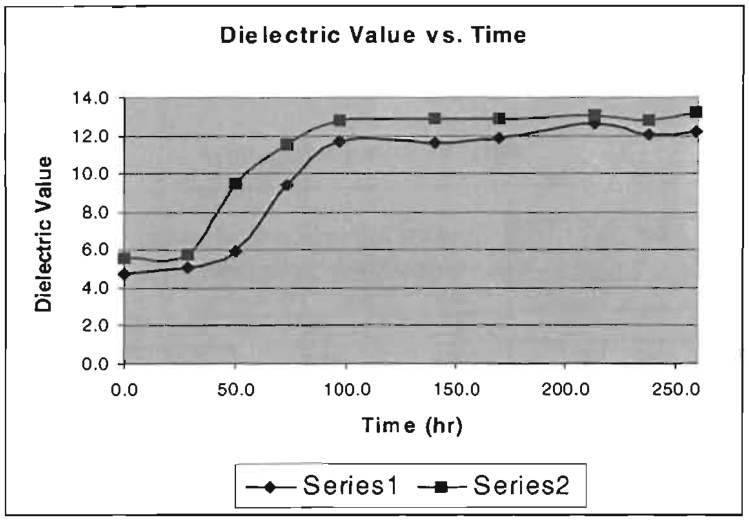
TUBE SUCTION TEST
Data Analysis Report

SUMMARY:	Tx Crushed Stone (10%)
Batch Date:	March 8, 2003

SAMPLE TESTING

Time (hr)	0.0	28.3	50.3	73.0	97.2	140.6	170.2	213.1	237.8	259.5
Specimen No	Average Dielectric Value									
1	4.8	5.1	6.0	9.4	11.8	11.7	11.9	12.7	12.0	12.2
2	5.6	5.8	9.5	11.6	12.8	12.9	12.9	13.1	12.9	13.2
Specimen No	Average Water Content During Soaking (%)									
1	1.0	4.8	5.9	6.8	7.3	7.5	7.5	7.6	7.6	7.7
2	0.9	5.5	6.6	7.2	7.3	7.5	7.5	7.5	7.6	7.6

Average Final Dielectric Value	12.7	Average Final Gravimetric Water Content (%)	7.6
Moisture Susceptibility Ranking	Good	Average Water Loss in Drying (% of OMC)	88.4



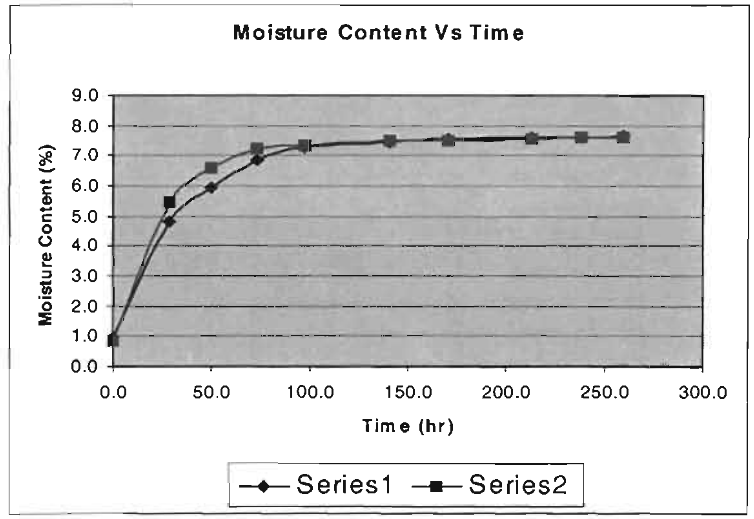


Figure 4.10 TST Results at 10% Fines Content

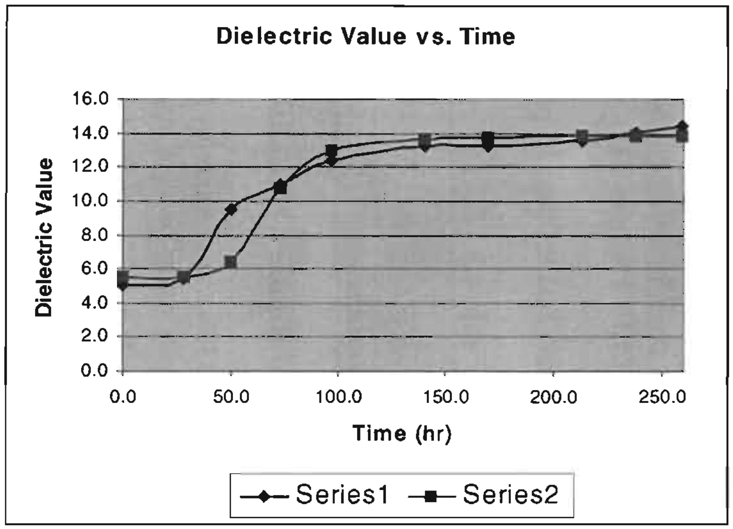
TUBE SUCTION TEST
Data Analysis Report

SUMMARY:	Tx Crushed Stone (17%)
Batch Date:	March 8, 2003

SAMPLE TESTING

Time (hr)	0.0	28.3	50.3	73.0	97.2	140.5	170.2	213.1	237.7	259.5
Specimen No	Average Dielectric Value									
1	5.1	5.4	9.5	11.0	12.4	13.3	13.3	13.6	14.1	14.4
2	5.5	5.5	6.4	10.7	13.0	13.7	13.8	13.8	13.8	13.9
Specimen No	Average Water Content During Soaking (%)									
1	1.1	5.7	7.0	7.8	8.0	8.2	8.2	8.2	8.3	8.3
2	1.2	5.5	6.5	7.3	7.7	7.9	7.9	8.0	8.0	8.0

Average Final Dielectric Value	14.2	Average Final Gravimetric Water Content (%)	8.1
Moisture Susceptibility Ranking	Marginal	Average Water Loss in Drying (% of OMC)	85.2



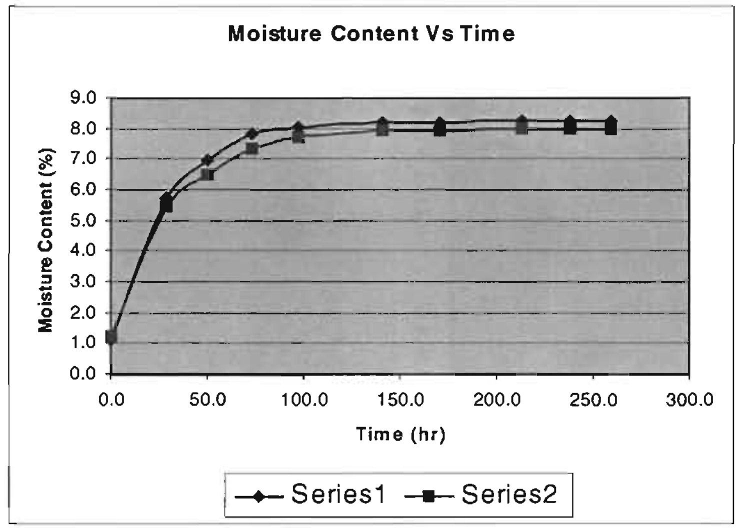


Figure 4.11 TST Results at 17% Fines Content

Figure 4.12 Triaxial Test at 0 psi Lateral Pressure

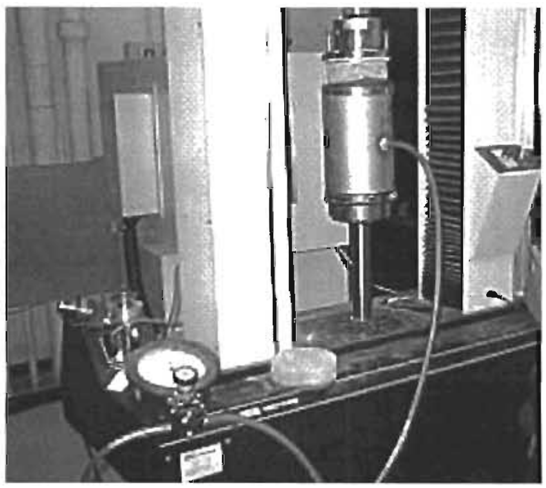


Figure 4.13 - Triaxial Test at 15 psi Lateral Pressure

The results of the Texas Triaxial Test at 0 and 15 psi lateral pressure are provided in Table 4.3.

Table 4.3 - Texas Triaxial Test Results

Sample	5% Fines	10% Fines	17% Fines
Strength @ 0 psi Lateral Pressure	29.4	25.38	21
Strength @ 15 psi Lateral Pressure	136.007	141.25	76.34

Permanent Deformation and Resilient Modulus Test

A 152 mm (6 in.) by 304 mm (12 in.) (diameter by height) specimen was prepared for all samples at the three fines content with maximum particle sizes greater than 19 mm (0.75 in.). All material greater than 25.4 mm (1 in.) was scalped off prior to testing. Test specimens were prepared to the maximum dry density (γ_d) and optimum moisture content (w). The moisture content of the sample was determined using AASHTO T265-93 (*15*). The standard method of sample preparation given in AASHTO T 307 was followed for the sample preparation. The test setup is shown in Figure 4.14. The compacted specimen was prepared for testing by placing a rubber membrane around it. The membrane was sealed to the top and bottom platens with rubber "O" rings as shown in Figure 4.15.

Figure 4.14 - Test Set-up for Resilient Modulus

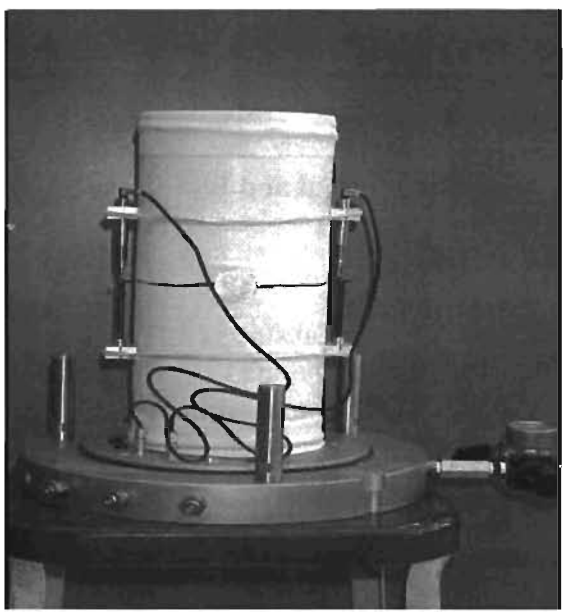


Figure 4.15 - Specimen Prepared for Testing

This test procedure consists of three stages:

1. Preliminary conditioning
2. Determination of permanent deformation properties
3. Determination of resilient modulus

Test Specimen Conditioning

The specimen was preconditioned before testing by applying 100 repetitions of a load equivalent to maximum axial stress of 41.4 kPa (6 psi) and a corresponding cyclic stress of 20.7 kPa (3 psi) using a haversine shaped 0.1 second load pulse followed by a 0.9 second rest period. A confining pressure of 103.5 kPa (15 psi) was applied to the test specimen. A schematic representation of the load and the placement of Linear Vertical Displacement Transducer's (LVDT) is shown in Figure 4.16. σ_d is the axial deviatoric stress and σ_3 is the confining pressure. LVDT's 1 and 2 measure the axial displacement and LVDT's 3 and 4 measure the radial displacement.

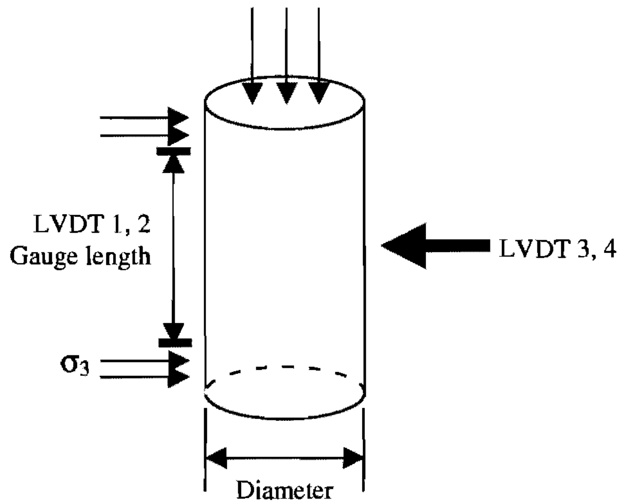


Figure 4.16 - Representation of Load and Position of LVDT's on Specimen

Permanent Deformation Test

A haversine load equivalent to a maximum axial stress of 227.7 kPa (33 psi) and a corresponding cyclic stress of 207 kPa (30 psi) with 0.1 second load pulse followed by a 0.9 second rest period was continued until 10,000 load applications or until the vertical permanent strain reaches 5% during the testing, whichever comes first. During load applications, the load applied and the axial deformation measured from two LVDTs through the data acquisition system was recorded. In order to save storage space during data acquisition, the data was recorded at specified intervals.

Resilient Modulus test

The same specimen was used to perform the resilient modulus test if the vertical permanent strain has not reached 5%. Otherwise, a new specimen was molded and the permanent deformation test was performed with the load repetitions reduced to 5,000 from 10,000. If the sample again reached 5% total permanent strain, the test was terminated. If not, the resilient modulus test was performed by initially decreasing the axial stress to 14.5 kPa (2.1 psi) and setting the confining pressure to 20.7 kPa (3 psi). The test is performed by following the sequence of loading at regular intervals shown in Table 4.4 which was recommended in NCHRP project 1-28 A (Witczak, 2004).

Table 4.4 - Permanent Deformation and Resilient Modulus Test Sequence for Granular Base and Subbase

Sequence	Confining Pressure		Contact Stress		Cyclic Stress		Maximum Stress		N_{rep}
	kPa	Psi	kPa	Psi	kPa	psi	kPa	psi	
Preconditioning									
	103.5	15.0	20.7	3.0	20.7	3.0	41.4	6.0	100
Permanent Deformation									
	103.5	15.0	20.7	3.0	207.0	30.0	227.7	33.0	10000
Resilient Modulus									
1	20.7	3.0	4.1	0.6	10.4	1.5	14.5	2.1	100
2	41.4	6.0	8.3	1.2	20.7	3.0	29.0	4.2	100
3	69.0	10.0	13.8	2.0	34.5	5.0	48.3	7.0	100
4	103.5	15.0	20.7	3.0	51.8	7.5	72.5	10.5	100
5	138.0	20.0	27.6	4.0	69.0	10.0	96.6	14.0	100
6	20.7	3.0	4.1	0.6	20.7	3.0	24.8	3.6	100
7	41.4	6.0	8.3	1.2	41.4	6.0	49.7	7.2	100
8	69.0	10.0	13.8	2.0	69	10.0	82.8	12.0	100
9	103.5	15.0	20.7	3.0	103.5	15.0	124.2	18.0	100
10	138.0	20.0	27.6	4.0	138	20.0	165.6	24.0	100
11	20.7	3.0	4.1	0.6	41.4	6.0	45.5	6.6	100
12	41.4	6.0	8.3	1.2	82.8	12.0	91.1	13.2	100
13	69.0	10.0	13.8	2.0	138	20.0	151.8	22.0	100
14	103.5	15.0	20.7	3.0	207	30.0	227.7	33.0	100
15	138.0	20.0	27.6	4.0	276	40.0	303.6	44.0	100
16	20.7	3.0	4.1	0.6	62.1	9.0	66.2	9.6	100
17	41.4	6.0	8.3	1.2	124.4	18.0	132.5	19.2	100
18	69.0	10.0	13.8	2.0	207	30.0	220.8	32.0	100
19	103.5	15.0	20.7	3.0	310.5	45.0	331.2	48.0	100
20	138.0	20.0	27.6	4.0	414.0	60.0	441.6	64.0	100
21	20.7	3.0	4.1	0.6	103.5	15.0	107.6	15.6	100
22	41.4	6.0	8.3	1.2	207	30.0	215.3	31.2	100
23	69.0	10.0	13.8	2.0	345.0	50.0	358.8	52.0	100
24	103.5	15.0	20.7	3.0	517.5	75.0	538.2	78.0	100
25	138.0	20.0	27.6	4.0	690.0	100.0	717.6	104.0	100
26	20.7	3.0	4.1	0.6	144.9	21.0	149.0	21.6	100
27	41.4	6.0	8.3	1.2	289.8	42.0	298.1	43.2	100
28	69.0	10.0	13.8	2.0	483.0	70.0	496.8	72.0	100
29	103.5	15.0	20.7	3.0	724.5	105.0	745.2	108.0	100
30	138.0	20.0	27.6	4.0	966.0	140.0	993.6	144.0	100

The test was stopped when the total permanent strain of the sample exceeds 5% and the result was reported. After the completion of the test, the confining pressure was reduced to zero and the specimen was removed from triaxial chamber. The moisture content of the specimen was determined at the end of the test using AASHTO T265-93. The testing sequence is shown schematically in Figure 4.17.

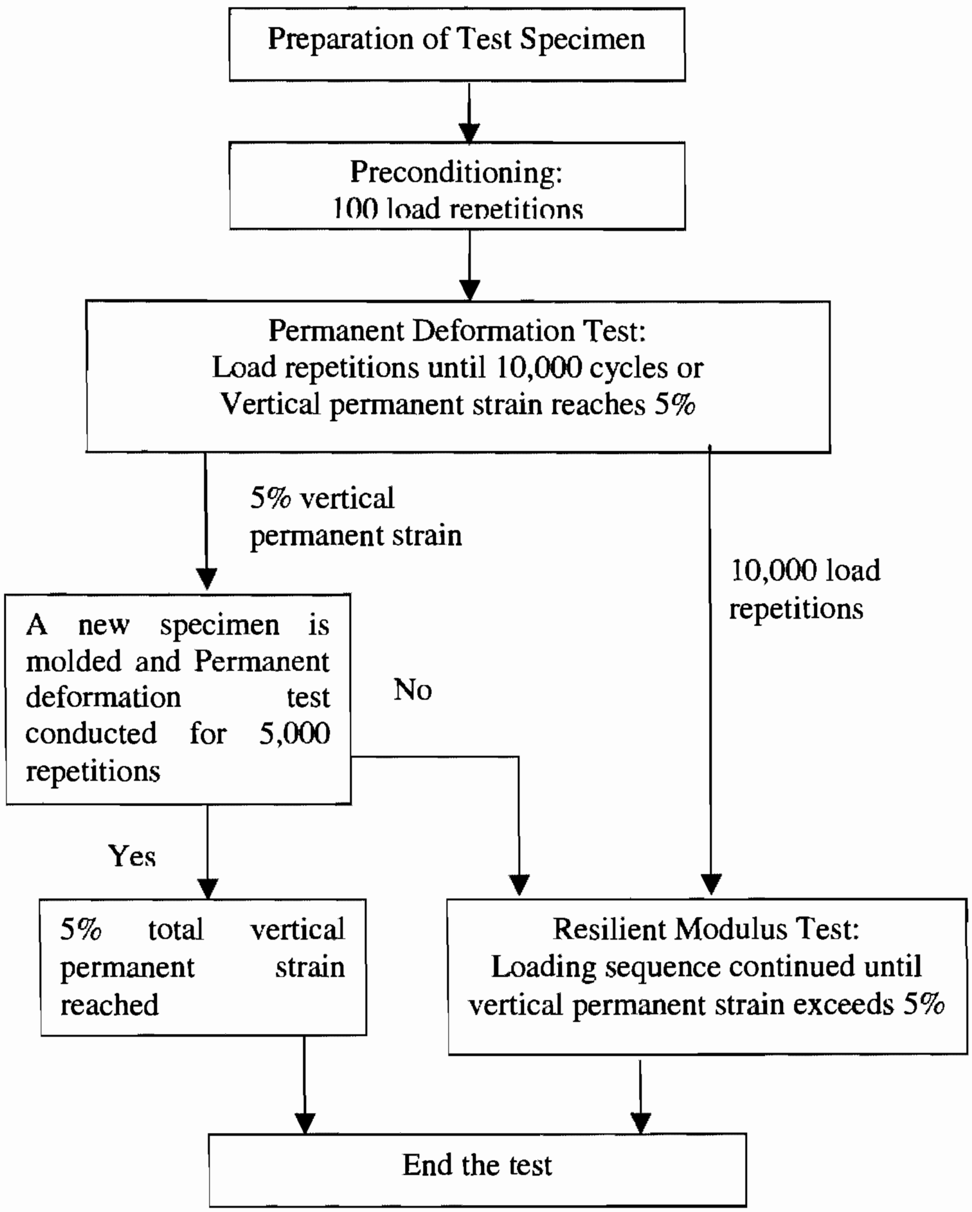


Figure 4.17 - Flowchart of the Test Procedure for Permanent Deformation and Resilient Modulus

Calculations

The following results are computed from the test:

- Cumulative axial permanent strain and resilient strain at 200^{th} load repetition
- The permanent deformation parameters intercept (a) and slope (b) from the plot of cumulative axial permanent strain versus the number of load cycles
- Rutting parameters α and μ using the following equation:

$$\alpha = 1 - b \tag{4.3}$$

$$\mu = \frac{a \times b}{\varepsilon_r} \tag{4.4}$$

The resilient modulus is calculated by the following equation which is being adapted in NCHRP 1-37A project (2002 design guide):

$$M_r = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \tag{4.5}$$

$$\log M_r = \log(k_1 \times P_a) + k_2 \log\frac{\theta}{P_a} + k_3 \log\left(\frac{\tau_{oct}}{P_a} + 1\right) \tag{4.6}$$

Where:

$k_1, k_2 \geq 0$
$k_3, k_6 \leq 0$
$k_7 \geq 1$

M_R = resilient modulus
τ_{oct} = octahedral shear stress
= $\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$
θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$
$\sigma_1, \sigma_2, \sigma_3$ = principal stresses
k_i = regression constants
P_a = atmospheric pressure

The permanent deformation and resilient modulus test results are shown in figure 4.18 and 4.19. The permanent deformation properties were determined at a confining pressure of 7 psi and a deviator stress of 28 psi. The resilient modulus values are determined at 5 psi confining pressure and a deviator stress of 15 psi.

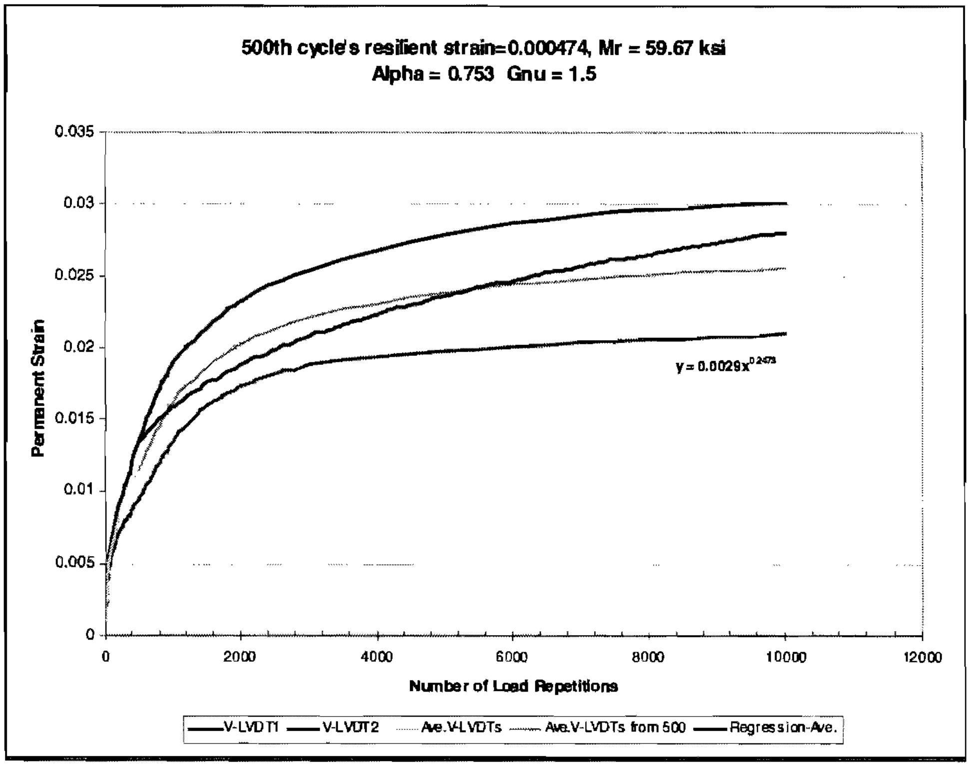


Figure 4.18 - Permanent Deformation Test Result

Regression Equation: Mr=K1*Pa*(THETA/Pa)^K2*(TAUoct/Pa+1)^K3

K1=3546.59

K2=.96

K3=-.98

R^2=.98

Recommended Resilient Vertical Modulus at Confining Pressure=5 psi and Deviator Stress =15 psi

Mr-v=70.45 ksi

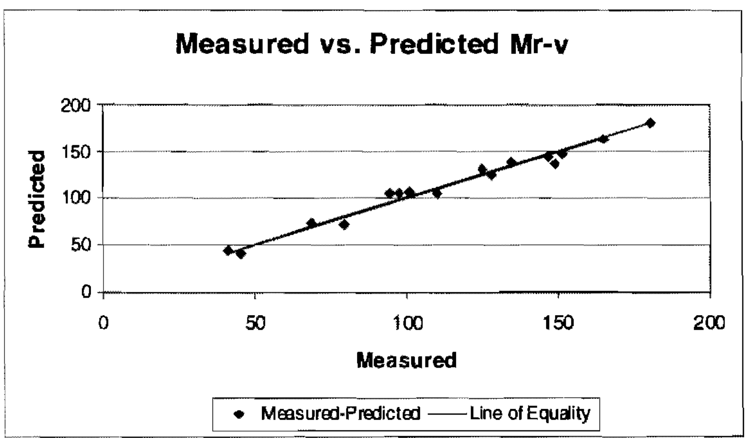


Figure 4.19 - Resilient Modulus Test Result

TEST RESULTS

The TST was conducted on the Texas crushed stone samples with fines content of 5 percent, 10 percent and 17 percent. The final average dielectric values obtained were 9, 12.7 and 14.2 for 5 percent, 10 percent and 17 percent fines content respectively. Based on these final dielectric values, these samples are ranked as excellent, good and marginal. Table 4.5 shows a summary of the TST results for each of the fines content. The final dielectric value of the specimens for the three samples is plotted against time.

Table 4.5 - Results of Tube Suction Test

Sample	5% Fines	10% Fines	17% Fines
Actual Compaction Moisture	8	8	8
Actual Dry Density	121.05	131.55	132.65
Relative Density	89.65	97.4	98.25
Average Water Content at End of TST	7.6	7.6	8.1
Loss of Water During Drying	92.8	88.4	85.2
Average Dielectric Value	9	12.7	14.2
Moisture Susceptibility Ranking	Excellent	Good	Marginal

For all of the samples the dielectric value increased with time and then stabilized at the end. The final dielectric value is used for the classification of the materials. From Figure 4.20, it is evident that as the fines content increases the dielectric value of the sample increases which implies that the moisture susceptibility of the material increased and further resulting in reduction in stability. It was found that the samples with 5% fines, 10% and 17% fines ranked in the order of excellent to poor material resistance to moisture susceptibility respectively.

The strength was measured at 0.6 in extension at 0 psi and 15 psi lateral pressure. The higher the lateral pressure the higher the axial strength. It is clear from Figure 4.21 that the axial strength decreased as the fines content increased. However, the axial strength at a lateral pressure of 15 psi

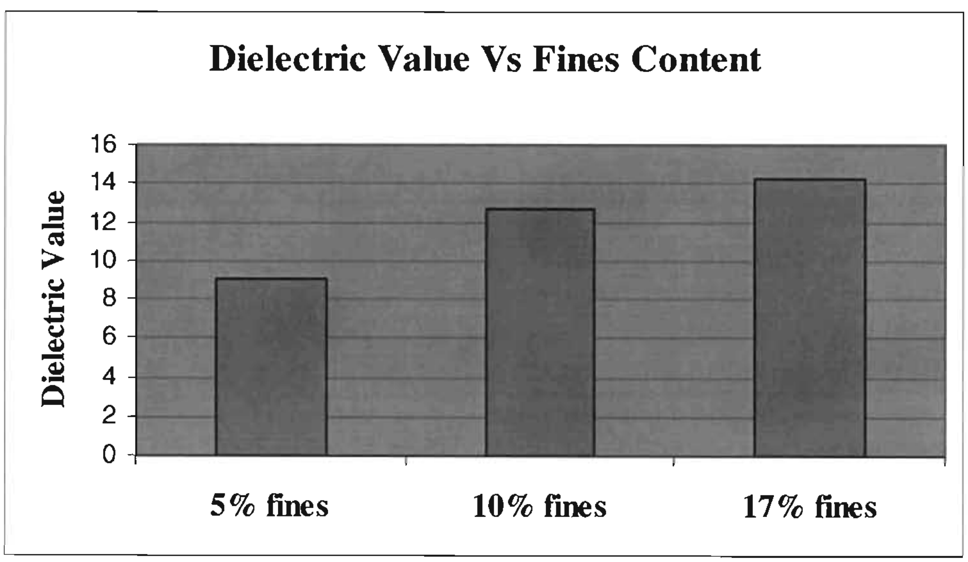


Figure 4.20 - Plot of Dielectric Value with Varying Fines Content

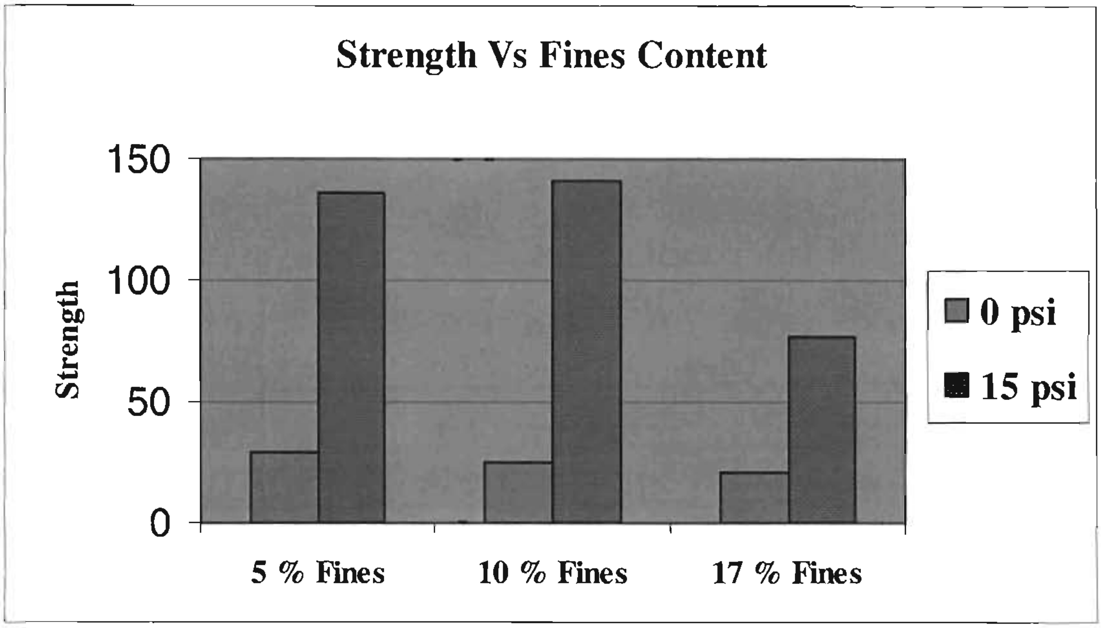


Figure 4.21 - Plot of Strength at Varying Fines Content

was almost the same for 5 and 10 percent fines content and decreased remarkably at 17 percent fines content. This may be due to the high fines content which may reorient the particles easily under load and compact the sample to attain a higher density and hence result in higher strength.

The results of the Permanent Deformation and Resilient Modulus test are shown in Appendix D. The regression equations obtained for both the permanent deformation and resilient modulus models used to determine these properties are given for each of the samples. Also, the permanent deformation parameters alpha and gnu obtained are shown in Table 4.6.

Table 4.6 - Results of Permanent Deformation and Resilient Modulus Test

Specimen Fines %	Resilient Strain	A	μ	Resilient Modulus	k_1	k_2	k_3
10	0.000895	.923	1.361	57.66	2748.68	1.06	-1.03
10	0.000474	0.753	1.5	70.45	3546.59	0.96	-0.98
5	0.000316	0.907	0.316	98.84	5315.09	0.82	-0.88
5	0.000923	0.894	0.709	54.42	2398.99	1.10	-0.90
17	0.000344	0.844	1.122	50.04	1989.5	1.15	-0.73
17	0.00054	0.874	0.572	55.2	2324.87	1.38	-1.29
17	0.000571	0.819	0.916	55.57	2477.64	1.08	-.088
17	0.000408	0.924	1.146	62.42	2786.93	1.11	-0.94
17	0.000351	0.733	1.14	61.08	2941.81	0.81	-0.59

The resilient modulus values were calculated at 5 psi confining pressure and 15 psi deviatoric stress. A plot of average value of resilient modulus for the various fines content is shown in Figure 4.22. It indicates that the resilient modulus value decreased as the fines content increased from 5 to 17 percent.

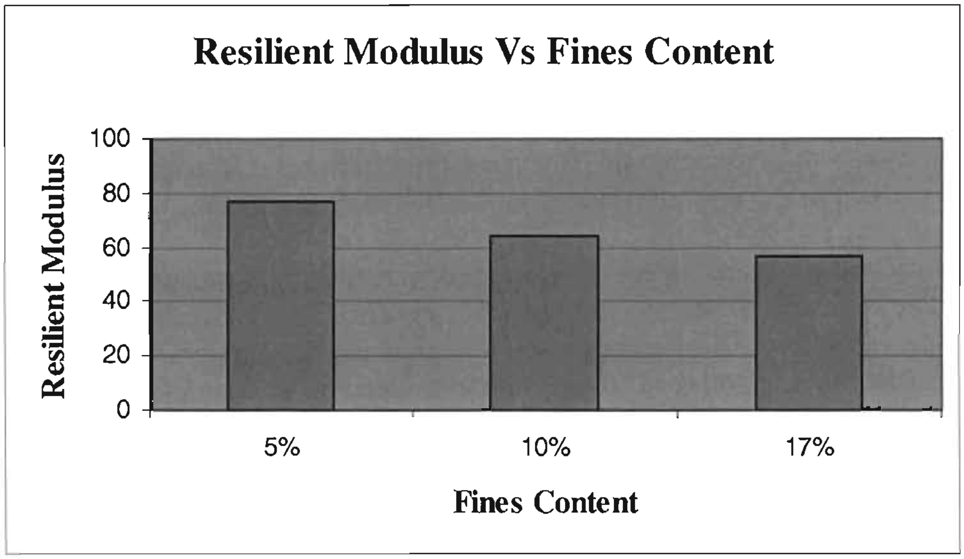


Figure 4.22 - Plot of Resilient Modulus with Varying Fines Content

Figure 4.23 shows the resilient deformation at 500 load cycles. This shows that the sample with 10 percent fines content had better tendency to recover the deformation it underwent under the load. While, the samples with 5 and 17 percent fines had lesser resilient (recoverable) deformation. This indicates that the permanent deformation property of 10 percent fines content is better than 5 percent or 17 percent. This may be due to the dense graded particle arrangement achieved by providing the adequate fines and hence after compaction and initial deformation, there is no room left for further deformation. Here, it should be noted that Figure 4.22 is the result of the resilient modulus test. While Figure 4.23 is the test result of the permanent deformation test.

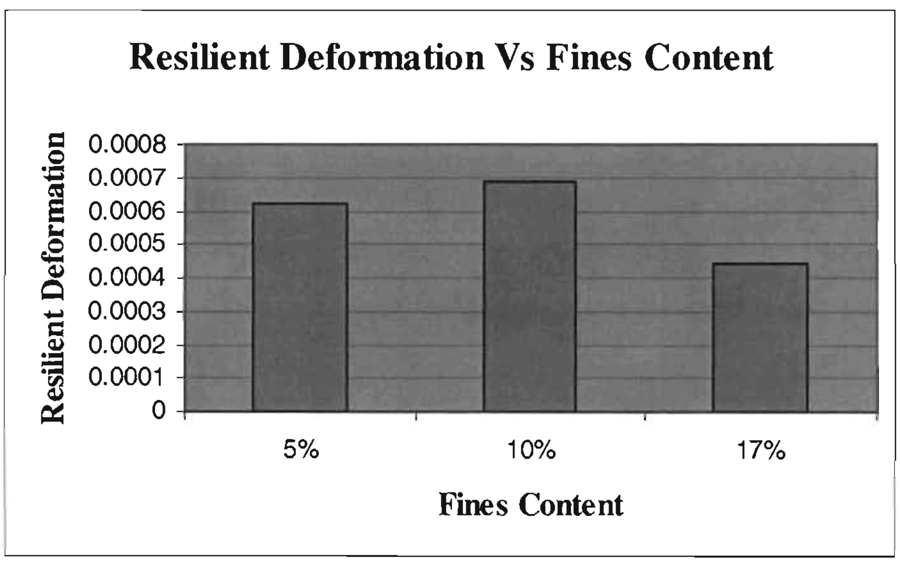


Figure 4.23 - Plot of Resilient Deformation at Varying Fines Content

Figure 4.24 shows the Plot of permanent deformation with number of load cycles. The figure indicates that the permanent deformation for 10 percent fines is more than that of the 5 percent and the 17 percent fines. This may be due to the rearrangement of the particles in the specimen with 10 percent fines during repeated loading. For a specimen with 5 percent fines, the permanent deformation increased gradually and then stabilized to an asymptote at around 4,000 load cycles. Similar is the case with a specimen of 10 percent fines. However, the specimen with 17 percent fines content reaches an asymptote level at around 1,000 load repetitions.

These results contradict our earlier conclusion based on resilient deformation after 500 load cycles. It should be noted here that these values of resilient and permanent deformation are the actual values measured in the laboratory. A better approach to comparing these values is the rate of permanent deformation. Also, it is appropriate to take into consideration the particle distribution within the specimen. The specimen with high fines content would have the coarse aggregate particles floating in the matrix while the specimen with lower fines content would have the smaller particles filling the voids of the coarse aggregate. Hence, the method of compaction and the compactive effort play a huge role in the particle distribution within the aggregate structure which affects the permanent deformation properties of the specimen. A specimen uniformly compacted would better represent permanent deformation under repeated load cycles.

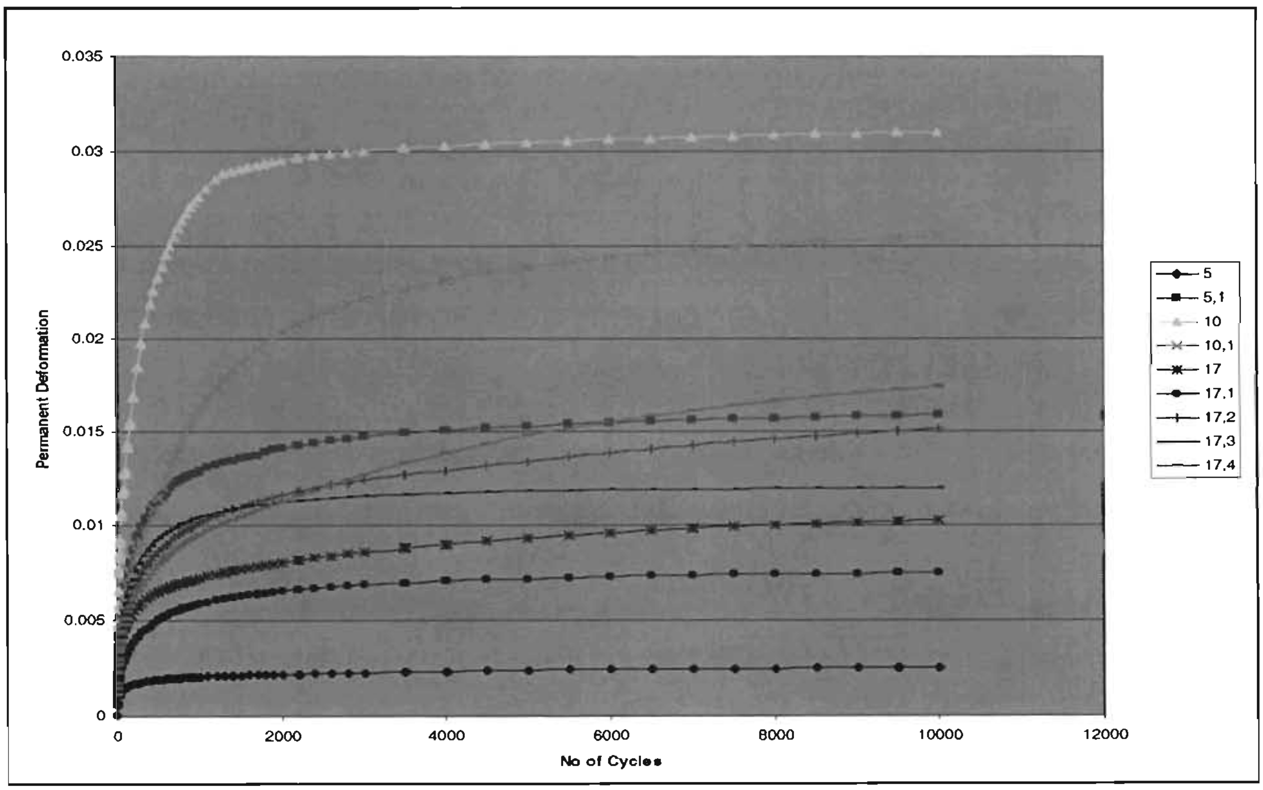


Figure 4.24 - Plot of Permanent Deformation vs. Number of Load Cycles at Varying Fines Content

CONCLUSIONS AND RECOMMENDATIONS

The TST indicates that the lower the fines content, the higher is the moisture susceptibility of the base. The dielectric value was lowest when the fines content was 5 percent. Further, at 15 psi lateral pressure the strength at 17 percent fines is reduced to around half of the strength at 5 and 10 percent fines content. Also, the resilient modulus and permanent deformation properties are higher for 5 percent fines content.

It is evident from the results that the lower the fines content better is the quality of the aggregate in terms of engineering properties. However, it is proposed that a limit of 10 percent could provide a good consensus between the quality requirements and the problem of fines disposal faced by the aggregate industry. This limit was chosen as there is no big difference in some properties at 5 and 10 percent fines content. It can be suitably concluded that the fines content should be limited to less than 10 percent and a lower limit can be set where conditions are favorable to moisture damage.

This research shows the tremendous impact the fines content has on properties of aggregate base layer. As discussed earlier since the method of compaction and the compactive effort have an impact on the permanent deformation properties, future research should concentrate on assessing the impact of compactive effort and method of compaction on particle distribution which affects the characterization of resilient modulus and permanent deformation properties.

CHAPTER FIVE

SUMMARY AND CONCLUSIONS

One of the underlying factors affecting the performance and life of flexible pavements is the base material that supports it. Several factors can be said to affect the performance of the base material and in turn the pavement. Among these are the material selection process, the density of the in-place base material, moisture content, exposure to the atmosphere prior to sealing, and thickness of layers used to place the base material. A manner in which to analyze base material in the laboratory and correlate the data to actual field performance of the base is needed. This is especially true when the traffic loading is taken into consideration.

Among the items that have been studied is the gradation of the base material used in pavement system design. More specifically the amount of fines found in the gradation has been found to have links to different types of pavement distress. The type of fines – high plasticity, low-plasticity, and no plasticity – has been found to affect the base material as well. The purpose of this project was to determine how fines content, gradation, and types of fines affected the engineering properties of base materials in the laboratory.

In Texas, the current specifications allow for various bases used in a district to be classified as Class 1; which under TxDOT specification is the highest classification attainable. However, more important is the possibility for there to exist large variances in the properties of these materials. Currently a set of testing procedures does not exist that provides a clear relationship between field performance and lab performance.

Although density is often used to determine the quality of a base layer, it cannot and should not be used as the deciding factor in predicting the performance of the base in the field. Other factors that need to be addressed are moisture retention, stiffness and moisture susceptibility.

By comparing base material specifications in Texas with other states it was found that Texas was the only state that did not control the amount of fines (-200 material) permitted in its base material and the plasticity of the fines permitted for use in base material was higher than most other state DOT's. Most states set an upper limit of 10% fines, but states in hard freeze areas typically drop the limit to 5%. These points are of concern due to the fact that the fines content has been linked to several of the distresses experienced by flexible pavements, namely alligator cracks, rutting and frost heave.

Past studies also have found that the gradation and fines content can affect the performance of base materials in the areas of shear strength, stiffness, and moisture susceptibility. Previous research had determined that increase in fines content increased the densities of base materials, however, as the percentage of fines increases the bearing ratio, permanent deformation and resilient modulus values decreased. The presence of plastic fines was also found to have negative effects on the field performance of base materials. Increased fines content was also found to affect the moisture susceptibility of base materials through the use of specimen dielectric constants.

Under research performed in Europe the amount and type of fines affected the internal angle of friction. The addition of fines was found to increase the angle of internal friction, however, plastic fines were found to decrease the internal angle of friction. It was also found under this study that although many European countries experience extreme climatic conditions, frost sensitivity and moisture susceptibility were not emphasized. Many countries agreed that the percentage of fines found in a base material needed to be controlled. However the type of fines and amount of fines found in base materials throughout Europe varied.

From previous research it could be said that any increase in fines content would increase density and permanent deformation. On the other hand an increase in the fines content would also have a negative effect on the strength, modulus, and moisture susceptibility properties of the base material. Various tests also demonstrated the existence of an upper limit for the fines content; where the properties improved as the fines were increased but reach a point where any increase in the fines resulted in a negative impact on properties.

With this background, the Center for Transportation Infrastructure Systems and the Texas Transportation Institute followed a similar approach in determining the effects of varying fines contents and fines types on base material mixtures. Under the TTI approach, studies were conducted on a locally available high fines base which typically contains 17% fines. Tests were conducted on this material at 5, 10 and 17% fines. . The CTIS approach altered the gradation while increasing the fines content such that the gradation was optimized for the given fines content. The CTIS approach also altered the base material mixtures by creating mixtures that contained both low plasticity and clay fines.

Both universities followed a similar testing pattern although various tests were performed or modified according to the capabilities and needs of each university. The tests that were used to analyze the engineering properties were sieve analysis, Atterberg limits, moisture-density relationships (Proctor values), triaxial strength, moisture susceptibility, permanent deformation, and resilient modulus.

The last four tests were performed differently at each university. The triaxial tests at UTEP were performed using test methods Tex-143-E and Tex-117-E. TTI also used test method Tex-117-E but not Tex-143-E. Moisture susceptibility was also studied under entirely separate methods.

TTI used the Tube Suction Test while UTEP followed the procedure developed under Project 1735 using the Free-Free Resonant Column Test. Permanent deformation tests differed in the loading used during the tests. Finally, resilient modulus tests differed in two manners. First, the manner in which the confining pressure was applied differed significantly. Second the loading sequence also differed. Further details on these differences can be found in Chapter 3.

The findings of the UTEP portion of this study showed that the addition of fines did have an effect on the properties of the base material. This is first observed in the moisture-density relationship where the increase in fines resulted in an increase in density with a decrease at a fines content of 20%. This increase can be attributed to the presence of fewer air voids since the voids are "filled in" with fines. However, as the fines content increases, the fines begin to take more space in the material and the larger aggregates are forced apart filling the specimens in with more fines.

Moisture susceptibility was also affected by the increase in fines. Previous studies had found that the addition of fines to the base layer would affect the layers susceptibility to moisture; and countries in Europe limited or completely avoided the use of fines in their base layers. The results from this study showed similar trends. Under the method used the mixtures that were more susceptible to moisture were those that had a higher modulus ratio. The highest ratio obtained was from the mixtures containing 20% fines. The lowest ratios were found to exist among the 5% low-plasticity fines, 10% clay fines, and 0% fines mixtures. The increase in the moisture susceptibility could be caused by the fact that there exist smaller pore spaces between aggregates creating more capillary action. This in turn results in an increase in the amount of moisture found in the base material.

The Texas triaxial tests provided results that were similar for both methods. Under test method Tex-143-E, the addition of fines to the base mixture improved the classification of the materials. For the materials with low-plasticity fines, the classifications achieved are typically better than those with high-plasticity fines. For the mixtures containing clay, the classification gradually improved reaching a Class 1 material with 10% clay. The same held true using TEX-117-E, however, in the low-plasticity fines the mixture containing 5% fines did not reach the Class 1 classification. The change in classification using clay may be due to the increased fines content that may have acted as a lubricant and attributed to the reorientation of soil particles as the load was applied to the specimen.

Previous work concerning the effects of the fines content on the resilient modulus of base materials had indicated that increases in the fines content resulted in higher permanent deformation and lower resilient modulus values. From the results obtained it was found that low percentages of fines had no effect on the resilient modulus values. The resilient modulus values decreases with increasing fines. Although the specimens containing 20% fines saw increases in the resilient modulus these values were still below the maximum value obtained for the 0% and 5% fines contents.

Increases in the fines content seemed to follow the findings of previous work on permanent deformation. The specimens containing 0%, 5% and 10% clay fines exhibited increases in the permanent deformation with the increase in fines. The specimens containing low-plasticity fines exhibited the same trends. However the specimens with 5% fines exhibited the most deformation. The specimens with 20% fines content in both cases deformed the least.

These findings seem to be in line with the results obtained by TTI. As the percentage of fines increased from 5% to 10% to 17% the asymptotic dielectric in the tube suction test increased from 9 to 12.7 to 14.2. The moisture content in the 17% fines sample increased to just above the optimum moisture content at the end of the 10 days capillary rise. It is anticipated that the high fines material may perform poorly in the field if exposed to water. The strength at 15 psi confining pressure reduced from over 135 psi at 5 to 10% fines to 76 psi at 17% fines. A similar reduction was also noted in the resilient modulus of the high fines base. The TTI findings agreed with the CTIS findings in that a limit exists for the percentage of fines that may be included in the base material mixture. TTI recommended for their test material the optimal fines content appeared to be between 5 and 10%.

From the results it can be said that the lower the fines content in a base material the better it will perform. However, the findings also saw that there exists a limit in the amount of fines that should be permitted in base mixtures. From the findings a 10% fines content seems to be the limit at which the fines content should be cut off. However the large variance between the 10% and 20% fines may provide the existence of a still higher cutoff percentage. It can be suitably concluded that the fines content should be limited to less than 10 percent and a lower limit can be set where conditions are favorable to moisture damage.

Through this project it was found that the impact that the fines content has on properties of base materials is not to be taken lightly. However, as discussed previously there exist many factors that affect the performance of base layers. Not only must laboratory tests exist that predict the performance of base materials in the field adequately; testing methods for in-place material must also be developed. In the end both tests must be looked at and a correlation developed that will properly provide an understanding of the behavior of the material in the field.

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