Effects of Pulverization on Properties of Stabilized Bases

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16. Abstract

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Phase I of this study consisted of an extensive laboratory study to determine the impact of changes in gradation on the desired stabilizer content of a base material. The impact of pulverization was also studied on an ongoing project. The results are provided in this report. It was found that the change in gradation indeed impacts the properties of the mix and should be considered in the design stages of FDR. In Phase II, the ways to address this matter will be investigated and reported.

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Abstract

Pulverization of pavement base materials is routinely carried out for rehabilitation of roads through full-depth reclamation (FDR). The primary stabilizers currently used in TxDOT districts for FDR are cement, lime, and fly ash. The optimum stabilizer content is currently determined either based on experience or through a series of laboratory tests that evaluates the strength, stiffness and durability of the base-stabilizer mix. For lab testing, base materials are retrieved from the site way before pulverization. The change in gradation due to pulverization can significantly impact the base strength and stiffness.

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Implementation Statement

At this stage of the project, the implementation of the results is not recommended. Recommendations for implementation will be made in the future reports.

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Chapter 1

Introduction

Rehabilitation of highway pavements through FDR is an option chosen by more and more state transportation agencies. This option decreases cost by reducing the use of virgin aggregate for base material. While FDR is a viable alternative, pulverization of the asphalt layer with base material or base material alone may change the strength of the base layer due to the formation of fine materials during the crushing action of the pulverizer. Typically, a stabilizer is used in the FDR process which aids in strength gain for the base layer. The stabilizers mostly used by TxDOT are cement, lime and fly ash. The optimum stabilizer content is currently determined either based on experience or through a series of laboratory tests that evaluates the strength, stiffness and durability of the base-stabilizer mix. For lab testing, base materials are retrieved from the site way before pulverization. The change in gradation due to pulverization can significantly impact the base strength and stiffness. This matter is addressed in this report.

Objective

The main objective of this research project is to evaluate the effects of pulverization on the base properties and to determine the optimum stabilizer content necessary to obtain a reasonably strong, stiff and durable base layer that will perform well for a long time.

The first task of the project was to perform an information search relevant to pulverization of pavements, utilization of the selected stabilizers, test procedures to determine base strength before and after pulverization, and nondestructive testing (NDT) methods to monitor the stabilized pavement sections. The second task required the selection of four sites ready for construction to observe the construction method and to monitor the strength and performance of the FDR projects under realistic conditions. The third task was to establish test protocol to characterize the change in properties of stabilized bases due to change in gradation after pulverization. For this task, a limestone base often used in El Paso was utilized for testing. The impacts of change in gradation, as well as stabilizer type and content, moisture-density curve, modulus, unconfined compressive strength, moisture susceptibility and structural design were studied. Task 4 involved evaluating the materials collected from four test sites prior to and during construction and performing the tests described in Task 3. The results from one of the sites in Odessa, TX are presented in this report.

Organization of Report

Chapter 2 contains a review of relevant literature regarding issues on stabilization, aggregates, chemical stabilizers, construction practices, density, curing, monitoring of sections, and impact of pulverization on structural integrity.

Chapter 3 outlines the testing protocol for characterization of stabilized base material. The material used for this portion of the study is a limestone base from El Paso. The topics discussed in this chapter are development of gradation curves, selection of stabilizer, test procedures, retained strength, modulus, retained modulus, moisture susceptibility and optimum stabilizer content.

Chapter 4 presents information and results from a case study for the site in Odessa. The topics discussed in this chapter are the description of the site, construction activity, testing activity, laboratory testing, determination of optimum cement content, and field structural evaluation.

Chapter 5 presents the preliminary conclusions drawn and the directions for the Phase II of the study.

Chapter 2

Background

Introduction

Deterioration of roads is a continuous problem across the United States. Improving these roads becomes tremendously expensive when the road is completely rebuilt using virgin materials. Many States use full-depth reclamation (FDR) and soil stabilization with diverse additives to rehabilitate their roads more economically (Mallick et al., 2002). In FDR process, the existing base is pulverized and mixed in-place. The concern with the pulverization is the crushing of coarse aggregates of the base; and, as a result, change in the gradation. Changes in gradation, may adversely affect the strength and stiffness of the final product. The pulverized materials that do not meet specifications may be stabilized with additives (such as cement, lime or fly ash) to improve their workability during construction and to improve their strength to withstand expected loading from traffic.

The topic of discussion in this chapter is the literature review regarding pavement structure, soil stabilization, stabilizers, and coarse aggregate issues as well as construction processes.

Stabilization

Stabilization is achieved by adding proper percentage of additives such as cement, lime, fly ash, bitumen, or combinations of these materials to the base. The selection of the type and determination of the percentage of additive are dependent upon the soil classification and the desired degree of improvement. Generally, smaller amounts of additives are required to modify soil properties such as gradation, workability and plasticity. Larger quantities of additives are used to significantly improve the strength, stiffness and durability (Army TM 5-822-14, 1994). Spreading and compaction are achieved by conventional means after the additive has been mixed with the base. The most common improvements achieved through stabilization include:

- Reducing plasticity index
- Reducing swelling potential
- Increasing durability and strength
- Reducing dust during construction

- Waterproofing the soil
- Drying of wet soils
- Conserving aggregate materials
- Reducing cost of construction
- Providing a temporary wearing surface

Stabilization may provide a working platform for construction operations, especially in wet regions. These types of improvements are referred to as soil modification. In addition, the improvement in strength and stiffness of a soil layer may permit a reduction in design thickness of the stabilized layer as compared with an unbound layer.

The selection of stabilizer type depends on the type of material present and their location in the pavement structure (Terrel et al., 1979). Table 2.1 provides varying stabilization methods for different materials. Coarse and fine grained soils, as well as clays are suitable for stabilization with portland cement and lime-fly ash and lime. Typically, several criteria must be followed for the selection of a stabilizer. Figure 2.1 demonstrates a basic flowchart used by TxDOT for the selection of additive used for base treatment. Aside from the physical properties of the soil, TxDOT also considers the goals of the treatment, mechanisms of additives, desired engineering and material properties, design life, environmental conditions and economical factors.

Table 2.2 presents TxDOT construction specifications for gradation after pulverization and after base stabilization with cement, lime or fly ash. Once the base and stabilizer are mixed, the material must pass the 1.75 in. sieve by 100% and the 0.75 in. sieve by 85%. Table 2.3 demonstrates the general criteria the U.S. Army utilizes to choose a stabilizing additive. The first criterion for selection of stabilizer is based on USCS (Unified Soil Classification System). Different stabilizer types are then recommended with restrictions on liquid limits (LL) and plastic indices (PI).

Table 2.1 - Stabilization Methods for Different Soil Types (Terrel et al., 1979)

Soil Types	Most Effective Stabilization Methods
Coarse granular soil	Mechanical blending, soil-asphalt, soil-cement, lime-fly ash
Fine granular soil	Mechanical blending, Portland cement stabilization, lime-fly ash, soil-asphalt, chlorides
Clays of low plasticity	Compaction, Portland cement stabilization, chemical waterproofers, lime modification
Clays of high plasticity	Lime stabilization

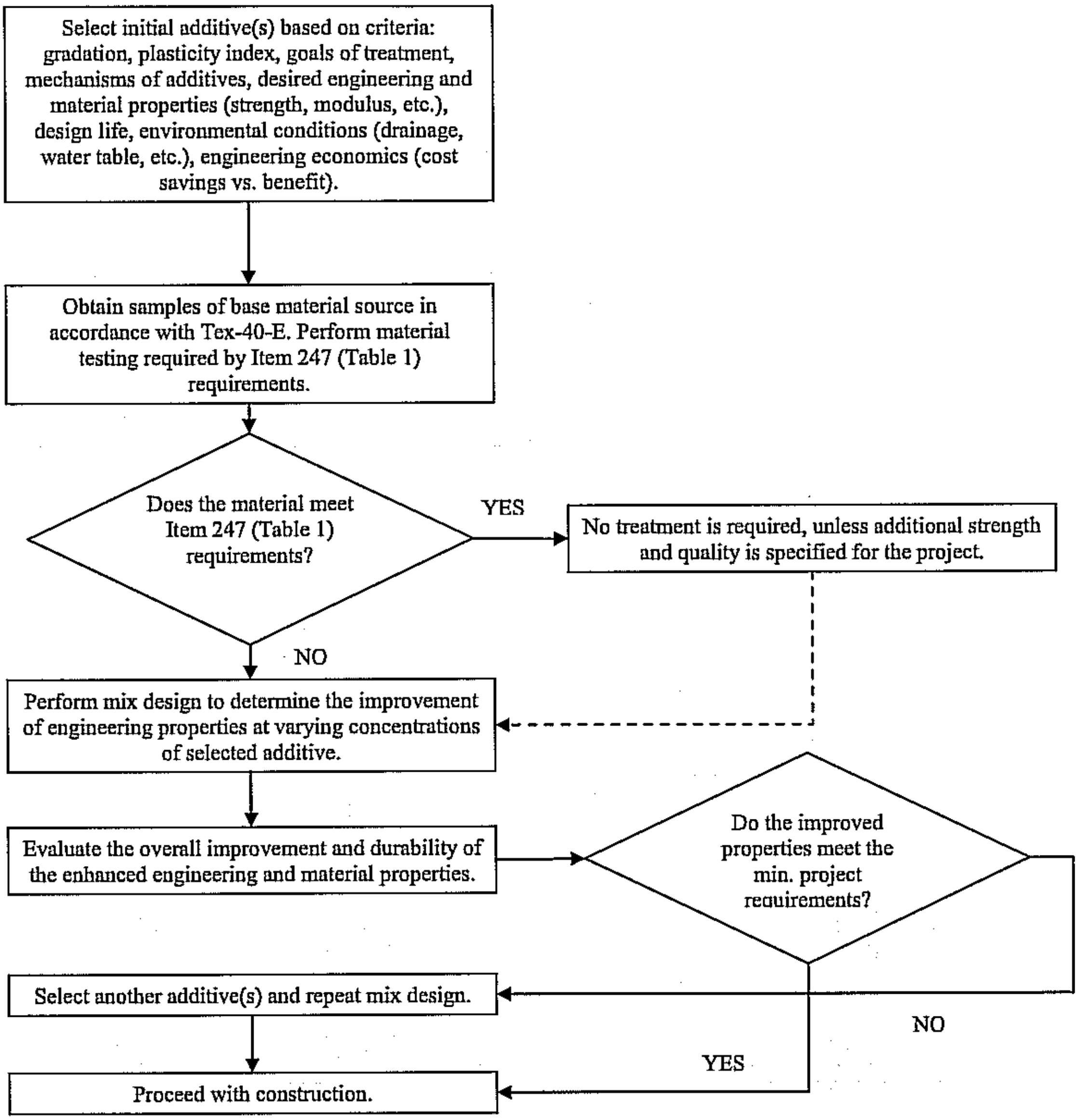


Figure 2.1 - TxDOT Flowchart for Base Treatment (TxDOT, 2005)

Table 2.2 - Construction Specifications for Stabilized Base (TxDOT, 2004)

Sieve size	Minimum % passing
1¾ in.	100%
¾ in.	85%
No. 4	-

Table 2.3 - Guide for Selecting a Stabilizing Additive (ARMY TM 5-822-14, 1994)

Soil Class.[a]	Type of Stabilizing Additive Recommended	Restriction on LL and PI of Soil	Restriction on Percent Passing No. 200 Sieve[a]	Remarks
SW or SP	Bituminous Portland cement Lime-cement-fly ash	 PI not to exceed 25		
SW-SM or SP-SM or SW-SC or SP-SC	Bituminous Portland cement Lime Lime-cement-fly ash	PI not to exceed 10 PI not to exceed 30 PI not to exceed 12 PI not to exceed 25		
SM or SC or SM-SC	Bituminous Portland cement Lime Lime-cement-fly ash	PI not to exceed 10 ---[b] PI not to exceed 12 PI not to exceed 25	Not to exceed 30% by weight	
GW or GP	Bituminous Portland cement Lime-cement-fly ash	 PI not to exceed 25		Well graded material only Material should contain at least 45% by weight of material Passing No. 4 sieve
GW-GM or GP-GM or GW-GC or GP-GC	Bituminous Portland cement Lime Lime-cement-fly ash	PI not to exceed 10 PI not to exceed 30 PI not to exceed 12 PI not to exceed 25		Well graded material only Material should contain at least 45% by weight of material Passing No. 4 sieve
GM or GC Or GM-GC	Bituminous Portland cement Lime Lime-cement-fly ash	PI not to exceed 10 ---[b] PI not to exceed 12 PI not to exceed 25	Not to exceed 30% by weight	Well graded material only Material should contain at least 45% by weight of material Passing No. 4 sieve
CH or CL Or MH or ML or OH Or OL or ML-CL	Portland cement Lime	LL less than 40 and PI less than 20 PI not to exceed 12		Organic and strongly acid soils falling within this area are not susceptible to stabilization by ordinary means

[a]Soil classification corresponds to MIL-STD-619B. Restriction on liquid (LL) and plasticity index (PI) is in accordance with Method 103 in MIL-STD-621A.
[b]$PI \leq 20 + [(50 - \text{percent passing No. 200 sieve})/4]$

Stabilization Additives

Stabilization additives are widely recognized for their strengthening ability in pavement base construction. A large variety of industry by-products and commercially produced additives is available for use in pavement stabilization, such as:

- Air-cooled blast furnace slag
- By-product lime
- Fly ash
- Ground granulated blast furnace slag
- Reclaimed asphalt pavement
- Recycled concrete material
- Portland Cement

For the purposes of this study, portland cement, lime and fly ash will be discussed. The major properties of these three calcium-based additives are given in Table 2.4.

Table 2.4 - Summary of Conventional Granular Stabilizers (Yoder et al., 1975)

Stabilizer	Portland Cement	Lime	Fly Ash
Mechanics of Stabilization	Principally hydration. Some modification of clay materials	Change water film, flocculation, and chemical	Some modification of clay materials
Suitable Soil	Most soils, except organic soils, highly plastic clays, and poorly reacting sandy soils	Highly effective for highly plastic soils (PI≥12)	Plastic clay soils
Maximum Dry Density	Varies	Decreases	Increases, however delay compaction time decreases density
Optimum Moisture	Varies	Increases	Decrease
Plastic Index	Decrease	Decrease	Decrease
Plasticity	Decrease	Decrease	Decrease
Strength	Increase	Moderate increment	Increases, however curing temperature and delay time affects strength

Portland cement

Portland cement is the product of two basic raw ingredients: a calcareous material and an argillaceous material. The calcareous material is a calcium oxide such as limestone, chalk, or oyster shells. The argillaceous material is a combination of silica and alumina obtained from clay, shale, and blast furnace slag (Mamlouk et al., 1999). These materials are then crushed, passed through a grinding mill and processed in a kiln. The raw materials are melted at 2500°F to 3000°F and converted to cement clinker. After a cooling period, gypsum is added and both materials are pulverized into a fine powder.

Since Portland cement is composed of several compounds, many reactions occur concurrently during hydration. The hydration process occurs through two mechanisms: through-solution and topochemical. The through-solution process governs the early stages of hydration and consists of:

- Dissolution of anhydrous compounds into components
- Formation of hydrates in solution
- Precipitation of hydrates from the supersaturated solution

Additionally, the topochemical hydration is a solid-state chemical reaction that occurs at the surface of the cement particles.

Cement is used to improve strength and stiffness of granular base and subbase materials, to reduce their plasticity or swell characteristics, to prevent consolidation, and to produce a firm-working platform as a subbase (Portland Cement Association, 2003). With the rapid depletion of acceptable granular materials for use as bases and subbases, it becomes very important to conserve the remaining limited supply of acceptable materials. Marginal granular materials are cement-modified to improve their bearing values and reduce their plasticity to meet specifications for acceptable base and subbase materials. The resulting product, however, is still primarily a granular base material with all the characteristics of that type of construction.

Specifications for pavement base and subbase course materials place limits on the amount and plasticity of the fines in granular materials. Excessive fines can lead to loss of stability, susceptibility to frost action, and mud-pumping under traffic loads. The most common and simple measurement of the improvement of a granular material containing an excessive amount of clay is to determine its plasticity index (PI). This index is a significant indicator of soil behavior; the higher the PI is, the more plastic and less suitable for use in construction the soil will be. Typically, specifications for base courses limit the PI to about 6 along with maximum fines content (No. 200 sieve) of 10% to 12%. For subbase courses, more fines are permitted with maximum PI's of 6 to 10.

An example of the effect of cement on reducing the PI of a clayey gravel is shown in Figure 2.2. For this substandard material, a cement content of about 3 or 4% by weight would reduce the PI sufficiently to meet the specifications. The figure also shows a continued reduction in the PI as measured in the field over a 10-year period.

A commonly used method to evaluate the quality of soils (in terms of bearing capacity) is the California Bearing Ratio (CBR) test. The high quality base materials will have CBR's in the range of 70 to 90 while suitable subbase materials will have lower values down to about 20. Flexible pavement design procedures of some agencies specify a minimum CBR for each layer; for example, 80 for the base course, 30 for the second layer (subbase), and 15 for a third layer (select material). Figure 2.3 demonstrates the CBR values for clayey gravel compared to its cement content for 21-day curing time.

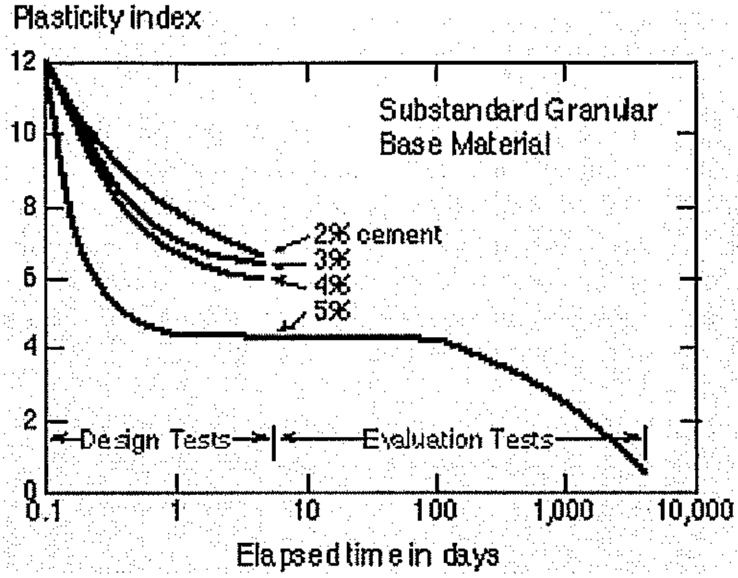


Figure 2.2 - Reductions in Plasticity Index due to Cement Content and Time (PCA, 2003)

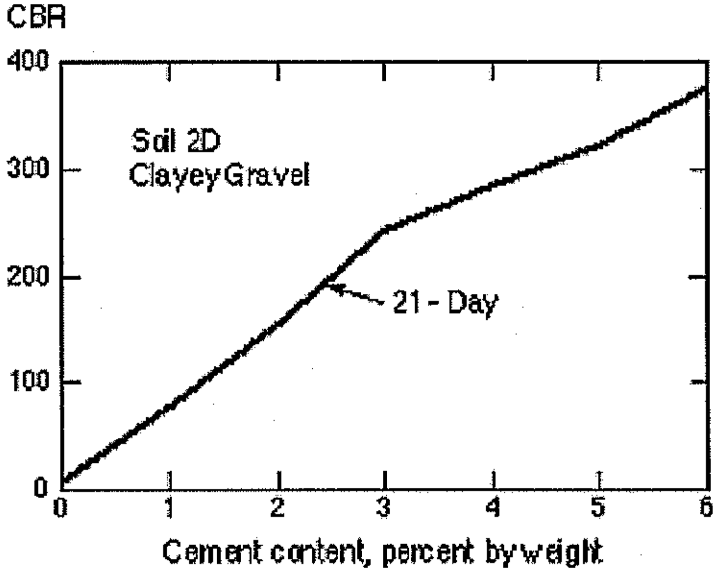


Figure 2.3 - California Bearing Ratio versus Cement Content (PCA, 2003)

Lime

The production of lime occurs as limestone (calcium carbonate) is heated in a kiln where carbon dioxide is removed and lime (calcium oxide) is formed. The kiln's exhaust gases are filtered using electrostatic precipitators, bag-houses, or other such methods. The filtered material is collected and sold as by-product lime. Lime Kiln Dust (LKD), which can vary chemically depending on the type of lime being manufactured, can be categorized according to its reactivity, which is based on the amount of free lime and magnesia content and this corresponds to the lime types: calcitic (chemical lime, quicklime, etc.) or dolomitic (Little, 2000).

By-product lime is a very fine, white powdery material of uniform size containing calcium and magnesium carbonates as its principle mineral constituents. Much of LKD's properties are determined during plant production based on the feedstock, kiln design, fuel type, and type of dust control/collection method.

By-product lime, which is used as a modifying and stabilizing agent in soil treatment, generally increases the workability of clayey soils by reducing the plasticity index and increasing the optimum moisture content. On the other hand, high levels of free lime content in LKD have been shown to result in poorer shrinkage or expansion. By-product lime provides a stable, working platform for paving operations during construction. Also, this material aids in the reduction of high moisture borrow soils in embankment construction. Adding lime to unstable subgrade soils provides one of the least expensive remedial actions.

A study performed by Little (2000) for the Lime Association on three soils stabilized with lime indicated that the soil strength significantly increased as indicated in the results from the unconfined compressive strength and the resilient modulus tests as shown in Table 2.5.

Table 2.5 - Summary of Unconfined Compressive Strength and Resilient Modulus (Little, 2000)

Soil Description	Plasticity Index[1]		Unconfined Compressive Strength[2], psi		Resilient Modulus[2] at Deviatoric Stress of 6 psi, ksi	
	Without lime	With lime	Without lime	With lime	Without lime	With lime
Moderately plastic silty clay (L=with 5% hydrated lime[3])	24	4	21	401	11	40
Moderately plastic tan clay (L=with 5.5% hydrated lime[3])	29	9	41	423	8	91
Heavy clay (L=with 6% hydrated lime[3])	38	10	23	330	5	30

These soils contained 25% or more of fines passing the No. 200 sieve. Unconfined compressive strengths and resilient moduli were determined following a capillary soak. The capillary soak was designed to simulate the critical moisture state of the layer in the pavement system. Based on the results presented in Table 2.5, the soils were sufficiently reactive with lime to produce increased unconfined compressive strengths when comparing the results of lime treated specimens to those with no lime specimens. The accelerated curing with available moisture for the lime specimens appears to have also influenced the resilient modulus by increasing the value of unstablized versus stabilized specimens by at least four times.

Fly Ash

Fly ash is a by-product produced in large quantities during the day-to-day operations of coal-fired power plants. In general, the coal source is pulverized and blown into a burning chamber where it ignites to heat boiler tubes. The heavier particles of ash (bottom ash or slag) fall to the bottom of the burning chamber, while the lighter particles (fly ash) remain suspended in the flue gases. Before leaving the stack, these fly ash particles are removed by electrostatic precipitators, bag-houses, or other dust collectors/air pollution control devices. Fly ash is divided into two classes: Class F and Class C depending on the type of coal source. Class F fly ash is produced by burning anthracite or bituminous coal; whereas, Class C fly ash is produced from lignite or sub-bituminous coal. Fly ash is a fine, powdery silt-sized amorphous residue. The varying amounts of carbon affect the color of fly ash. Gray to black fly ash indicates an increased percentage of carbon. While a tan fly ash is indicative of lime and/or calcium content. Fly ash may exhibit pozzolanic properties and, in certain types, cementitious properties.

When combined with Portland cement concrete (PCC), Class F fly ash has pozzolanic properties, whereas Class C fly ash is naturally cementitious due to its high amount of calcium oxide. Fly ash can be added to PCC to modify pH, change the hydration process (fly ash retards hydration thus lowering heat of hydration), reduce water demand, and reduce permeability. Dry fly ash can be used as an inert fill material or supplementary cementitious material to improve cohesion and stability of bituminous concrete binder and soil embankments. Fly ash is also used as a fine aggregate or supplementary cementitious material in PCC. However, some states limit the use of Class F to no more than 15% by weight, and Class C to no more than 20% by weight. In combination with sand, fly ash may be a supplement or substitute for cement to make a flowable fill, or as grout for concrete pavement sub-sealing. The use of fly ash as a supplementary cementitious material aids in the reduction of landfill space, and reduced emissions and fuel consumption required for cement production.

Singh (2001) performed CBR tests and compressibility tests to study the suitability of fly ash as base and subbase. The CBR tests were conducted on fly ash samples compacted with compactive energy varying from 89 ft-lb to 797 ft-lb and moisture content ranging from 0% to 45%. These tests were conducted on pond ash and hopper fly ash. Pond ash is fly ash residue that has been formed as slurry and deposited in a holding pond. Hopper fly ash is ash residue collected by an electrostatic precipitator or similar device. The test results are given in Tables 2.6 and 2.7. At constant moisture content, a rapid increase in CBR values is observed with compactive energy. However, at a constant compactive energy, the optimum moisture content results in maximum CBR value. Yet, the moisture content required for maximum CBR value decreases as the compactive energy increases. The decrease in CBR value at higher moisture content indicates that if the compacted fly ash bases can be isolated from ground water they can serve as a good road base material.

Table 2.6 - Results of CBR Test on Pond Ash (Singh et al., 2001)

Compactive Energy, ft-lb	Moisture Content, %					
	0	5	15	25	35	45
89	0.9	6.5	11.1	8.3	7.9	3.7
266	1.7	7.3	21.2	31.5	28.5	4.6
443	9.2	26.3	28.1	42.9	23.3	12.9
620	10.0	33.2	29.2	59.5	26.3	15.0
797	34.9	39.1	34.3	65.7	29.0	16.4

Table 2.7 - Results of CBR Test on Hopper Fly Ash (Singh et al., 2001)

Compactive Energy, ft-lb	Moisture Content, %					
	0	5	15	25	35	45
89	1.5	5.2	13.9	8.6	3.2	2.0
266	5.2	11.4	24.9	21.3	5.7	3.2
443	11.5	23.9	42.5	32.3	7.1	5.1
620	15.9	28.5	49.0	50.0	8.5	6.3
797	17.5	40.3	69.2	54.5	9.9	7.8

Aggregate Related Issues

A major concern during the construction of base layers is the degradation of aggregates due to handling, transportation and placement. An aggregate base must meet its purpose as the main structural layer of a pavement by performing the following three functions: subgrade protection, support for surfacing, and as a construction platform. In order to protect the subgrade, the base layer must be able to distribute loads sufficiently so that the subgrade can carry repeated traffic loads without significant deformation. Usually, rutting rapidly advances due to the stress on the soil exceeding a significant limit or approaching failure. The function of the base consequently satisfies the stress level on the subgrade soil to a level that the soil can withstand without significant deformation.

While the base layer protects the layer below, it also has to provide adequate support for the surface layer. If the base fails to provide this support then upper pavement layers will be forced to perform a structural role for which they are not designed. Therefore, the pavement will experience accelerated failure mechanisms such as wearing course slippage, map cracking and surface pot-holing (Dawson, 2003).

Finally, the aggregate base must be able to withstand heavy machinery during construction. In order to apply the pavement surfacing, the base must be level and stable. If the base is not constructed properly then application of the surface layer will be problematic because sufficient compaction may not be achieved.

The secondary functions of a base layer are drainage and subgrade protection against frost. Base performance is conditional on its resistance to moisture effects on its strength. A base that has failed due to moisture intrusion is reflected in the surface pavement as fatigue cracking or rutting. Therefore, a well draining base is vital to maintain the strength within the base as well as other pavement layers.

Subgrade protection against frost is accomplished by base aggregate that is not excessive in fines which acts as a shielding layer. Because the lack of fines permits the movement of water to drain more easily from the aggregate base, the aggregate layer is less likely to experience heave. Unfortunately, there is a large portion of the U.S., which experiences soil freezing during the winter; and, an aggregate base layer cannot protect the soil from freezing which leads to differential heave. Severe cases of differential heave will usually reduce traffic speeds significantly and may cause damage to vehicles or loss of control of the vehicle. In Figure 2.4, the differential heave has created surface irregularities in the form of waves on the surface of the pavement.

Figure 2.4 - Pavement Damage Due to Frost Heave (www.tfhrc.gov)

Aggregate is a durable material being tolerant to mishandling. Insufficient compaction or segregation will not cause immediate failure of a pavement base but a decrease in performance may well result. In addition, aggregate properties are dependent on geologic and moisture characteristics, as well as, particle shape. At the macroscopic level, aggregate performs by being stiff, resistant to permanent deformation and having a balanced value of permeability (Dawson, 2003). The principle mechanism by which loads are distributed to the aggregate layer by stresses produced by vehicle tires is resilient modulus. Resilient modulus is the strength or stiffness of the subgrade layers resistant to severe deformation. The distribution or spreading of loads through the layers is the modular ratio. Should the modular ratio of the base be greater than that of the underlying layers, then the load spreading through the layer is satisfactory. Conversely, high stiffness is equivalent to a high stress gradient in the base that requires the base aggregate to be resistant to deformation.

In addition, for weak aggregate, resistance to permanent deformation may be demonstrated as visible and irrecoverable damage in a single loading application. As a substitute, the additional increase of the small irrecoverable deformations, which occur under each cycle of loading, is the area of concern (Dawson, 2003). For most aggregates, the initial rapid rate of development of deformation will quickly slow; but, for aggregate bases, deformation will continue and perhaps even accelerate as shown in Figure 2.5

At the microscopic level, the particle roughness and texture each affect the mechanical performance of the aggregate. According to Dawson (2003), the resilient modulus is largely dependent on stone surface friction and the resistance to the accumulation of permanent deformation is dependent on the roughness of individual particles. In a graded mix, the breakage of whole particles is not likely due to the support provided by the surrounding particles. During roller compaction, some breakage may occur at the surface but these are minor changes as shown in Figure 2.6. From Figure 2.6, it is evident that after 32 passes of compaction, all of the aggregates retained on the various sieve sizes did not break considerably. Therefore, the gradation does not change significantly.

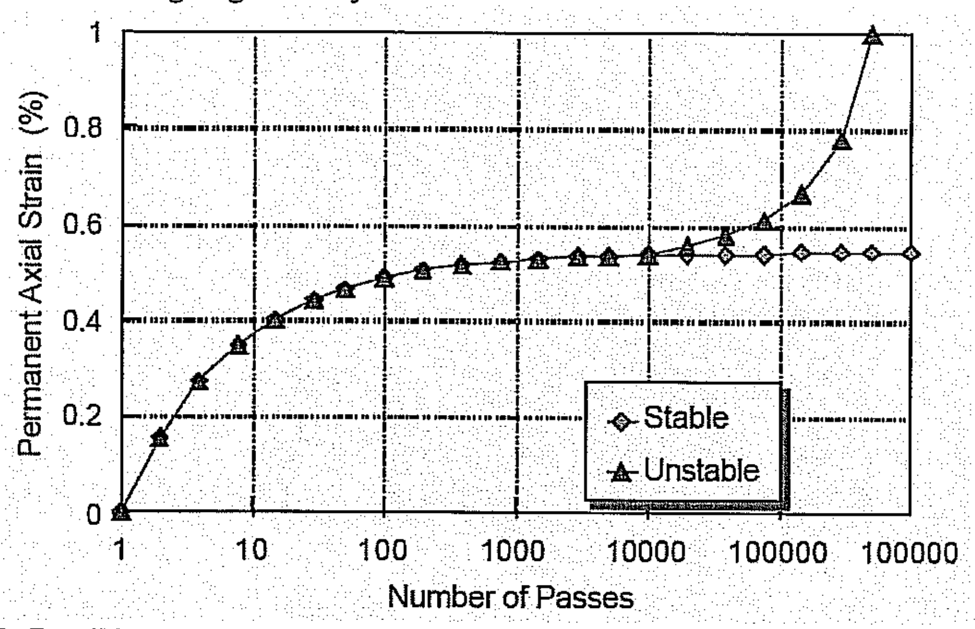


Figure 2.5 - Possible Permanent Deformation Behavior of an Aggregate Base (Dawson, 2003)

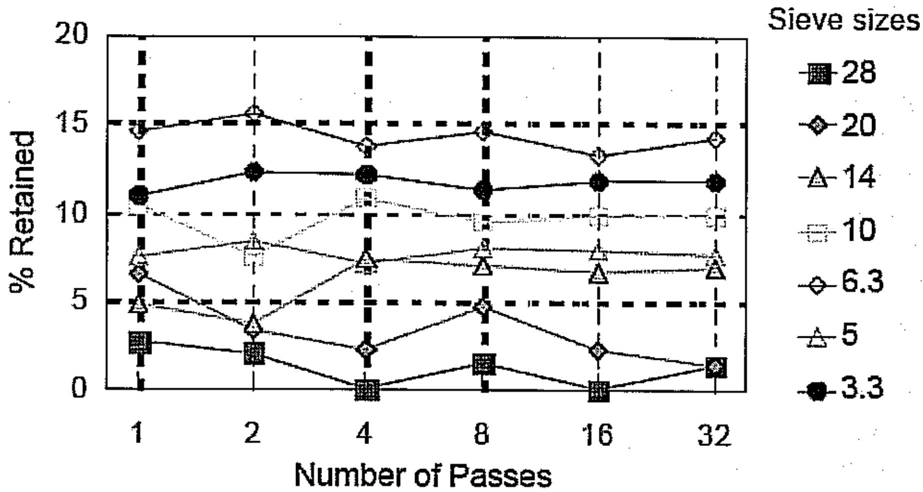


Figure 2.6 - Damage of an Ash Aggregate during Compaction (Dawson, 2003)

Aggregate Toughness

Changes in gradation and aggregate toughness can be measured in the laboratory by the British test procedures (British Standard 812-112:1990) of aggregate impact value (AIV) and aggregate crushing value (ACV). For AIV, a coarse aggregate sample contained within a mold is used to perform the test procedure. The sample is subjected to successive blows from a falling hammer to simulate its resistance to rapid loading. The resulting sample is sieved with the AIV being the amount of fines passing the 2.36mm sieve (No. 8 sieve); and, expressed as a percentage of the initial sample weight. The AIV is given by the following equation:

$$AIV = \frac{M_2}{M_1} \times 100\% \tag{2.1}$$

where M_1 is the mass of test specimen and M_2 is the mass of the specimen passing No. 8 sieve. For weak aggregates (AIV>30) the test produces excessive fines which buffers the remaining particles thus preventing the completion of the test. Consequently, a modified AIV was developed in which the number of blows is reduced to produce between 5% and 20% of fines. The modified AIV is equal to the total amount of fines produced and the number of blows given, proportioned to the standard 15 blows.

The ACV is a value which indicates the ability of an aggregate to resist crushing. The lower the figure is, the stronger the aggregate or the greater its ability to resist crushing will be. A sample of 0.55 in. (14 mm) size aggregate is placed in a steel mold and a steel plunger is inserted into the mold on top of the aggregate. The aggregate is then subjected to a force rising to 90 kip (400kN) over a period of 10 minutes. This test is typically performed by placing in a concrete crushing apparatus. The fine material, which is produced and passes the 0.09 in. (2.36 mm) sieve, is represented as a percentage of the original mass. This percentage is the ACV. Similarly, the ACV is also calculated by using Equation 2.1.

Stabilizer Related Issues

Chemical (calcium-based) stabilizers have been used for decades to improve strengths of substandard base materials. Along with the improvement of strength and workability of these materials, there are also inherent adverse reactions that may occur due to the combinations of chemicals within the soil and the stabilizers. One of the more common problems is the sulfate reaction with lime, cement or fly ash. The sulfate reaction manifests as swelling or heave of the soil. Four components must be present to cause sulfate-induced distress in stabilized soils: calcium, aluminum, water and sulfates.

These components must be present in the appropriate proportions to produce calcium-aluminate-sulfate-hydrate minerals, which have a very large expansion potential, in some cases as high as 250% (www.lime.org). One of these minerals is ettringite and is capable of retaining very large quantities of water within its structure. During the formation of ettringite very high swell pressures can develop, and very large volume increases can and do occur. The formation of ettringite and similar problematic minerals can be prevented by decreasing the amount of any one of the four components previously mentioned. As lime, water, and clay are combined, aluminum is released from the clay due to the high pH system produced by the reaction of lime and water. If the soil contains a high sulfate concentration in the form of gypsum, then all the components with the exception of water are present for the formation of the expansive minerals.

Cement treated base or soil cement is a combination of pulverized soil, portland cement and water which forms a durable structural material. Although it is used extensively to improve the soil strength of bases for roads, there are many problems that arise during the curing process. As soil, water, and cement are mixed and compacted, hydration begins and chemical modification of the soil occurs. During hydration, the paste formed by cement and water binds the soil particles together. As mixture cures and hardens, a durable base is formed but the material can also contract and form shrinkage cracks. Additionally, this layer can be characterized as a "low grade concrete slab" (Gaspard, 2000) without joints and reinforcements. Without reinforcement to counter stresses, the cement stabilized base must depend on its tensile strength and the friction with underlying layers to oppose shrinkage. There are other factors that affect shrinkage cracking such as cement content, moisture content, density, compaction, curing and fine grained soils. Regardless of the percentage of cement used in a stabilized base, the unconfined compressive strength (UCS) must be at least 300 psi after seven day moist cure (Scullion, et al., 2003).

One of the major sources in reduction of strength and stiffness of most aggregate bases is moisture infiltration. To counteract pavement failure by moisture infiltration, an increase in stabilizer content is often utilized. But if the treated material is repeatedly exposed to moisture infiltration then the heavily stabilized base is prone to leaching. Leaching is a phenomenon that reverses the stabilizing influence of the chemical treatment (www.sspco.com). Increasing the stabilizer content to reduce the time required to leach the stabilizer from the base is a costly option that many organizations may not consider using. Although the use of cement, lime, and fly ash as stabilizers requires an increase in water to reach the optimum moisture content during compaction, the maximum dry unit weight is reduced. The problem with a reduction in dry unit weight is that the shear strength decreases, chance of future settlement increases, and permeability increases (Liu et al., 2003).

Construction Related Issues

During construction, it is crucial to provide the required moisture to the stabilized base in order to achieve the maximum strength and to provide adequate compaction of the base. Two major factors that contribute to these items are construction practices and type of machinery used in the placement of the stabilized base. For the purpose of this study, the focus is using FDR in conjunction with stabilizers for base construction. In some cases, the existing asphalt concrete pavement is completely removed and the base is prepared and treated. The removed pavement can be further processed by various milling, ripping or pulverizing equipment to produce reclaimed asphalt pavement (RAP). There are other situations, which the RAP is mixed with the base and used in the stabilization construction.

Typically, there are seven steps in the construction of a stabilized base:

- Scarification and pulverization
- Stabilizer spreading
- Preliminary mixing and watering
- Mellowing period (for lime)

- Final mixing
- Compaction
- Final curing

Scarification and Pulverization

After the asphalt concrete pavement layer has been removed, the base can be scarified to the specified depth and width and then partially pulverized to loosen the soil for combination with stabilizers, as shown in Figure 2.7a. If FDR is to be utilized then the asphalt pavement is ripped with a predetermined depth of base as well. This process is shown in Figure 2.7b. A scarified or pulverized base offers more surface contact area for the stabilizer at the time of application.

a) Scarification of base material (www.lime.org) **b) Pulverization of asphalt and base (www.cement.org)**

Figure 2.7 - Scarification of Base Material

For new construction there are gradation specifications that must be followed but just as importantly, there are gradation specifications for the material after it has been pulverized. For example, TxDoT has specification Item 265 (Fly Ash or Lime-Fly Ash Treatment Road Mixed) and Item 275 (Cement Treatment Road Mixed) that require 100% of the pulverized material to pass a 2.5 in. sieve, as shown in Table 2.8. There are critical time limits for cement and fly ash stabilized base to be placed, mixed, and compacted. Compaction for the lime stabilized base is not as crucial due to the range in mellowing period.

Previously, it was common practice to scarify before spreading. Today, because of the availability of superior mixers, additives such as lime are often applied without scarification. Some of the equipment used for scarification and initial pulverization are grader-scarifier and/or disc harrow for scarification and rotary mixer for initial pulverization.

Table 2.8 - TxDoT Specifications for Road Mixed Stabilized Base (TxDoT Construction Specifications, 2004)

Stabilizer	Gradation requirements		Gradation after Pulverization		Mellowing	Compaction	Curing
	Sieve Size, in.	Min. Percent Passing	Sieve Size, in.	Percent Passing			
Cement					None	Within 2 hours of cement application	3 days, by sprinkling or asphalt prime coat
Lime	1.75 0.75	100 85	2.5	100	1-4 days	After mellowing, mix until friable consistency, then compact	Up to 7 days
Fly ash					None	Within 6 hours of fly ash application	Allow 48 hours to dry before applying prime coat, then allow 24 hours before opening to traffic

Stabilizer Spreading

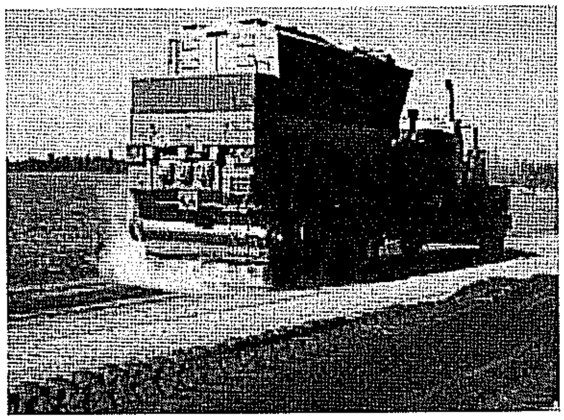
There are several ways that cement, lime or fly ash can be applied. First, the most common method is to spread the dry stabilizer in measured amounts on a prepared soil/aggregate and blend it with a transverse single-shaft mixer to a specified depth. Another method is to spread cement, lime, or fly ash slurries using a slurry jet mixer with a recirculation pump. This method is used to reduce dusting and improve mixing with the base. To insure that the correct quantity of stabilizer is spread, a pan or cloth of known area can be placed on the ground between the wheels of the spreader truck as it drives across the site. The collection container with the stabilizer is weighed to insure that the quantity of stabilizer being spread is correct.

The stabilizers can be applied in dry or slurry form to the prepared base. More commonly, windrows are constructed along each side of the roadbed to prevent runoff and loss due to wind. For example, the application of lime slurry, dry cement, and moist conditioned fly ash with windrows is shown in Figure 2.8a through 2.8c. Regardless of the method used, the amount of stabilizer applied to a site should not exceed the amount that can be mixed into the soil during the day of application.

Preliminary Mixing and Watering

Preliminary mixing is required to distribute the stabilizer throughout the soil in order to pulverize and add water to begin the chemical reaction process. This mixing can begin with scarification; however, this may not be necessary for some modern mixers. During this process or immediately after, water should be added as shown in Figure 2.9

a) Lime slurry
(www.lime.org)



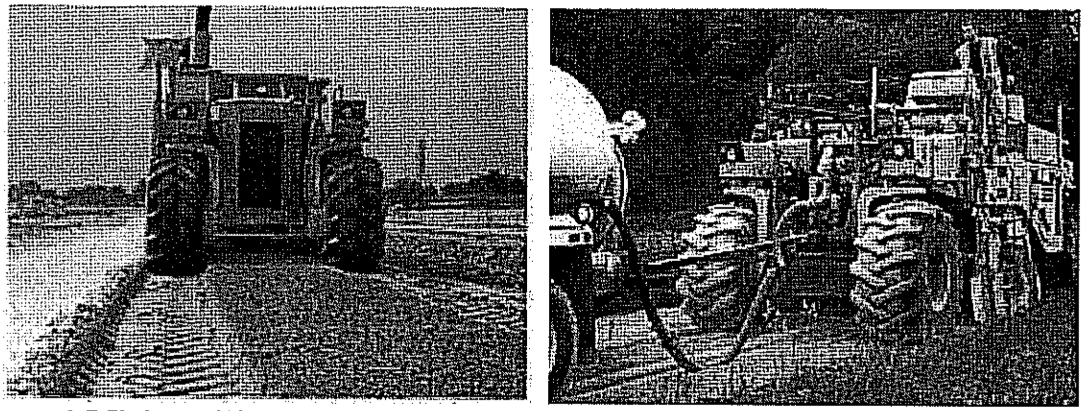
b) Dry cement
(www.cement.org)

c) Moist conditioned fly ash
(www.flyash.info)

Figure 2.8 - Applications of Stabilizers with Windrows

Figure 2.9 - Adding Water after Dry Stabilizer Application
(www.lime.org)

Rotary mixers should be employed to ensure thorough mixing of the stabilizer, soil, and water as shown in Figure 2.10.



a) **Mixing of lime and fly ash (www.lime.org)**   b) **Mixer attached to water truck (www.cement.org)**

Figure 2.10 - Rotary Mixer Used for Initial Mixing

With many rotary mixers, water can be added to the mix drum by attaching a water truck to the mixer during processing. This is the optimal method to add water to dry cement, lime, and fly ash and soil during the preliminary mixing and watering stage. Regardless of the method used for water addition, it is essential that adequate water be added before final mixing to ensure complete hydration and to bring the soil moisture content 3 to 5 percent above optimum for lime, ±2 percent of the optimum for cement and 1 to 3 percent below the optimum moisture content for fly ash before compaction.

Mellowing Period (For Lime)

While cement does not require a mellowing period, lime and lime-fly ash soil mixtures must be allowed to mellow sufficiently to allow the chemical reaction to change (break down) the material. The duration of this mellowing period should be based on engineering judgment and is dependent on soil type. The mellowing period is typically 1 to 7 days. After mellowing, the soil should be remixed before compaction. For low plasticity index soils, or when drying or modification is the goal, mellowing is often not necessary (www.lime.org).

Final Mixing and Pulverization

Final mixing and pulverization is applicable to cement, lime and fly ash treated base materials. As mentioned previously, mixing and pulverization should continue until 100 percent of stone material passes the 1.75 in. sieve and at least 85 percent of material passes the 0.75 in. sieve. Additional water may be required during final mixing (prior to compaction) to bring the soil to the required optimum moisture content of the treated material. In the case of lime, if the previously mentioned gradation can be met during preliminary mixing, then the mellowing and final mixing steps may be eliminated (www.lime.org).

Compaction

Cement stabilized base must be mixed and compacted within 2 hours of cement application. After the materials are well mixed, it is time for compaction and final grading as shown in Figure 2.11.

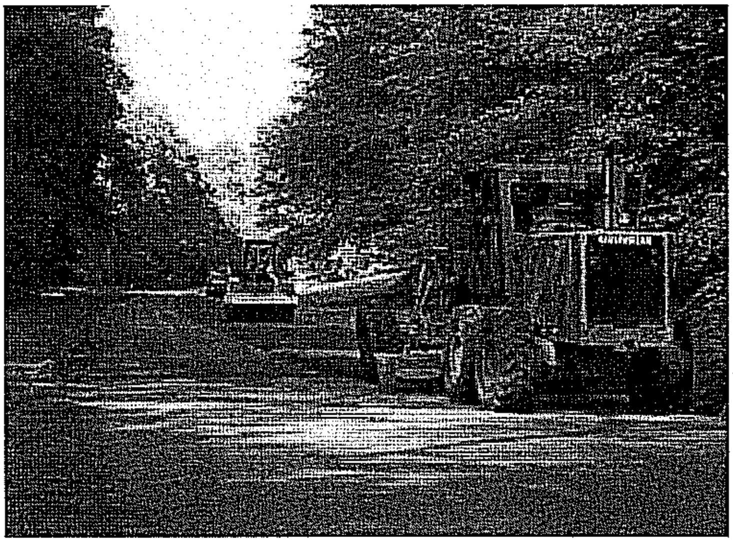


Figure 2.11 - Compaction and Final Grading (www.cement.org)

Smooth-wheeled vibrating rollers, sheepsfoot or tamping rollers can be used to provide initial compaction, as shown in Figure 2.12. Next, smooth-wheeled or pneumatic-tire rollers are used to provide a smooth surface. For lime, compaction must occur immediately after the mellowing period if there is one. Fly ash stabilized soil should be compacted to the density required by specification within 6 hours of fly ash application.

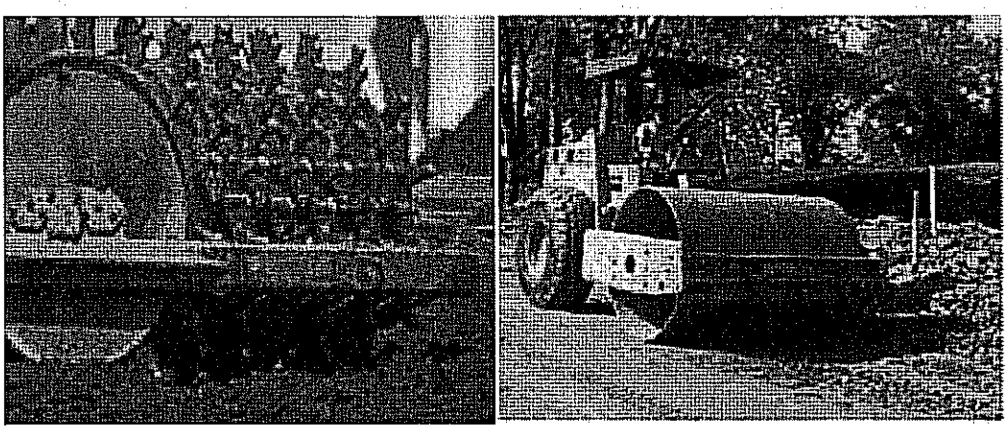


Figure 2.12 - Compaction with Sheepsfoot and Drum Roller (www.lime.org)

Final Curing

Before placing the next layer of pavement, the compacted base should be allowed to harden until heavy vehicles can operate without rutting the surface. During this time, the surface of the stabilized base should be kept moist to aid in strength gain. This curing can be done in two ways by moist curing and membrane curing. Moist curing consists of maintaining the surface in a moist condition by light sprinkling and rolling when necessary. Membrane curing involves sealing the compacted layer with a bituminous prime coat emulsion, either in one or multiple applications as shown in Figure 2.13. A typical application rate is 0.10 to 0.25 gallons/square yard.

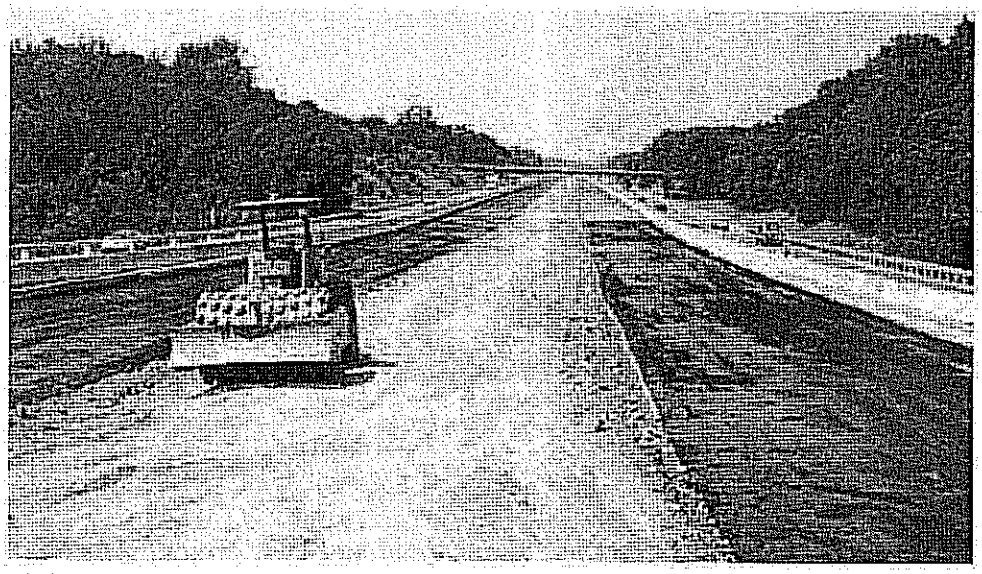


Figure 2.13 - Placing Prime Coat to Retain Moisture to Allow Curing (www.lime.org)

Curing of cement stabilized base requires a minimum of three days using sprinkling or prime coat (TxDoT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, 2004). Lime stabilized base requires a period of up to 7 days to cure before construction can continue. The strength gain and compacted density of fly ash-treated soil are sensitive to compaction delays. The compaction delay can significantly decrease compacted unit weight and strength gain. As the ash hydrates, the fly ash-soil mixture flocculates and agglomerates. While uncompacted, the mixture tends to become aggregated and requires more compaction effort to break up the cemented particles. According to a study performed by White et al. (2005), compaction delay decreases the densities by 10 pcf or more. The study also found that the loss of strength is probably due to the loss of cementitious reaction products expended during hydration and the loss of particle to particle contact points that result from a lower compacted density. Materials compacted without delay after mixing show evidence of six to twelve times the strength of non-stabilized soils. Mixtures compacted at times exceeding one hour only show an increase in strength three to five times that of non-stabilized soils. This decrease in strength can be as much as 50% compared to the material compacted without delay. Also, the unconfined compressive strength and CBR of low plasticity clay with 20% fly ash decreased after a two hour compaction delay. Therefore, it is vital that self-cementing fly ash-stabilized materials be compacted within two hours of initial mixing (White, et al., 2005). Since the base strength is substantially dependent upon compaction delay times, it is important to have a well organized construction operation as well as stabilizing at a mixture-specific moisture content.

Density Related Issues

Field density of compacted soil-cement can be determined by the nuclear gauge method or sand-cone method or volumeter method (Tex-115-E). The optimum moisture and maximum density must be determined prior to start of construction and can be found by using TxDOT procedure Tex-113-E: Laboratory Compaction Characteristics of Moisture Density Relationship of Base Material or ASTM D 558 or AASHTO T 134. Typically, the base is compacted to at least 95% of the maximum dry density achieved through laboratory tests. For TxDOT construction procedures, cement, lime or fly ash treated base must be compacted as stated in the given specification as shown in Table 2.9

Table 2.9 - TxDOT Specifications for Stabilized Base Material (TxDOT, 2004)

Specification		Procedure
Item 260	Lime Treatment (Road Mixed)	Compaction of bottom course at least 95% of maximum dry density obtained from Tex-121-E, compact subsequent courses at least 98% of Tex-121-E
Item 265	Fly Ash or Lime-Fly Ash Treatment (Road Mixed)	Compaction of bottom course at least 95% of maximum dry density obtained from Tex-127-E, compact subsequent courses at least 98% of Tex-127-E
Item 275	Cement Treatment (Road Mixed)	Compact to at least 95% of maximum dry density obtained from Tex-120-E.

Curing Related Issues

A typical curing practice for stabilized bases involves sealing the base layer after compaction with varying coatings. This allows the stabilizer to hydrate and gain the required strength per specifications prior to placing the remainder pavement layers. Availability of moisture, temperature during curing, and length of cure time all affect the strength gain of stabilized soils, particularly fly ash treated soils. Usually, mixtures are cured by sprinkling with water or by coating with a thin layer of emulsion or cutback asphalt. The Joint Departments of the Army and Air Force and the Federal Highway Administration (FHWA) recommend that the sealer be applied within one day of completing the section and that multiple coats may be required (Singh, 2001). Completed sections can also be cured with water for a short time and then sealed with thin coats of asphalt products. Before heavy traffic or surface layers are placed, the completed sections should be cured for three to seven days. From observations by the Joint Departments of the Army and Air Force, paving can begin within a day or two after completing the stabilized section, so long as the subgrade can support paving traffic (Singh, 2001). In contrast, a cure time of 28 days for fly ash stabilized base was specified for one project in eastern Iowa. For this type of situation, it has been recommended that a protective layer of crushed stone be applied to areas where traffic will be present before paving is completed. Conversely, the protective layer can delay the release of the volatiles in an asphalt seal coat. Reportedly, the volatiles react negatively with the stabilized base and inhibit strength gain during curing (Singh, 2001).

Monitoring of Sections

Monitoring of sections is very important because it allows for detection of pavement performance problems that may have developed during and after construction. While destructive testing provides a field sample for analysis of various strength parameters, nondestructive testing (NDT) is equally important in providing these parameters as well as surface properties but without disturbing the pavement and the underlying layers.

Nondestructive Testing

A popular nondestructive testing device is the Falling Weight Deflectometer (FWD). The FWD (Figure 2.14) is a device capable of applying dynamic loads to the pavement surface, similar in magnitude and duration to that of a single heavy moving wheel load. The response of the pavement system is measured in terms of vertical deformation, or deflection, over a given area. Through a backcalculation process, the moduli of the layers are determined.

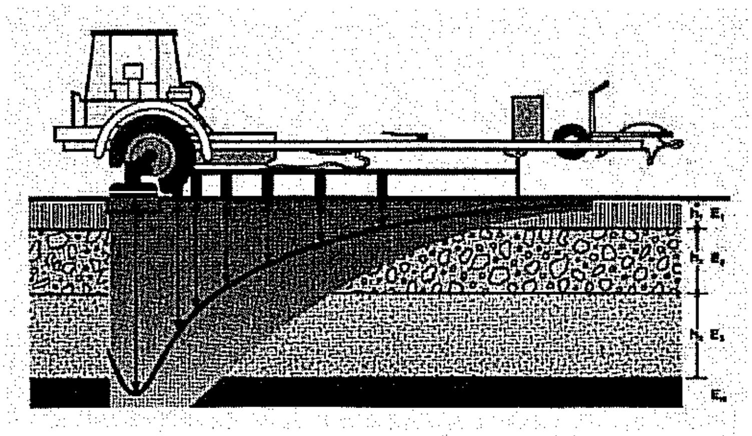


Figure 2.14 - Pavement Response to FWD Load (www.aidpe.com)

Another device used for pavement evaluation is the portable seismic pavement analyzer (PSPA) as shown Figure 2.15. The PSPA is a hand held nondestructive test device capable of providing pavement stiffness.

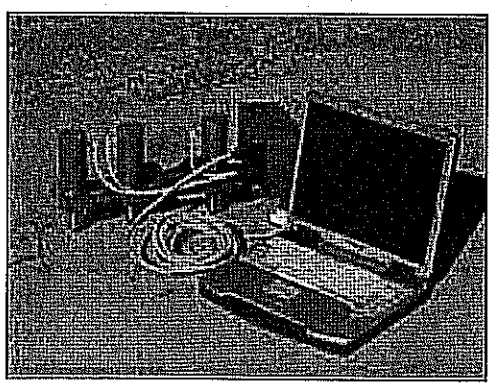


Figure 2.15 - Portable Seismic Pavement Analyzer (www.cflhd.gov)

Propagation of stress waves due to impact is illustrated in Figure 2.16. A momentary stress pulse is initiated onto a test object by mechanical impact on the surface. The stress pulse propagates into the object along spherical wave fronts as P (compressional) and S (shear) waves. Simultaneously, a surface wave (R wave) travels along the surface moving away from the point of impact. The PSPA utilizes the ultrasonic surface wave (USW) method to estimate the modulus of an exposed layer.

As long as the wavelengths studied are smaller than the thickness of the exposed layer, the modulus computed will be indicative of that layer. The USW method uses this approach with two receivers to measure the properties of the exposed layer. The modulus measurement with a PSPA at each location takes approximately fifteen seconds.

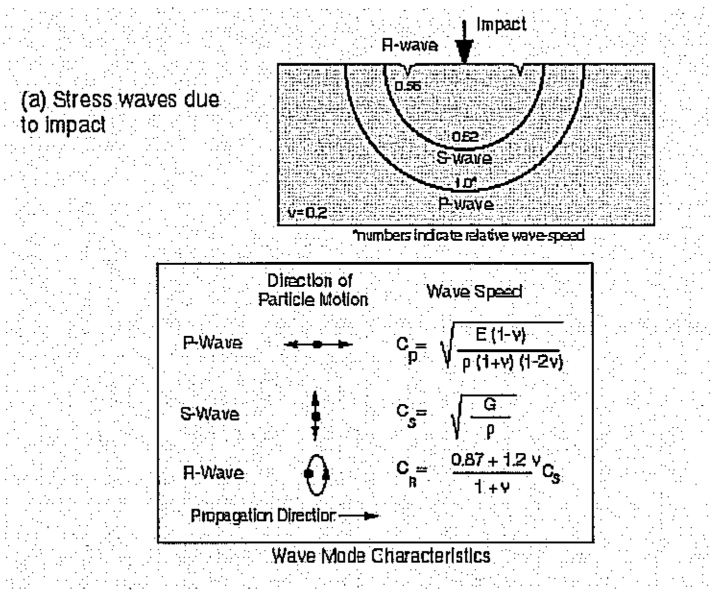


Figure 2.16 - Stress Wave Propagation (http://ciks.cbt.nist.gov/)

Destructive Testing

Two destructive methods commonly used for testing during and after construction are Dynamic Cone Penetrometer (DCP) and coring. While nondestructive testing provides information of the response of a pavement to an applied load, destructive testing provides the in situ strength parameters of the soil through laboratory testing. For example, Iowa State University researchers have monitored the compressive strength of stabilized subgrade materials on two projects in which hydrated fly ash and conditioned fly ash were used as fill materials. In-service testing involved coring the pavement to recover samples for unconfined compression testing, as well as, DCP testing. Coring was used successfully to monitor the strength gain of a cement-fly ash–stabilized base in Des Moines County, Iowa. Coring and sample extraction allow visual observation of the subgrade material in addition to the strength testing data (White et al., 2005). Because of its relative ease to use, a DCP test can be completed in five to ten minutes, which depends on the test depth and stiffness of the material.

Impact of Pulverization on Structural Integrity

Aggregates used in road construction are typically tested for their suitability as a pavement construction material. The goal is to utilize an aggregate that is resilient and will last the design life of the road and not affect the performance because of excessive deterioration. Although aggregate properties such hardness and porosity can be determined though laboratory testing, it is difficult to predict an aggregates long-term performance and how well it will respond to construction practices and dynamic traffic loading.

Aggregate is a durable material that can tolerate much mishandling but its initial characteristics and properties can provide an indication to the type of degradation that may occur. For example, an aggregate's mineralogical composition and climatic exposure can indicate its suitability for various construction applications. According to research performed by Wylde (1976), mineralogical and properties of the source rock directly affect various service conditions for which the resulting aggregate will be utilized. The relationship between service conditions and mineralogical properties are illustrated in Figure 2.17.

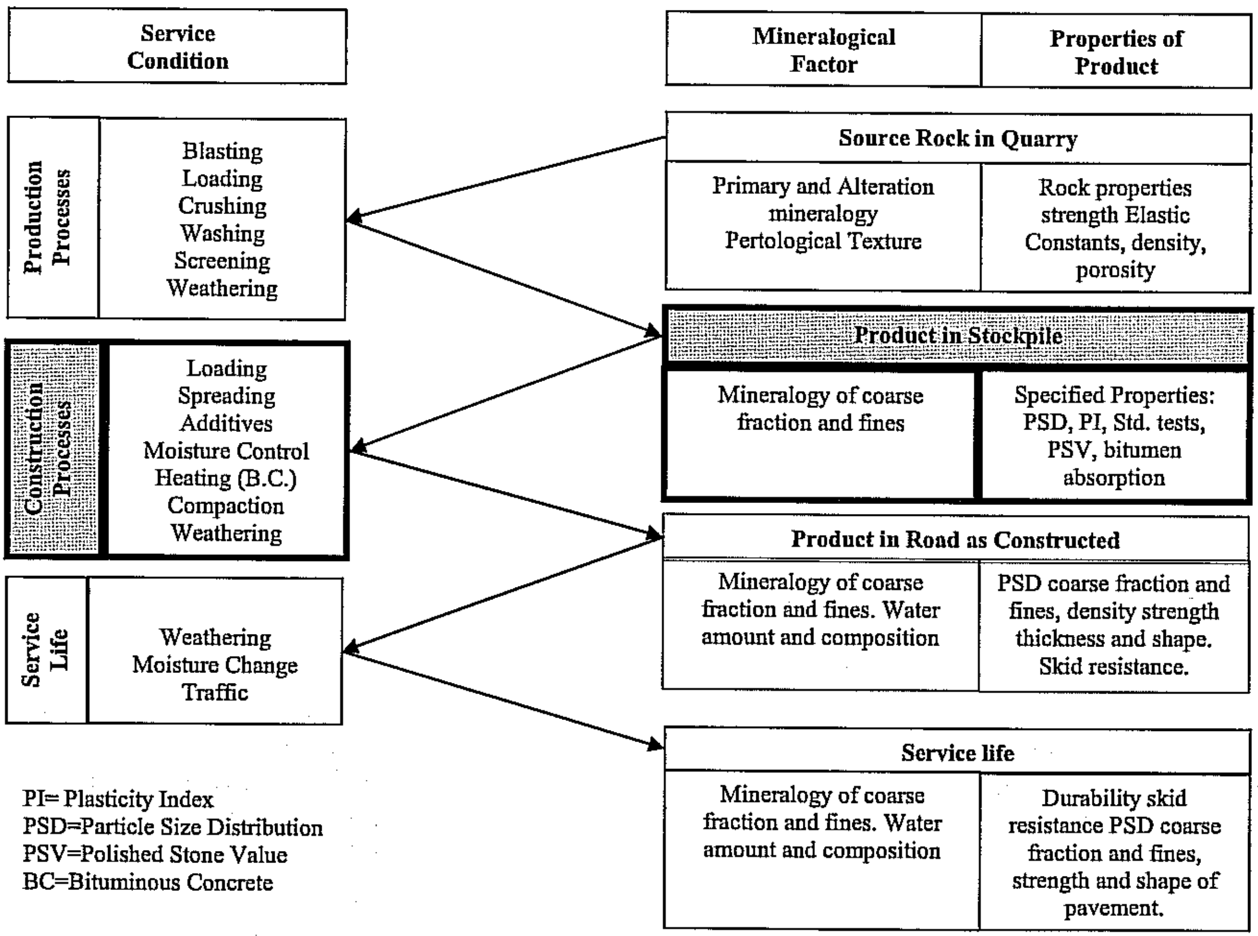


Figure 2.17 - Relationships between Main Variables Affecting Durability of Crushed Rock and Aggregates in Roads (Wylde, 1976)

From Figure 2.17, the construction process would increase the portion of fines due to pulverization. Excess fine material from pulverization can reduce soil strength; therefore, granular base should have high stability, chiefly in a flexible asphalt pavement structure. For base material, it is preferred to use a dense graded mix with large and angular aggregate. It is preferred that the aggregate consist of hard, durable particles which will provide stability. In order for the granular base to provide maximum stability, the base should have enough fines to barely fill the voids and the entire gradation should be close to its maximum density. Although, as the base density is maximized at fines content between 6 and 20 percent, the load-carrying capacity decreases when the fines content exceeds about 9 percent (www.tfhrc.gov). Stability also increases with the percentage of crushed particles and an increase in coarse aggregate size.

An increase in fines due to pulverization may tend to cause the base material to have high moisture susceptibility. In a study performed by Tian et al.(1999), it was determined that by using an open-graded granular material with reduced fines allows the base to be less moisture sensitive which improves base performance. The open-graded aggregates are impervious to pore water pressure accumulation and as a result are expected to reduce potential damage to pavement base in saturated conditions. The study also showed that when using crushed aggregates, increasing the moisture content would decrease the resilient modulus values. However, the use of the equivalent compaction energy gives different dry densities depending on the gradation of the aggregate. This demonstrates that a high dry density resulted in high resilient modulus values and the base gradation has a great influence on these two properties.

The base failure manifestations for flexible and rigid pavements are presented in Tables 2.10 and 2.11. A common factor to pavement base failures is the presence of high fines content. The flexible pavement failures constitute fatigue cracking and rutting. In the case of fatigue cracking, the base stiffness is very low and deflections/strains in the asphalt surface become high. For rutting to occur, the shear strength in the base is not sufficient and there is lateral displacement of soil particles. The rigid pavement failures with high fines content are cracking and pumping/faulting. Cracking in a rigid pavement occurs when there is insufficient shear strength and low stiffness. Due to these actions, there is an increase in tensile strength under repeated wheel loads and the crack initiates at the bottom of the slab and propagates to the surface. Pumping/faulting is due to water seepage into the base that creates fines slurry and is ejected by repeated wheel loads. Since there is loss of base material because of the pumping action, the adjoining slabs may no longer be at the same elevation.

Table 2.10 - Flexible Pavement Distresses and Contributing Factors (Hall et al., 2001)

DISTRESS	DESCRIPTION OF DISTRESS	BASE FAILURE MANIFESTATION	CONTRIBUTING FACTORS
Fatigue Cracking	Fatigue cracking first appears as fine longitudinal hairline cracks running parallel to one another in the wheel path and in the direction of traffic; as the distress progressed the cracks will interconnect, forming many-sided, sharp angled pieces (resulting in the commonly termed alligator cracking); eventually cracks become wider and in later stages some spalling occurs with loose pieces prevalent. Fatigue cracking occurs only in areas subjected to repeated traffic loading.	Lack of base stiffness causes high deflection/strain in the asphalt concrete surface under repeated wheel loads, resulting in fatigue cracking of the asphalt concrete surface. The same result can also be caused by inadequate thickness of the base. Changes in base properties with time can render the base inadequate to support loads.	Low modulus base Improper gradation High fines content High level moisture Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Rutting	Rutting appears as a longitudinal surface depression in the wheel path and may not be noticeable except during and following rains. Pavement uplift may occur along the sides of the rut. Rutting results from a permanent deformation in one or more pavement layers or subgrade, usually caused by consolidation and/or lateral movement of the materials due to load.	Inadequate shear strength in the base allows lateral displacement of particles with applications of wheel loads and results in a decrease in the base layer thickness in the wheel path. Rutting may also result from consolidation of the base due to inadequate initial density. Changes in base properties with time due to poor durability or frost effects can result in rutting.	Low shear strength Low density of base material Improper gradation High fines content High level moisture Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
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. Depressions can contribute to roughness and can cause hydroplaning when filled with water.	Inadequate initial compaction or non-uniform material conditions result 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
Frost Heave	Frost heave appears as an upward bulge in the pavement surface and may be accompanied by surface cracking, including alligator cracking with resulting potholes. Freezing of underlying layers resulting in an increased volume of material causes the upheaval. An advanced stage of the 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.

Table 2.11 - Rigid Pavement Distresses and Contributing Factors (Hall et al., 2001)

DISTRESS	DESCRIPTION OF DISTRESS	BASE FAILURE MANIFESTATION	CONTRIBUTING FACTORS
Cracking	Longitudinal cracks parallel to the pavement centerline, generally along the wheel path (typically divide slab into two pieces). Cracks result from load applied stresses that exceed the flexural strength of the portland cement concrete (PCC). Fatigue cracking generally results from repeated load stresses but may also be caused by thermal gradients and moisture variations. Corner breaks appear as hairline cracks across slab corners where the crack intersects the joints less than 6 ft. from the corner; cracking progresses to result in several broken pieces with spalling of crack and faulting at the crack or joint up to ½ in. or more. The corner break is a crack completely through the slab(as opposed to corner spalls, which intersect the joint at an angle).	Inadequate support from the base/subbase resulting from inadequate shear strength and/or stiffness can increase tensile stresses of slab under repeated wheel loads and result in longitudinal cracking; cracking initiates at the bottom of the slab and propagates to the surface and migrates along the slab; when a crack develops, increased load is placed on the base resulting in deformation within the base and surface roughness of the pavement; the crack introduces moisture to the base resulting in further loss of support and thereby further deformation and roughness. Corner breaks (and associated faulting) may be caused by lack of base support; loss of base support may result from erosion and pumping of the base material; freeze-thaw damage of the base may contribute to loss of support.	Low base stiffness and shear strength Pumping of base/subgrade fines Low density in base Improper gradation High fines content High level moisture Lack of adequate particle angularity and surface texture Degradation under repeated loads and freeze-thaw cycling
Pumping/ Faulting	Begins as water seeping or bleeding to the surface at joints or cracks and progresses to fine material being pumped to the surface; ultimate condition is an elevation differential at the joint termed faulting. Pumping action is caused by repeated load applications that progressively eject particles of base and subgrade from beneath the slabs.	Pumping involves the formation of a slurry of fines from a saturated base or subgrade, which is ejected through joints or cracks in the pavement under the action of repetitive wheel loads.	Poor drainability (low permeability) Free water in base Low base stiffness and shear strength High fines content Degradation under repeated loads
Frost Heave	Differential heave during freezing and formation of ice lenses causes roughness due to uneven displacement of PCC slabs; thaw weakening results in loss of support from base and subgrade which may cause pumping and faulting and corner breaks; under heavy loads the loss of support can result in cracking of slabs.	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. Moisture migrates toward the freezing front. During spring thaw, large quantities of water are released from the frozen zone, which can include all unbound materials	Freezing temperatures Capillary source of water Permeability of material high enough to allow free moisture movement to the freezing zone.

Chapter 3

Evaluation and Modification of Test Protocols

Introduction

As presented in Chapter 2, many factors affect the performance of stabilized bases in terms of strength and stiffness. Therefore, it is important to control material-related factors that may impact the strength or stiffness of the base, such as moisture, soil gradation, and stabilizer type and content. To quantify the impact of these parameters, a limestone base material from El Paso area was used to evaluate and modify the test protocols.

Figure 3.1 illustrates the test protocol employed to assess the current test procedures and to develop a uniform test protocol for sites to be tested. The first step, Preliminary Testing, consists of establishing the gradation, index properties and the hardness of the aggregates. The next step is to establish the moisture-density/moisture-modulus relationships for the raw materials as well as the blends with varying contents of stabilizers. Finally, the strength, stiffness and moisture susceptibility of the mixes are evaluated.

Development of Gradation Curves

Item 247 of TxDOT specification specifies the construction of the flexible base for pavement in terms of material use and construction practices. The base material requirements from Item 247 are presented in Table 3.1. The main requirements beside soil gradation are liquid limit, plasticity index (PI) and compressive strength.

Cooper et al. (1985) proposed an equation to establish stable gradations when the percentage of fine material passes the No. 200 sieve. Their equation is as follows:

$$P = \frac{(100 - \%F)(d^{0.3} - 0.075^{0.3})}{(44.45^{0.3} - 0.075^{0.3})} + \%F \tag{3.1}$$

where P is the percent passing per sieve, F is the percentage of material passing the No. 200

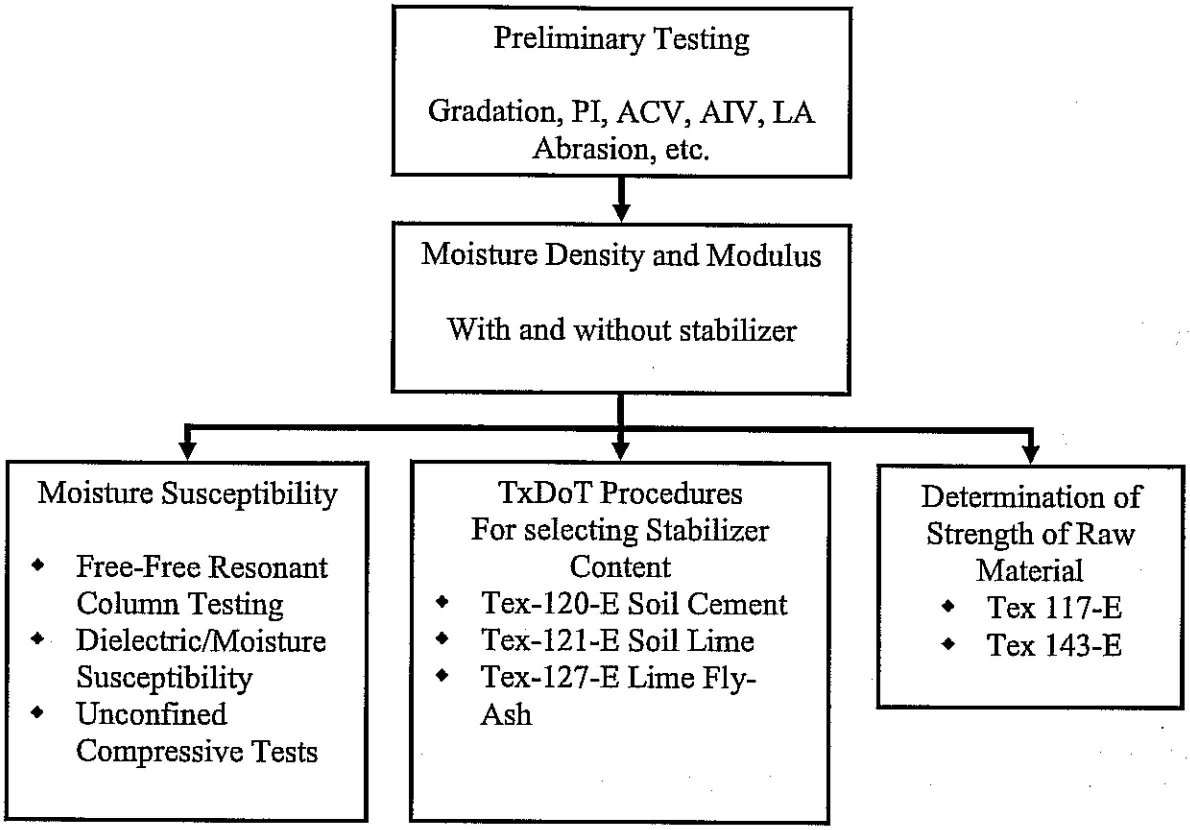


Figure 3.1 - Testing Protocol Developed Based on El Paso Limestone

(0.075 mm) sieve, and *d* is the sieve opening in mm. This formula allows the desired percentage of fines to be used as an input in order to calculate the quantity of aggregate that passes a certain sieve size. For example, by choosing 0%, 5%, 10%, and 20% fines, the gradation curves shown in Figure 3.2 are obtained. The gradation curves with the varying percentages of fines lie within the Item 247 maximum and minimum allowable ranges, except for the blend with no fines. This demonstrates that the current gradation specifications maybe reasonable. However, requirements for a sieve finer that No. 40 is desirable.

One of the concerns with the pulverization activity is the possibility of the change in gradation. The impact of change in gradation was studied using a limestone base from El Paso. The average of minimum and maximum allowable limits for each sieve from Item 247 was used to develop the control gradation. The control gradation, called Avg. 247 hereafter, is shown in Table 3.2 and Figure 3.3, along with the current gradation of the base used.

Three gradation curves were developed that contained excessive sand (ES), excessive fines (EF), and excessive sand and fines (ESF). These gradations are shown in Table 3.3 and Figure 3.4. The ESF curve is identical to 20% fines curve obtained by using Equation 3.1. The EF curve was developed by following the same gradation as Avg. 247 up to the No. 4 sieve, and then following the curve as suggested by Equation 3.1 for a blend with 20% fines. The ES curve was developed similarly by following the Avg. 247 curve up to the 3/8 in. sieve, paralleling the ESF

Table 3.1 - Specification Item 247: Base Material Requirements (TxDoT, 2004)

Property	Test Method	Grade 1	Grade 2	Grade 3	Grade 4
Master Gradation sieve size (% retained)	Tex-110-E				As shown on the plans
2½ in.		-	0	0	
1¾ in.		0	0-10	0-10	
⅞ in.		10-35	-	-	
⅜ in.		30-50	-	-	
No. 4		45-65	45-75	45-75	
No. 40		70-85	60-85	50-85	
Liquid limit, % max.	Tex-104-E	35	40	40	As shown on the plans
Plasticity index, max.	Tex-106-E	10	12	12	As shown on the plans
Plasticity index, min.		As shown on the plans			
Wet ball mill, % max	Tex-116-E	40	45	-	As shown on the plans
Wet ball max. Increase passing the No. 40 sieve		20	20	-	
Classification	Tex-117-E	1	1.1-2.3	-	As shown on the plans
Min. compressive Strength, psi Lateral pressure 0 psi Lateral pressure 15 psi		 45 175	 35 175	 - -	As shown on the plans

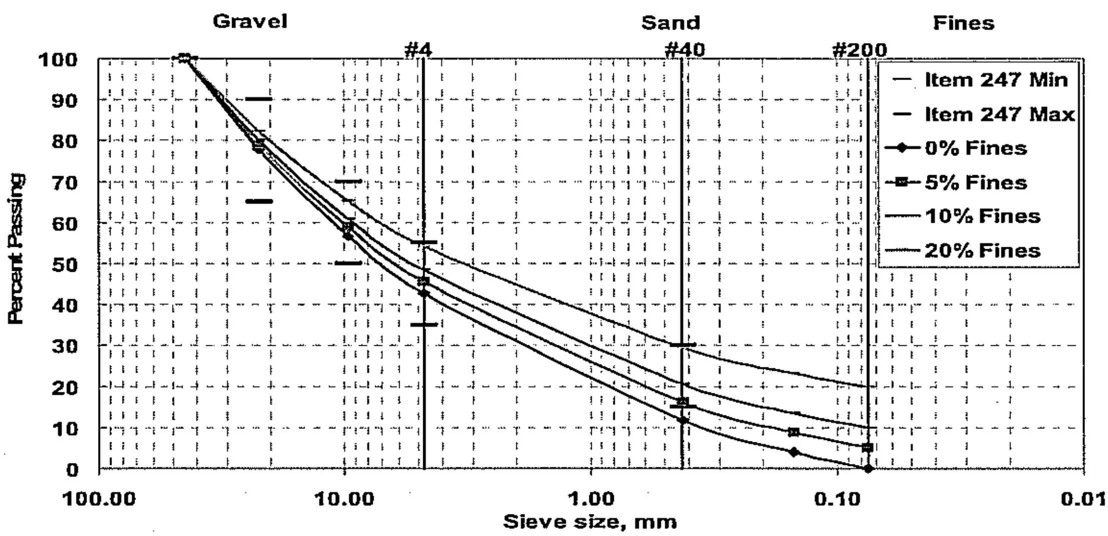


Figure 3.2 - Gradation Curves Developed by Using Cooper Equation

Table 3.2 - Gradation for El Paso Limestone and Average of Item 247

Sieve size	Percent Retained per Sieve		
	Dry Sieve	Wet Sieve	Avg. 247
1¾ in.	0	0	0
⅞ in.	27	26	23
⅜ in.	54	49	41
No. 4	69	62	56
No. 40	86	83	79
No. 100	86	93	90
No. 200	95	95	95
Pan	100	100	100

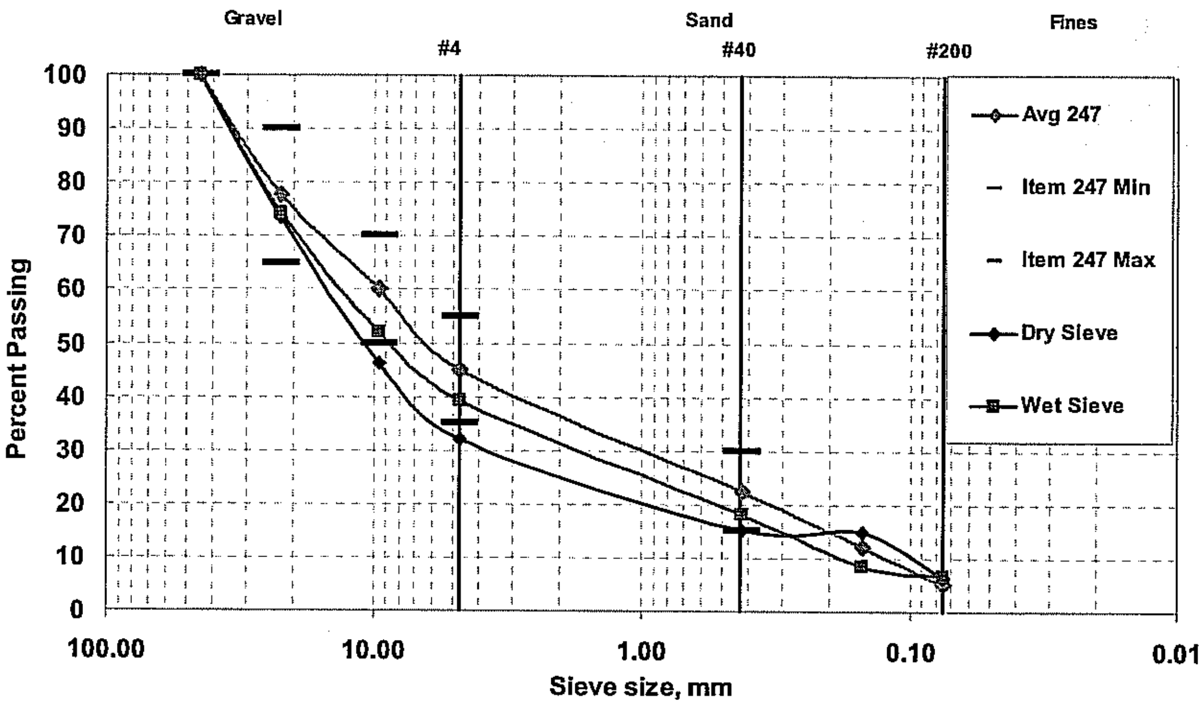


Figure 3.3 - Comparisons of Gradation Curves of El Paso Limestone and Average of Item 247

Table 3.3 - Gradation Curves for Four Blends used in this Study

Sieve	Size, mm	Percent Passing per Sieve			
		Avg. 247	Excess Sand (ES)	Excess Fines (EF)	Excess Sand and Fines (ESF)
1¾ in.	44.450	100	100	100	100
⅞ in.	22.225	78	78	78	82
⅜ in.	9.525	60	60	60	65
No. 4	4.750	45	52	45	54
No. 40	0.425	23	27	28	29
No. 100	0.150	12	15	23	23
No. 200	0.075	5	5	20	20

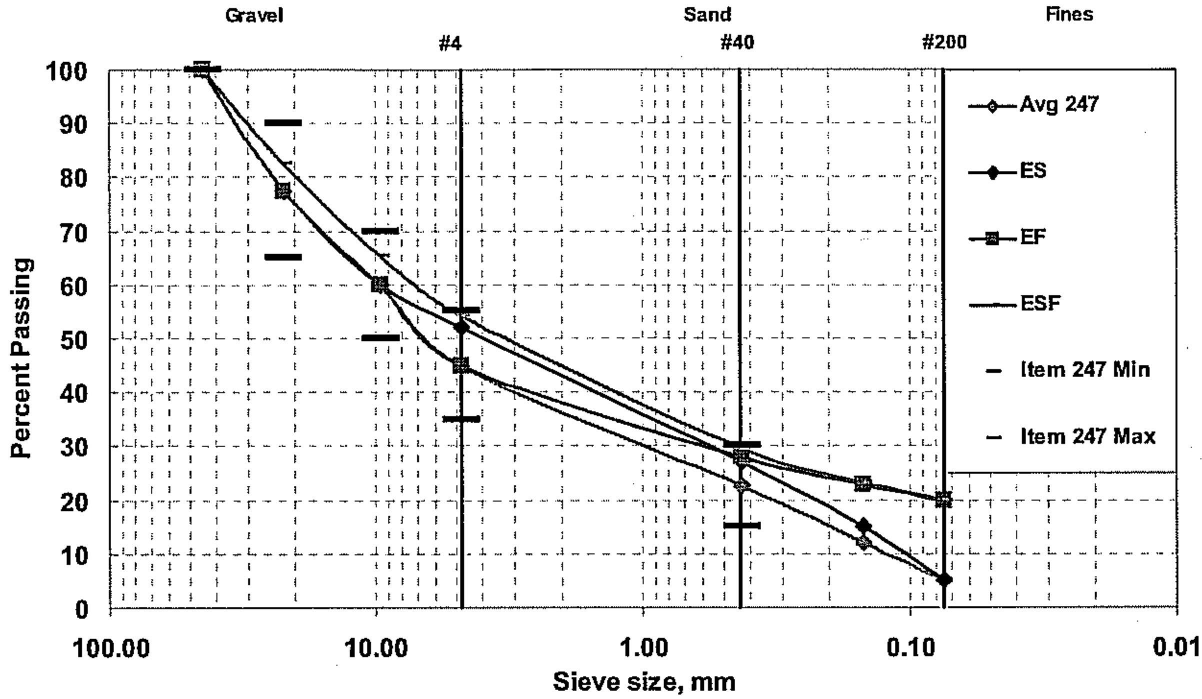


Figure 3.4 - Gradation Curves for Blends Used in This Study

curve from the No. 4 sieve to the No. 40 sieve, and finally following Avg. 247 to 5% passing at the No. 200 sieve.

The liquid limit and plastic limit of the base material are 27 and 19, respectively. As such the plasticity index (PI) of the material is 8.

Texas triaxial tests as per Tex-117-E and Tex-143-E were carried out on the four blends. The results from Tex-117-E correspond to the moisture conditions strength, whereas those from Tex-143-E corresponds more to the strength at optimum conditions. The results are summarized in Table 3.4.

From Tex-117-E test results, Avg. 247, ES, and EF blends resulted in a classification of 1.0 which indicated that the soil is a "good flexible base material." The ESF blend resulted in a classification of 2.9; therefore, the material is considered a borderline one.

The results from Tex-143-E also indicated that the Avg. 247 and ES blends are good flexible base materials because of its classification value of 1.0. However, the EF blend was classified as a Class 2.2, while the ESF blend yielded a classification of Class 1. The minimum compressive strengths for lateral pressures of 0 psi and 15 psi for a Grade 1 base are 45 psi and 175 psi, respectively. As shown in Table 3.4, all blends exceed the 45 psi unconfined compressive strength for 0 psi lateral pressure; but the ES blend marginally fails to meet the minimum value of 175 psi for the 15 psi lateral pressure. Under Item 247, blends Avg. 247 and EF are classified as Grade 1 base material. Similarly, blends ES (marginally) and ESF are classified as Grade 3.

Table 3.4 - Results of Triaxial Testing for El Paso Limestone

Parameters	Tex-117-E				Tex-143-E*			
Gradation	Avg. 247	Excess Sand	Excess Fines	Excess Sand and Fines	Avg. 247	Excess Sand	Excess Fines	Excess Sand and Fines
Classification	1.0	1.0	1.0	2.9	1.0	1.0	2.2	1.0
Angle of Internal Friction, φ	58	49	51	58	59	55	50	53
Cohesion, c, psi	10	14	13	4	8	7	10	9
Strength at Zero Lateral Pressure, psi	62	77	67	55	*	*	*	*
Strength at Lateral Pressure of 15 psi, psi	230	167	185	208	*	*	*	*
Grade as per Item 247	1	3	1	3	*	*	*	*
* Not Applicable								

Selection of Stabilizers

The decision tree for selecting the appropriate types of stabilizer as per current TxDOT guideline (Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures, 2005) is shown in Figure 3.5. The two main factors used are the percentage of material passing the No. 200 sieve and the PI. Since less than 25% of all blends were fines and since the PI was less than 12, cement, lime and fly ash were selected. Preliminary stabilizer contents of 0%, 2%, 4%, and 6% by weight were used.

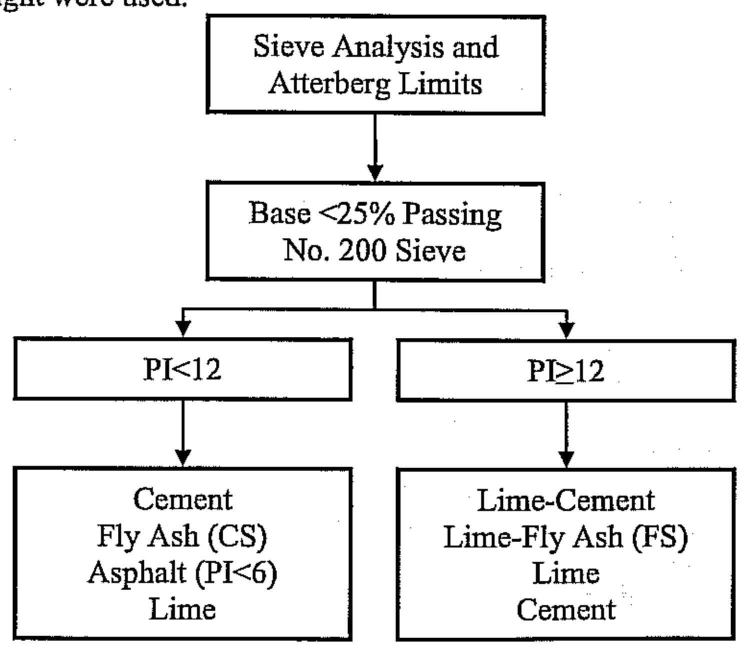


Figure 3.5 - TxDOT Stabilization Selection Decision Tree

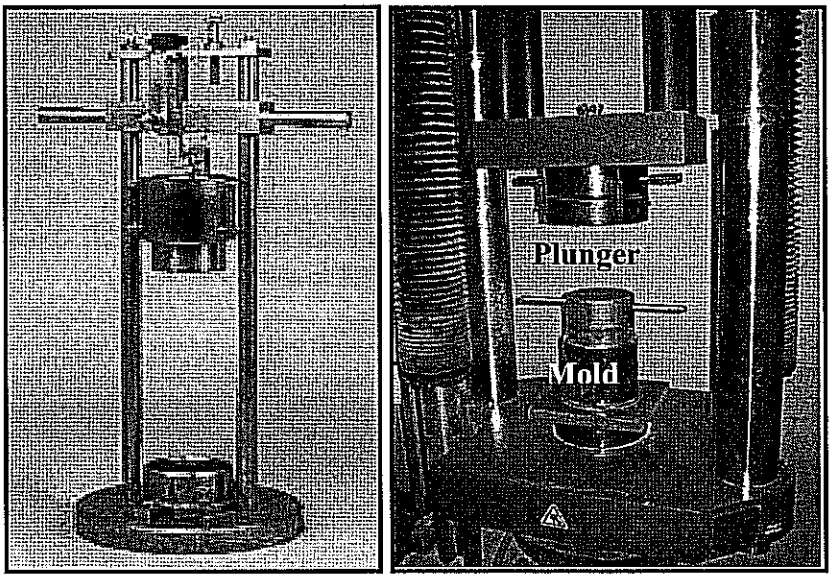
Test Procedures

Hardness of Aggregates

TxDOT employs LA abrasion and MicroDeval tests for determining the resistance of aggregates to crushing. For the aggregate used here the LA Abrasion and the MicroDeval values were 19.0 and 9.5, respectively.

Two other tests as shown in Figure 3.6, the Aggregate Impact Value (AIV) and the Aggregate Crushing Value (ACV), were also used to determine whether the hardness of the rock can be correlated to the potential for degradation of the aggregates during pulverization (see Chapter 2). An AIV greater than 30 indicates a soft aggregate while a low percentage indicated a hard aggregate. Similarly, an ACV below 10 indicates a hard rock while and ACV greater that 35 designates a soft rock.

In the dry state, the El Paso limestone is a fairly tough aggregate as indicated by its AIV of 20 and ACV of 18.5; but after the aggregate has been soaked for 24 hours then impacted, the AIV significantly increases to about 32.



(a) AIV **(b) ACV**

Figure 3.6 - Aggregate Impact Value Apparatus and Aggregate Crushing Value Setup

Moisture-Density and Moisture-Modulus Relationships

For each gradation and stabilizer content, the moisture-density relationships were determined as per test procedure Tex-113-E. The exception to this procedure was allowing the specimen to mellow for one day in order to perform modulus tests. The moisture-modulus relationship was developed as per proposed Tex-147-E on specimens prepared for moisture-density tests. A free-free resonant column (FFRC) device was used for this purpose. In conjunction with obtaining the optimum moisture content and the dry density of the base material, the FFRC provides the stiffness in terms of seismic modulus for different moisture contents.

The FFRC test measures the low-strain modulus of a solid specimen. The working principle of the test is based on detecting the fundamental mode resonant frequencies of vibration of a specimen. As an impulse load is applied to a cylindrical specimen, seismic energy over a large range of frequencies will transmit through the specimen. The objective of FFRC test is to establish the resonant frequencies. The main components in the setup of the test are shown in Figure 3.7. These components are the hammer, accelerometer and sensor box.

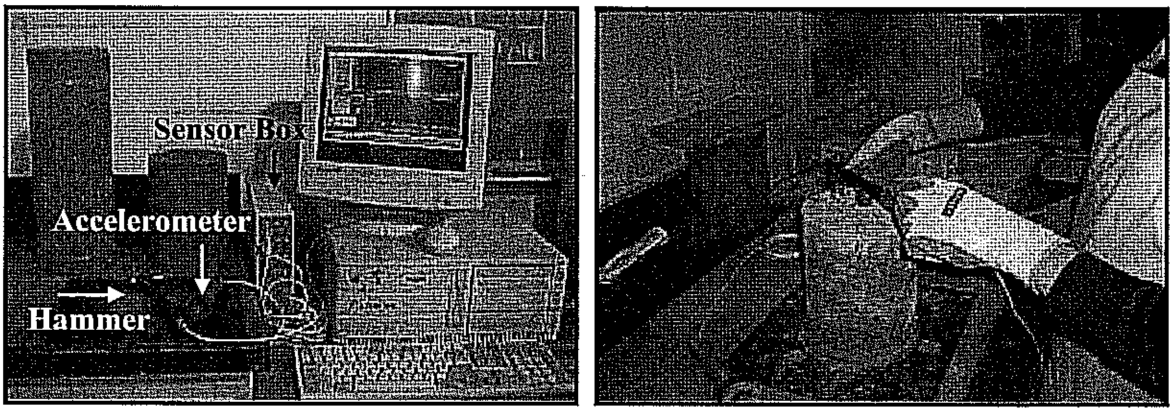


Figure 3.7 - Free-Free Resonant Column System

Typical moisture-density moisture-modulus test results are shown in Figure 3.8. Between moisture contents of about 3% to 9%, the dry unit weight varies from a minimum of 128 pcf to a maximum of 138 pcf at an optimum moisture content of about 6.5%. A change in moisture content of about 6% (from 3% to 9%) results in a change in dry unit weight of 10 pcf (8% of the maximum dry unit weight). The moisture-modulus relationship on the other hand behaves somewhat differently. The maximum modulus is typically achieved at a moisture content that is

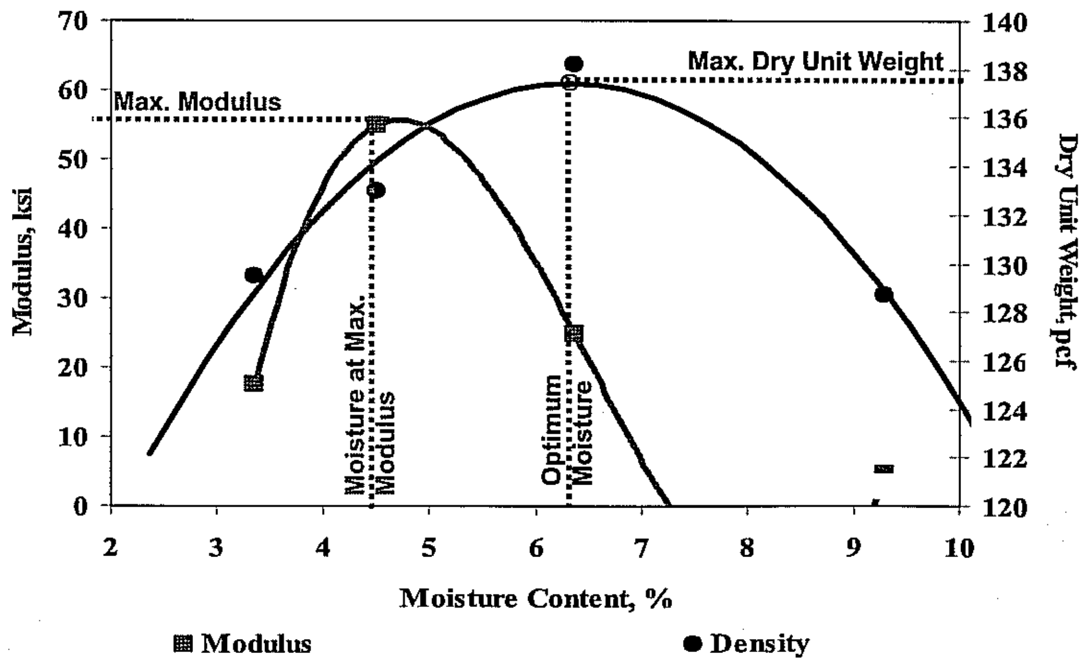


Figure 3.8 - Typical Moisture Density and Modulus Curves for ESF Blend with 2% Fly Ash

dry of optimum moisture content (typically about 2% for Texas bases). In Figure 3.8, the maximum modulus is about 56 ksi. At the traditional optimum moisture content (6.5% here), the modulus is about 22 ksi. At a moisture content of 9.5%, which is 3% above the optimum moisture content, the specimen is too soft as the modulus is only about 5 ksi. In this case, varying the moisture content by 6% (from 3% to 9%) results in a 10 fold change in modulus. This example demonstrates the importance of controlling the moisture content during the construction.

The optimum moisture contents from all gradations and the three stabilizers are shown in Figure 3.9. The optimum moisture contents for the four blends with no additives are fairly similar with an average value of about 7.5%. When the four blends were stabilized with cement, the optimum moisture contents did not significantly change. The optimum moisture content (OMC) generally increased by about 0.3% to 1% (see Figure 3.9a). As shown in figure 3.9b, the OMC increased for the materials with excess sand and excess fines as the content of lime was increased. This trend was not observed for the blend with the excess sand and fines.

As shown in Figure 3.9c, the fly ash specimens demonstrated a reduction in the OMC as the fly ash content increased. The reduction in moisture content was more prevalent for the ES and EF blends with the OMC reducing from 7.5% to 5.2% and 7.7% to 5.1% for 0% fly ash and 6% fly ash. For the Avg. 247 and ESF specimens, the OMC's were about 1% less than the OMC's of the corresponding blends with no fly ash.

As shown in Figure 3.9c, the fly ash specimens demonstrated a reduction in the OMC as the fly ash content increased. The reduction in moisture content was more prevalent for the ES and EF blends with the OMC reducing from 7.5% to 5.2% and 7.7% to 5.1% for 0% fly ash and 6% fly ash. For the Avg. 247 and ESF specimens, the OMC's were about 1% less than the OMC's of the corresponding blends with no fly ash

The variation in maximum dry density with aggregate blend and stabilizer type are presented in Figure 3.10. The highest maximum dry density (MDD) for each blend was obtained when no stabilizer was used except when the fly ash was added. For the four raw blends, MDD varied between 138 pcf and 142 pcf. When the cement was used as the additive, the MDDs varied between 134 pcf and 139 pcf (see Figure 3.10a). With the lime as additive (Figure 3.10b), the MDDs were generally lower than the raw materials with a variation between 126 pcf and 138 pcf. With the fly ash, the MDDs were more or less independent of the gradation and fly ash content as shown in Figure 3.10c. In that case the MDDs varied between 136 pcf and 142 pcf.

Figure 3.11 presents the variation in seismic modulus (SM) at OMC cured for 24 hours with the different blends at varying stabilizer contents. Referring back to Figure 3.8, the maximum modulus usually occurs at a lower moisture content than OMC. As such these moduli correspond to the impact of the stabilizers as well as the difference between the OMC and the moisture content where the maximum modulus occurs. These values may correspond best with those measured in the field during quality management.

The blends with no stabilizer yielded the lowest SM values for all of the gradation-stabilizer mixes. For raw blends, the SM did not exceed 20 ksi.

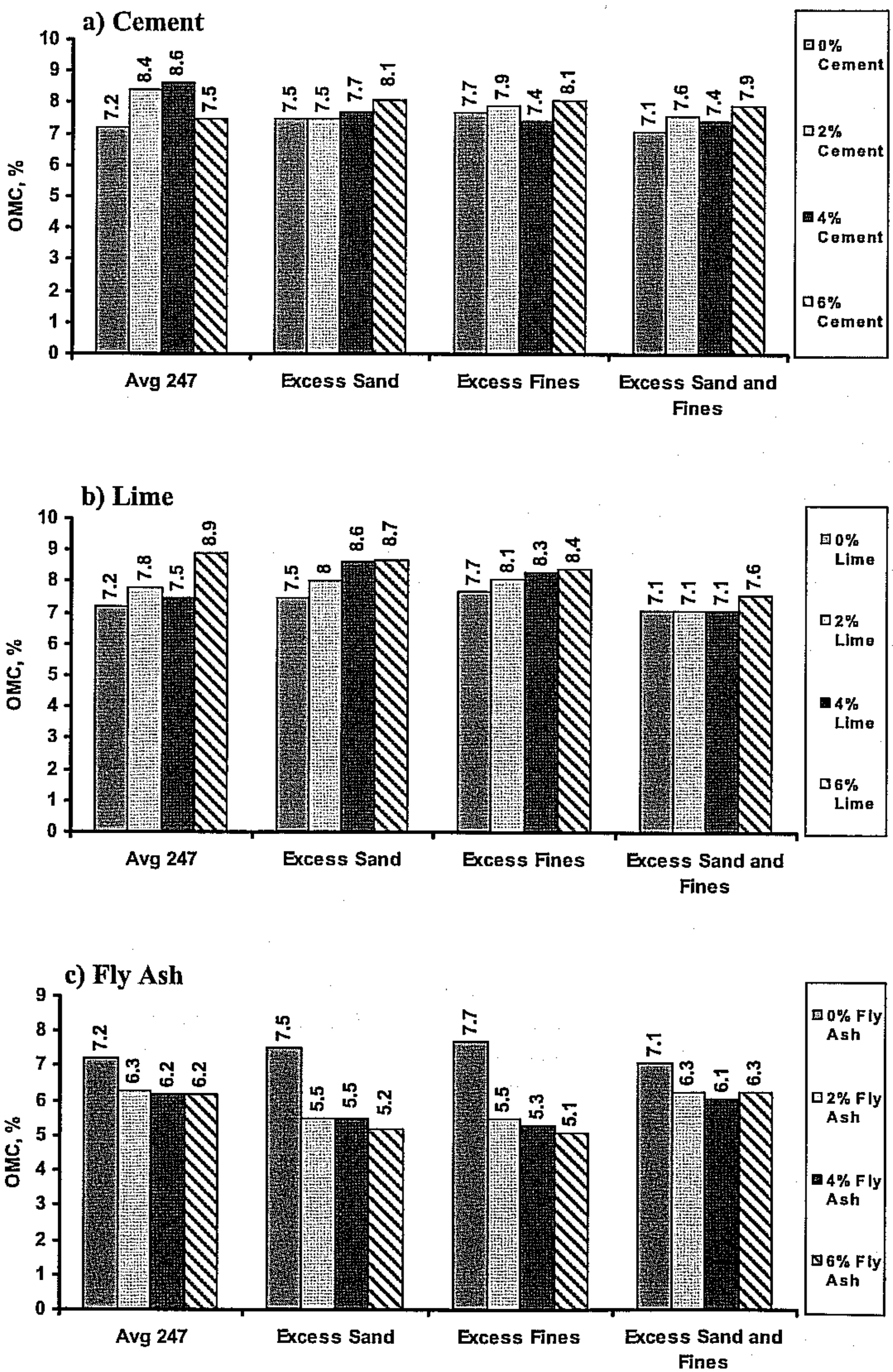


Figure 3.9 - Variation in Optimum Moisture Content with Aggregate Blend and Stabilizer Type

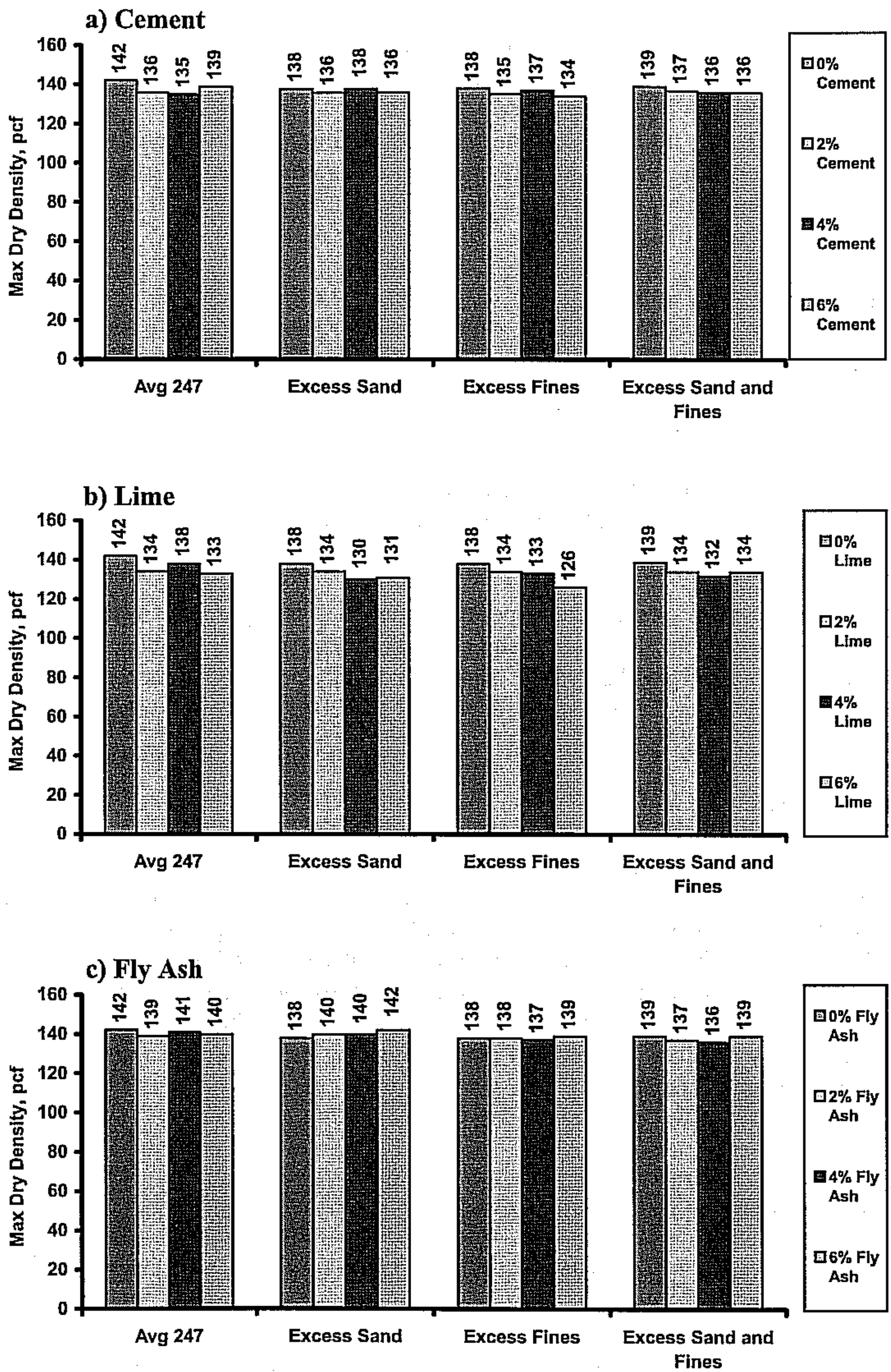


Figure 3.10 - Variation in Max. Dry Density with Aggregate Blend and Stabilizer Type

As shown in Figure 3.11a, the Avg. 247 blend with 6% cement yielded the highest seismic modulus of 816 ksi, and the ESF blend with 2% cement the lowest at 164 ksi. The cement as additive seems to work well with all blends except the EFS blend.

As shown in Figure 3.11b, the lime does not seem to be improving the modulus of the blends much. For the ES blend, the lime is not effective at all. The increase in modulus of the EF and most of the ESF mixes is marginal as well. Some improvement in the stiffness of the Avg 247 blend is observed at higher dosages of lime.

The fly ash does not seem to be very effective, as shown n Figure 3.11c. The only marginal improvement can be observed for the EF and ESF blends. The reason for the significantly high modulus of 70 ksi for the EF blend and 2% fly ash is not know at this time.

Unconfined Compressive Strength

The unconfined compression strength (UCS) test is an axial compression test in which the specimen is provided with no lateral pressure while undergoing vertical compression. All UCS tests were performed following TxDOT test procedure Tex-117-E.

Regardless of the gradation or stabilizer type, each specimen was prepared at the corresponding optimum moisture content for the given blend and stabilizer content. The curing method before testing, however, depended on the type of stabilizer used. For the soil-cement materials, the specimens were cured as per TxDOT test procedure Tex-120-E by placing them in a moist room for seven days.

In the case of the base-lime and base-fly ash specimens, TxDOT test procedures Tex -121-E and Tex-127-E were followed, respectively. This process is demonstrated in Figure 3.12 with a brief description for each step. The curing process consisted of leaving the specimen in a latex membrane for seven days at room temperature, then placing in an oven for six hours at 105°F. After the specimens returned to room temperature, they were wrapped in filter paper to draw water into the specimen through capillary action and finally enclosed in a stainless steel triaxial chamber for ten days. The chamber was pressurized to transmit approximately 1.0 psi of lateral and vertical pressure to the specimen. The specimen enclosed in the triaxial chamber was then subjected to a ten-day capillary saturation. The total curing period for the soil-lime and soil-fly ash specimens is seventeen days.

Once the specimens were cured, they were subjected to unconfined compression strength tests. The strain rate utilized for the testing was 2% per minute to conform to Tex-117-E. An Instron Compression Testing Apparatus as shown in Figure 3.13a was used to test the specimens.

Typical stress-strain curve from a specimen test is shown in Figure 3.13b. The peak strength, strain at peak strength and the modulus of the specimen were determined. The modulus, which corresponds to the straight portion of the stress-strain curve, can be determined by choosing two points along this portion of the curve. These moduli are typically unreasonably small, and as such are of little value for stabilized materials. The residual strength was not calculated since the area of interest for this study is the peak strength of the stabilized material.

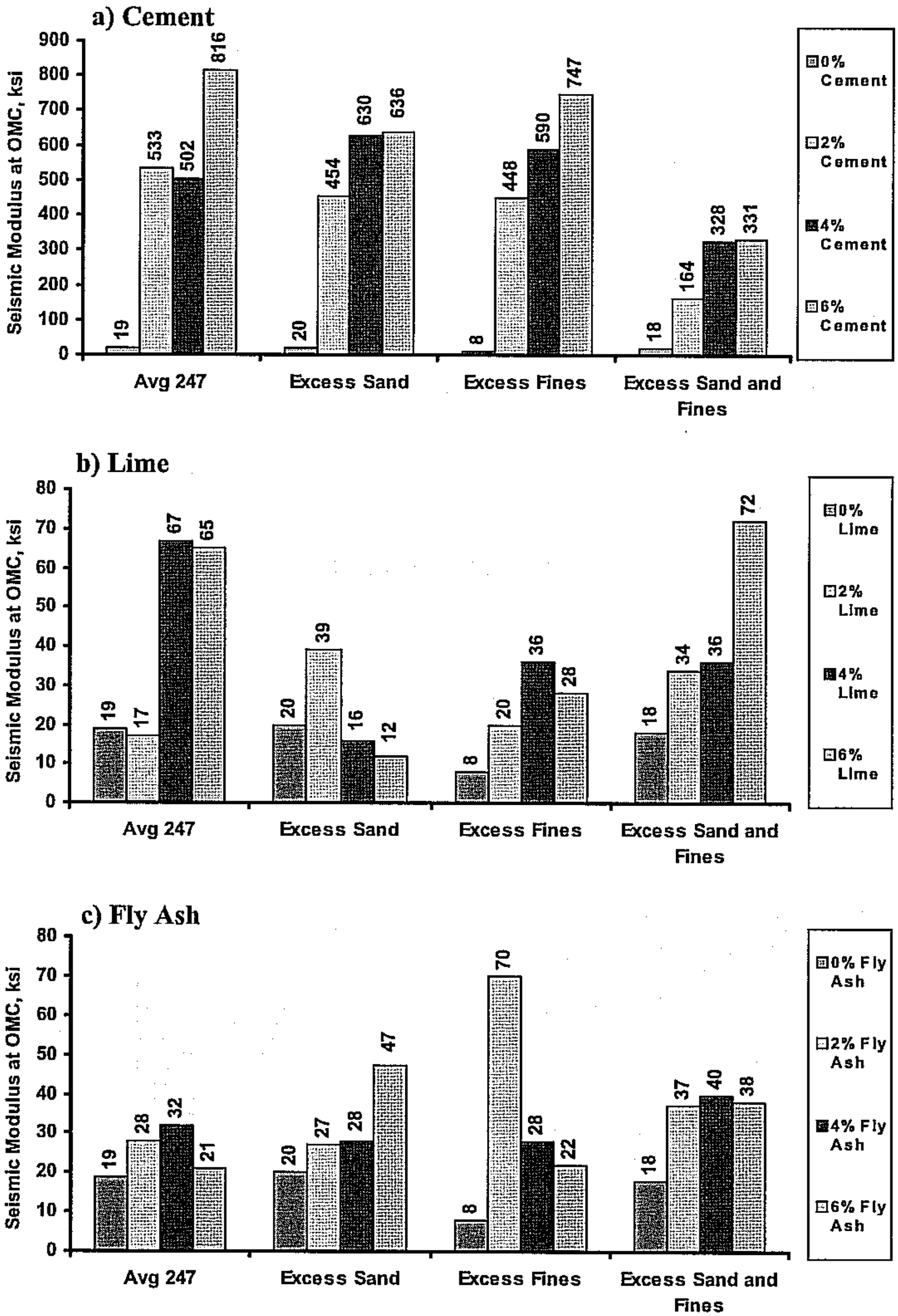


Figure 3.11 - Variation in Seismic Modulus at OMC with Aggregate Blend and Stabilizer Type

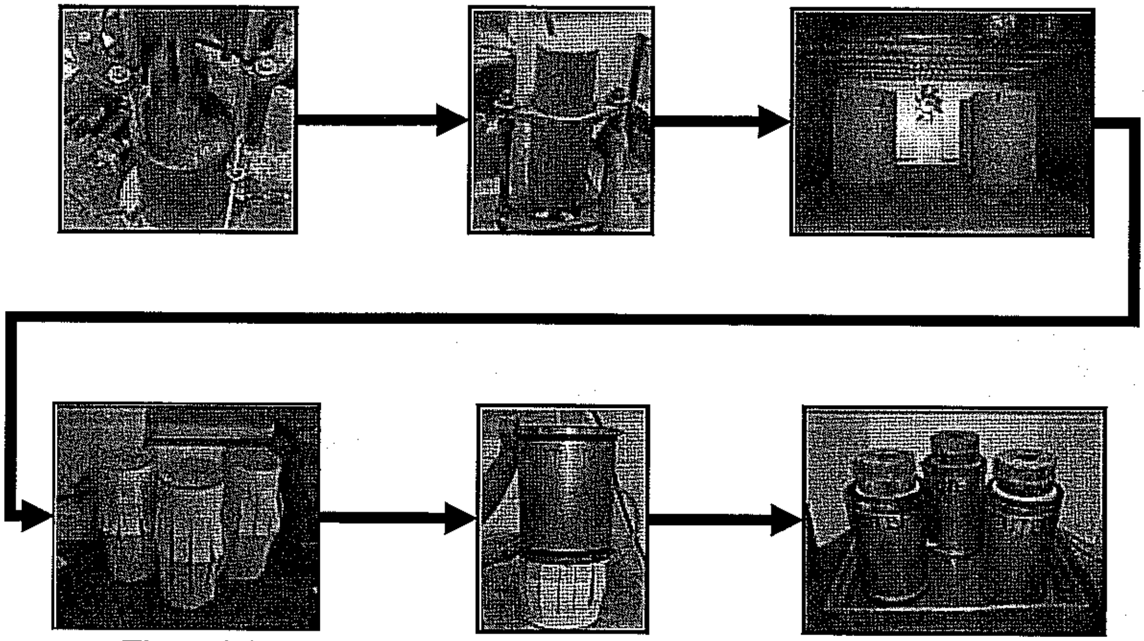
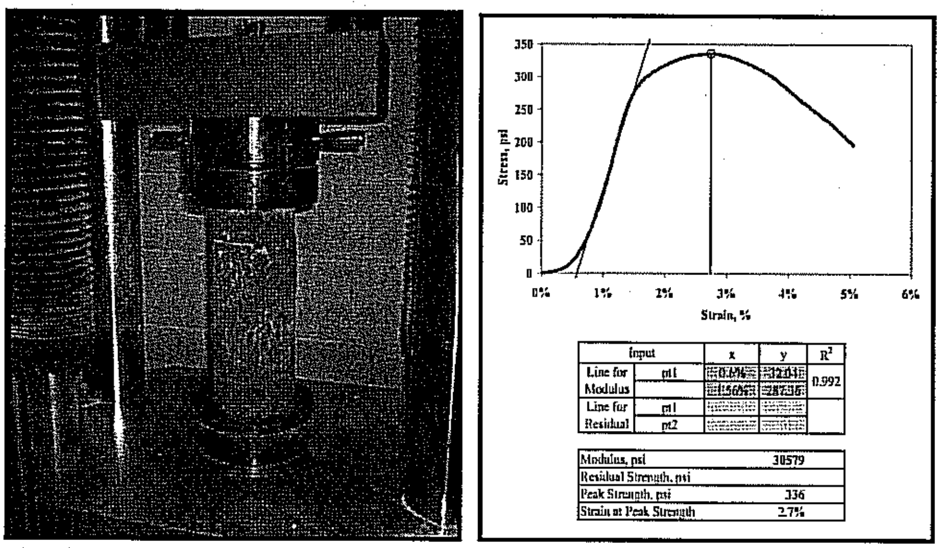


Figure 3.12 - Process for Preparing Soil-Lime and Soil-Fly Ash Specimens



(a) Unconfined Compressive Test (b) Stress-Strain Curve

Figure 3.13 - Compression Testing Apparatus and Stress-Strain Curve

According to Scullion et al. (2003), a value that is greater that 300 psi is a satisfactory strength for a cement stabilized base. The compressive strength limit stated in the TxDOT procedures for soil-lime and soil-fly ash treated base is 150 psi. These limits were applied to the appropriate soil-stabilizer combination when evaluating the results.

The unconfined compressive strengths of the combinations of gradations and cement as stabilizer are presented in Figure 3.14. The amount of cement needed to reach 300 psi is highly dependent on the gradation. The Avg. 247 and Excess Fines blends require about 4% cement. The blend with Excess Sand and Fines required about 6% cement to achieve the strength of 300 psi, whereas the blend with excess sand requires more than 6% cement.

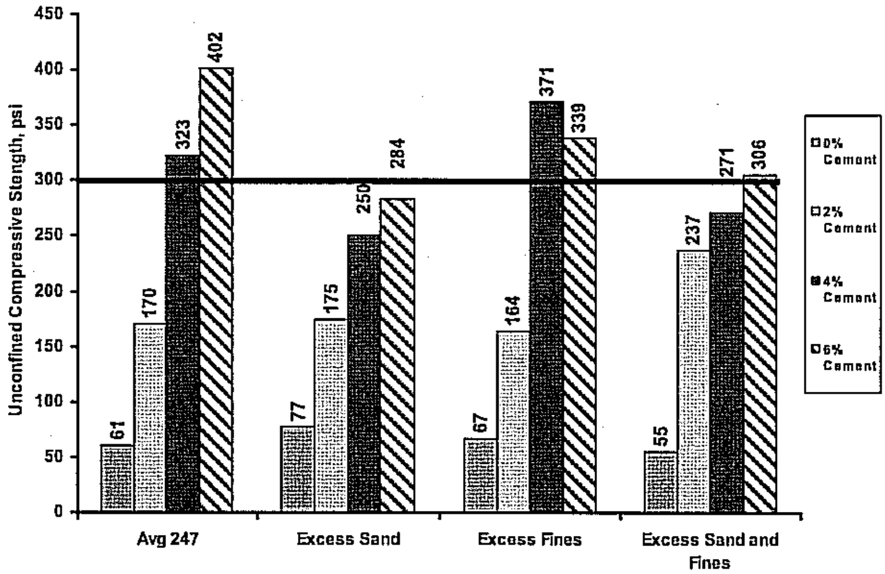


Figure 3.14 - Unconfined Compressive Strengths of Base with Cement

None of the specimens prepared with lime achieved the desired 150 psi strength as shown in Figure 3.15a. The maximum strength achieved was about 100 psi. The blend with the Excess Sand and Fines reacted most favorably with lime. At higher lime contents the strengths were lower indicating that perhaps the lime is acting as filler as opposed to stabilizer.

The fly ash also does not seem to be a compatible stabilizer with the El Paso base material (see Figure 3.15b). The maximum unconfined compressive strength achieved was less than 80 psi. One of the reasons for such low strengths as compared to cement-stabilized mixes can be in the method of curing. The ten-day of capillary saturation may negatively impact the results. To study the impact of the exclusion of capillary saturation on strengths, another series of unconfined compressive tests were carried out on specimens that were cured as per TxDOT specifications but without 10-day capillary saturation. The results are included in Figures 3.15c and 3.15d. These results are deemed more compatible with the results obtained for blends with cement. The unconditioned strengths are significantly higher than the moisture-conditioned ones for both lime and fly ash. With further investigation, perhaps the unconditioned strengths can be used as is done for cement.

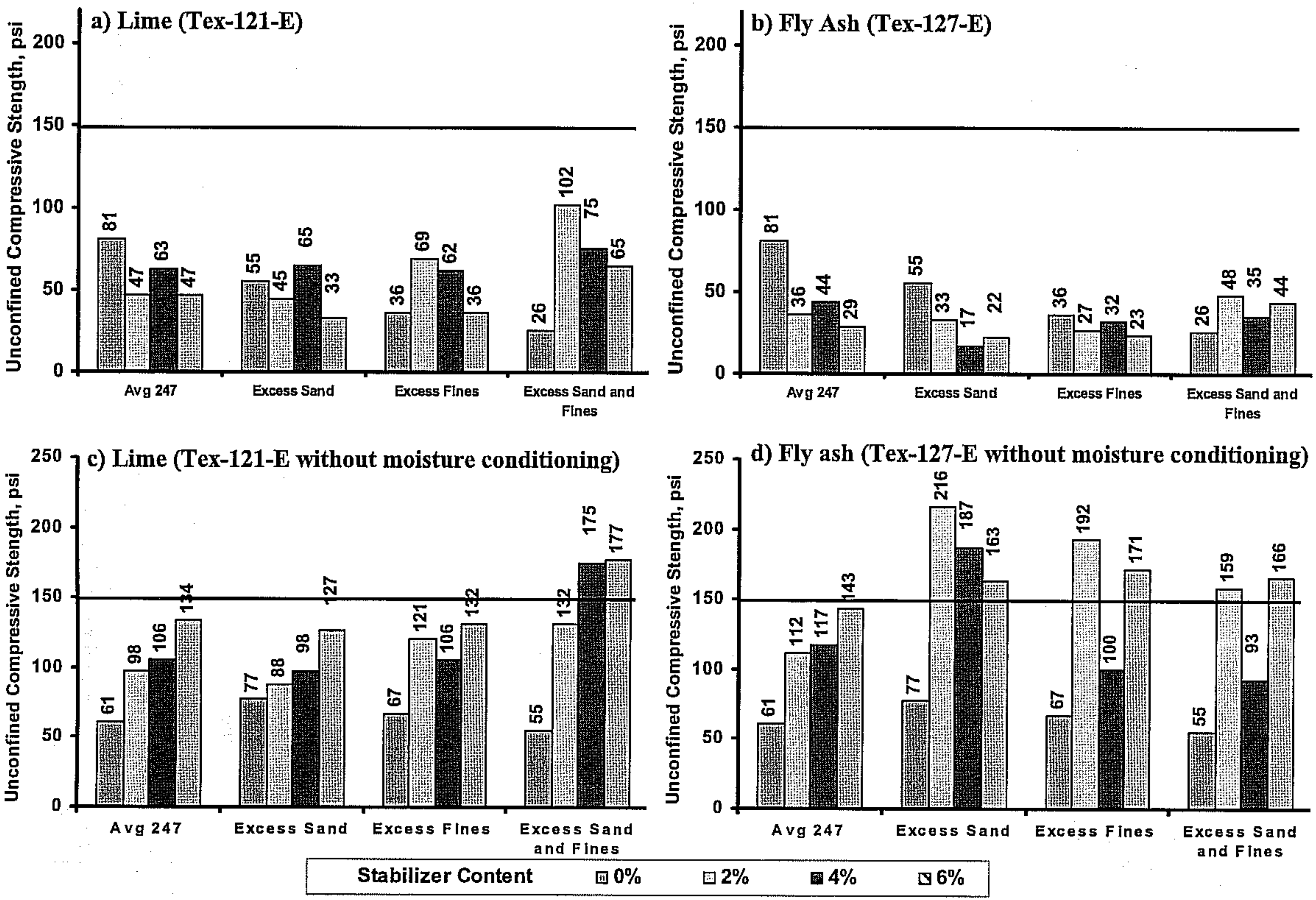


Figure 3.15 - Unconfined Compressive Strength of Soil-Lime and Soil-Fly Ash Specimens

Retained Strength

The retained strength concept corresponds to the strength of the mixtures after being subjected to water saturation. As indicated above, the current TxDOT protocols for obtaining the strength already provides the retained strength. For the cement-stabilized specimens, the retained strengths were obtained by preparing two sets of specimens and subjecting them to different methods of saturation. One set of specimens were subjected to ten days of capillary moisture saturation and the other set were submerged in water for four hours. The specimens were then subjected to unconfined compressive strength tests.

The retained strength ratio (RSR) was determined using:

$$\mathrm{RSR} = \frac{\text{Compressive Strength after Moisture Conditioning}}{\text{Compressive Strength without Moisture Conditioning}} \tag{3.2}$$

A retained strength ratio of above 85% is usually considered desirable for stabilized mixtures.

The retained strength ratios from the 10 day capillary conditioned specimens prepared with cement are shown in Figure 3.16a. For almost all cases, the RSR's are greater than 85%. The EF blend with 4% cement only achieved an RSR of 77%. Aside from experimental errors, the reason for this matter is unknown. The RSR for the ESF blend with 2% cement is 65%, perhaps because of the low amount of stabilizer and high amount of fines.

The retained strength ratios for specimens that were moisture-conditioned with 4 hour soaking are shown in Figure 3.16b. In this case, the patterns are somewhat different than the previous moisture conditioning. Even though most mixtures obtained retained strength ratios above 85%, the variability between the specimens with the same aggregate blends but different cement contents oscillated more. One observation was that in some cases, the materials at the edge of the specimens become dissolved in the water.

A comparison between the retained strength ratios of the 10 day capillary moisture conditioned specimens and the 4 hour soaked specimens is made in Figure 3.17. A relationship between the results from the two methods cannot be developed. However, it appears that the 10 day capillary condition procedure is a more robust testing method and more conservative than the 4-hour soaking, for mixes with 4% or more cement content. For most mixtures with 2% additives, the 4 hour soak seems to be harsher moisture-conditioning than 10 day capillary soak.

Similarly, the RSR was determined for lime and fly ash mixes. In order to determine the RSR for the lime and fly ash specimens, the strength with and without moisture conditioning as shown in Figure 3.15 were used. The RSRs are presented in Figure 3.18. Moisture conditioning with 4-hr soak was not possible because most specimens would disintegrate in the water.

In Figure 3.18a, the RSRs for the lime-treated specimens fail to reach the 85%. The maximum RSR was about 77%. The fly ash specimens (Figure 3.18b) yielded substantially smaller RSRs with a maximum RSR of 38%.

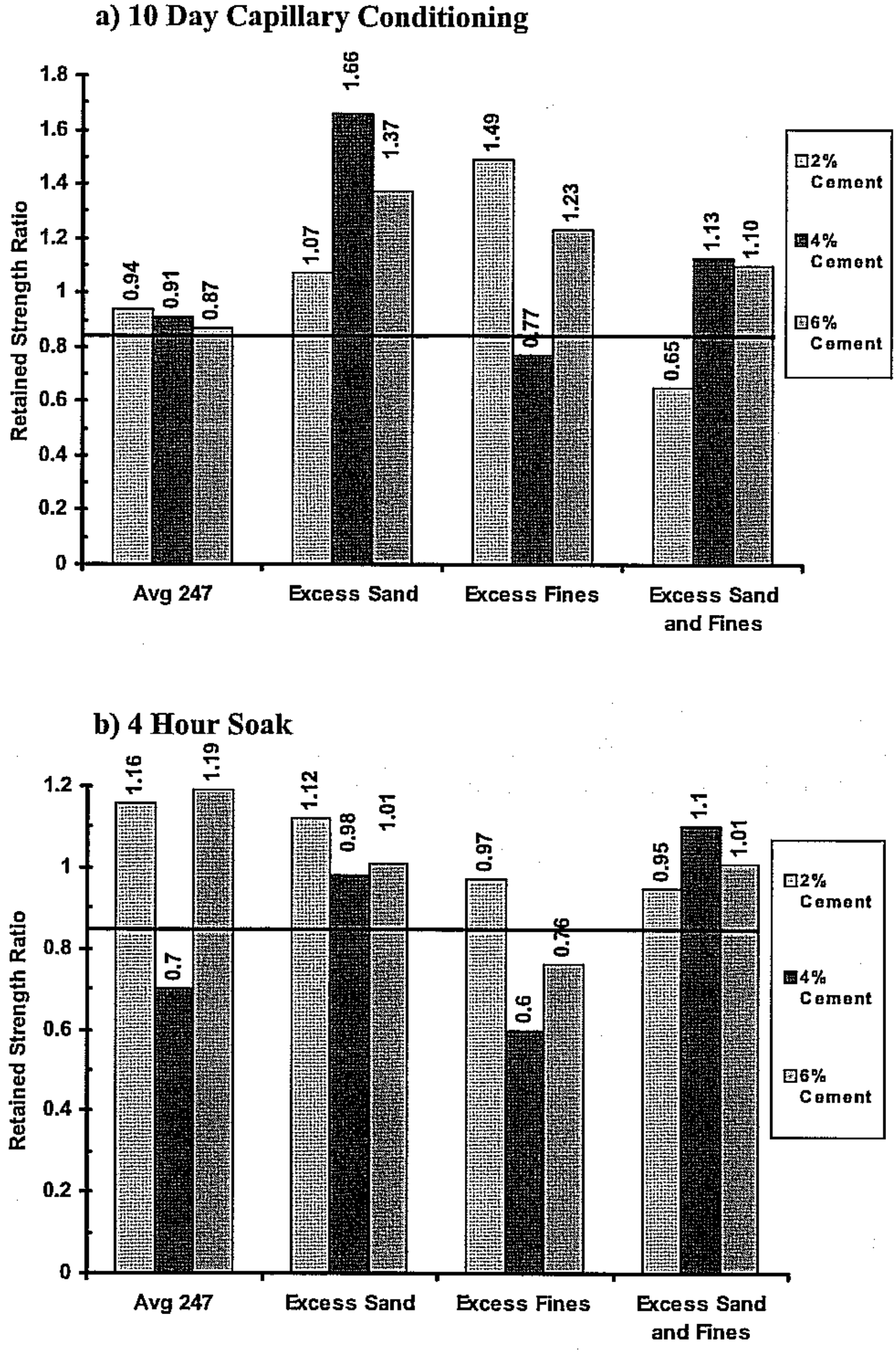


Figure 3.16 - Retained Strength Ratios for Capillary Moisture Conditioning and 4-Hour Soak for Specimens Stabilized with Cement

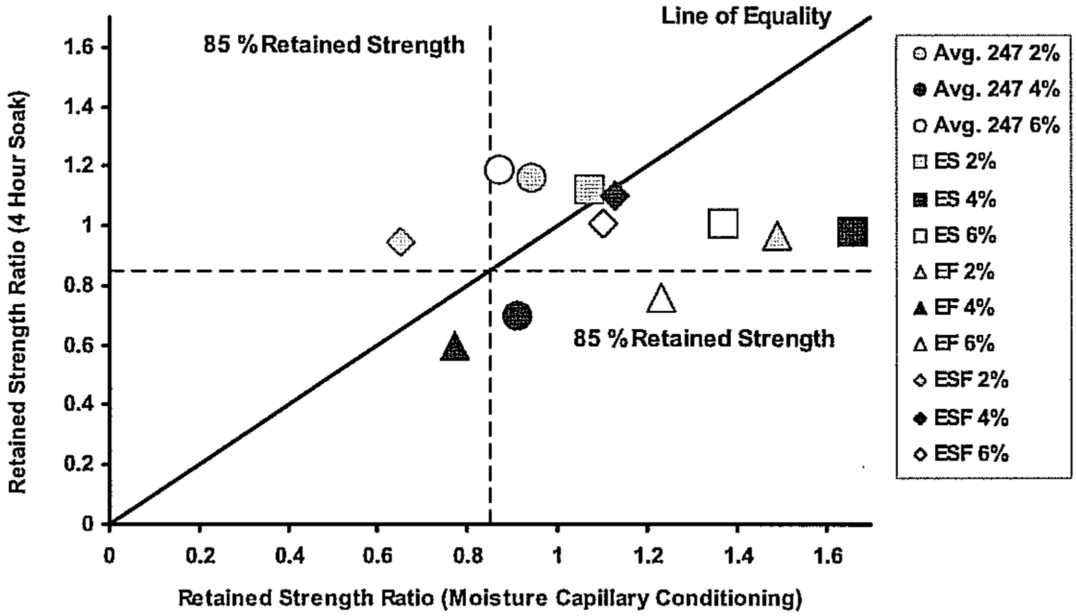


Figure 3.17 - Retained Strength Ratio Comparison Between 10 Day Capillary Conditioned and 7 Day-4 Hour Soak Specimens

Modulus

Aside from strength, the modulus of each mix should be ideally determined. Modulus is one of the most important parameters considered in the structural design of flexible pavements. The traditional method of determining modulus is to perform resilient modulus or repeated-load triaxial tests. Hilbrich and Scullion (2007) at the Texas Transportation Institute have shown that for stabilized materials, the seismic moduli determined with the FFRC device are the most convenient and precise values as compared to the resilient modulus or repeated-load triaxial methods. As such, only the seismic moduli of the specimens were measured. A minimum seismic modulus of 1,000 ksi was perceived reasonable for soil-cement mixes. For lime or fly ash treated soils, a minimum modulus of 500 ksi was selected.

The variations in seismic moduli with the gradation and cement content are shown in Figure 3.19a. The results are quite consistent with those from UCS tests shown in Figure 3.14. For the Avg 247 and EF blends, the required cement to achieve a modulus of 1000 ksi is 4%. The ES blend did not achieve the 1,000 ksi modulus with any percentage of cement, while the ESF blend requires about 6% to achieve a modulus of 1,000 ksi.

Seismic moduli for the lime-stabilized blends are shown in Figure 3.19b and for fly ash stabilized blends in 3.19c. The modulus requirement for these two stabilizers is 500 ksi. None of the gradation-stabilizer combinations met the desired modulus. For the lime-stabilized materials, the maximum modulus was about 300 ksi and for the fly ash specimens less than 100 ksi.

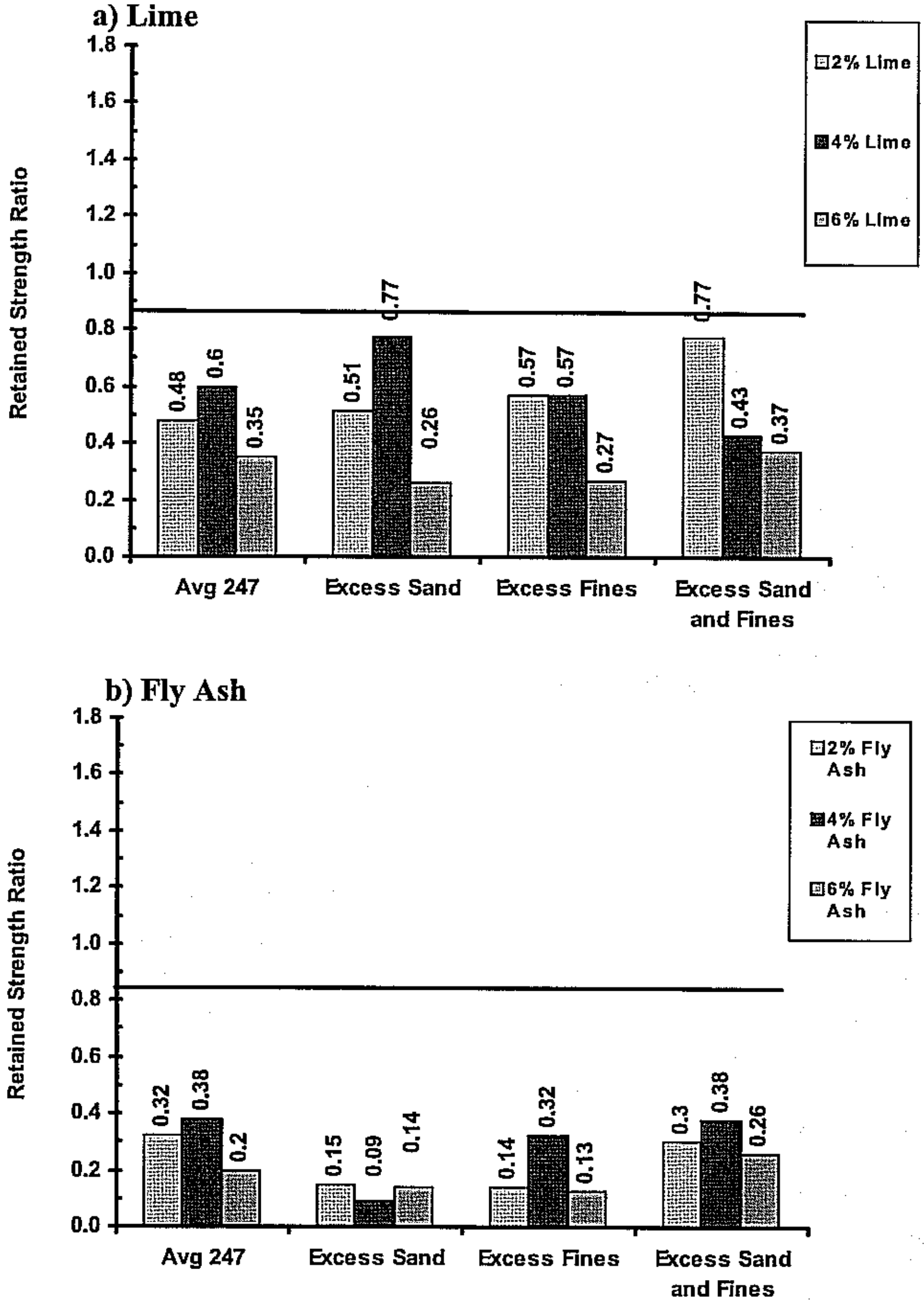


Figure 3.18 - Retained Strength Ratios for Capillary Moisture Conditioning

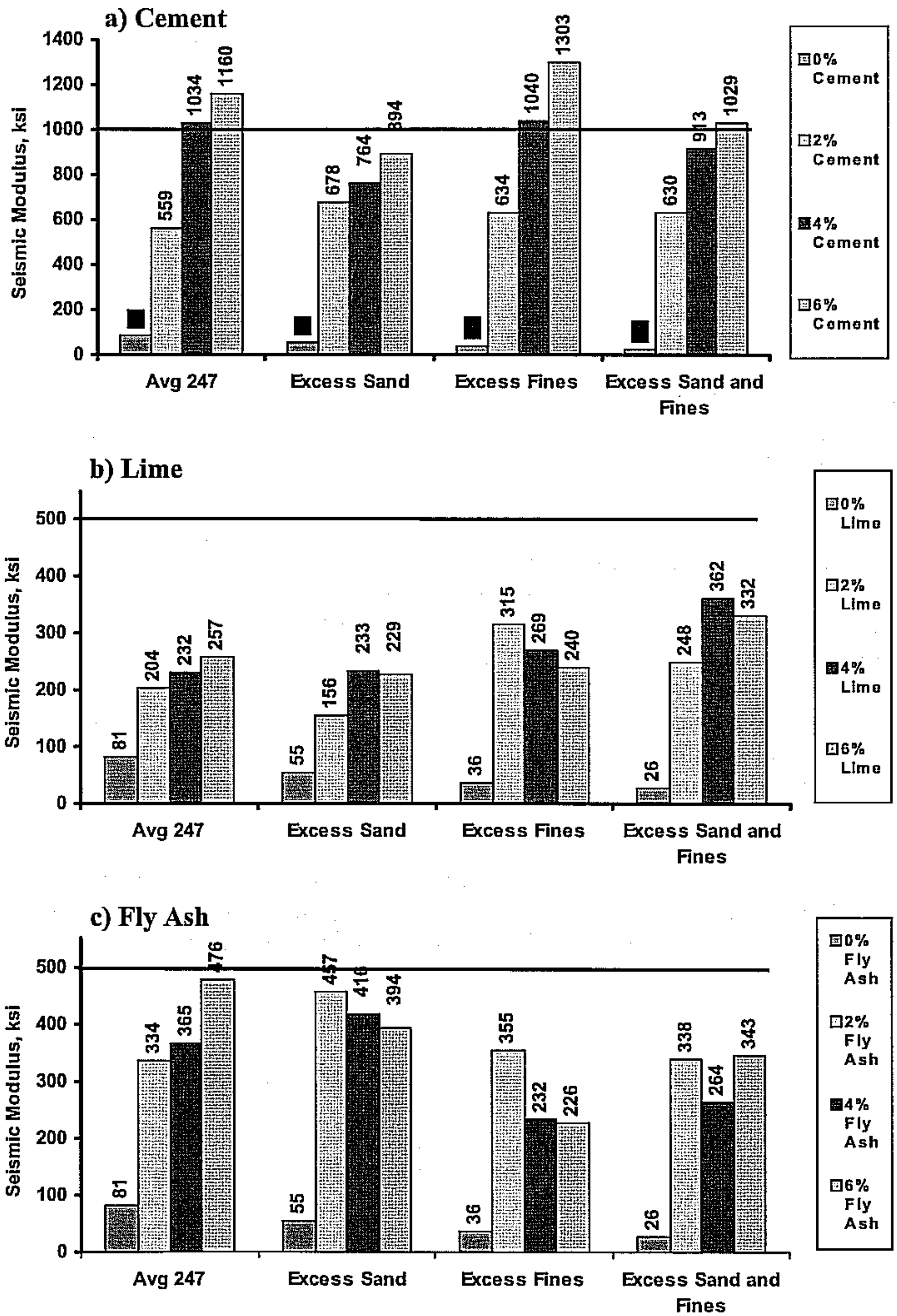


Figure 3.19 - Seismic Modulus vs. Stabilizer Content for TxDOT Procedures

Retained Modulus

The retained modulus idea follows the same requirements set by the retained strength ratio. For the cement stabilized specimens, the SM of the ten day capillary conditioned and four-hour soak are compared to the SM of the seven-day-cured (Tex-120-E) specimens. The retained modulus ratio (RMR) is determined by:

$$RMR = \frac{\text{Seismic Modulus after Moisture Conditioning}}{\text{Seismic Modulus without Moisture Conditioning}} \tag{3.3}$$

The RMR for the ten day capillary conditioned and seven day-four hour soak specimens with cement are presented in Figure 3.20a. The RMRs of almost all the blends with cement exceed the 85% value with two exceptions. The ES blend with 2% cement and EF with 4% cement achieved RMRs of 70% and 84%, respectively. With the four-hour soak, majority of the mixes exhibited RMRs of significantly less than 85% (Figure 3.20b). This, once again, demonstrates that in general the four-hour soak moisture conditioning is harsher than the 10-day capillary saturation.

Figure 3.21 presents the comparison of RSR and RMR for all cement blends for the 10-day capillary conditioned specimens and the 4-hour soaked specimens. For the 10-capillary saturation, all the specimens that pass the RSR criterion of 85% also pass the RMR criterion of 85% (Figure 3.21a) except one outlier. In majority of cases, the RMR is more conservative indicator of moisture susceptibility as well. Similar trends are observed for 4-hour soak. As such, the RMR can be perhaps used as a screening tool for loss of strength/stiffness as opposed to RSR. The practical benefit of this suggestion is that the number of specimens required will be halved since the same specimen can be tested before and after moisture conditioning.

Figure 3.22 present the RMRs for the capillary moisture conditioned blends for lime and fly ash mixes. For the lime specimens, none of the mixes achieved an RMR of 85% (Figure 3.22a). The maximum RMR is about 0.80 in three cases. For the fly ash-stabilized blends, the RMRs are significantly less than 85% as well. The highest RMR in this case is about 25%.

Moisture Susceptibility

Moisture susceptibility, the affinity of mixtures to absorb water, of the materials has been the subject of attention for the last few years. The reason for evaluating the moisture susceptibility of the mix is to ensure the long term durability and strength/stiffness of the materials when exposed to moisture. Two procedures have been advocated for evaluating the moisture susceptibility of mixes: one based on change in the dielectric properties of the mix (Tube Suction Test) and another based on the change in the modulus of the mix (using FFRC tests). These two processes were implemented on all blends and additive contents as discussed below.

To assess the moisture susceptibility of a specimen with both methods, the specimen is prepared at the optimum moisture content. Upon extraction, the specimen properties such as weight, height, and moisture content are recorded. The specimen is then placed in a 105°F (40°C) oven for two days then placed on porous stones in a water bath for eight days. Over the course of

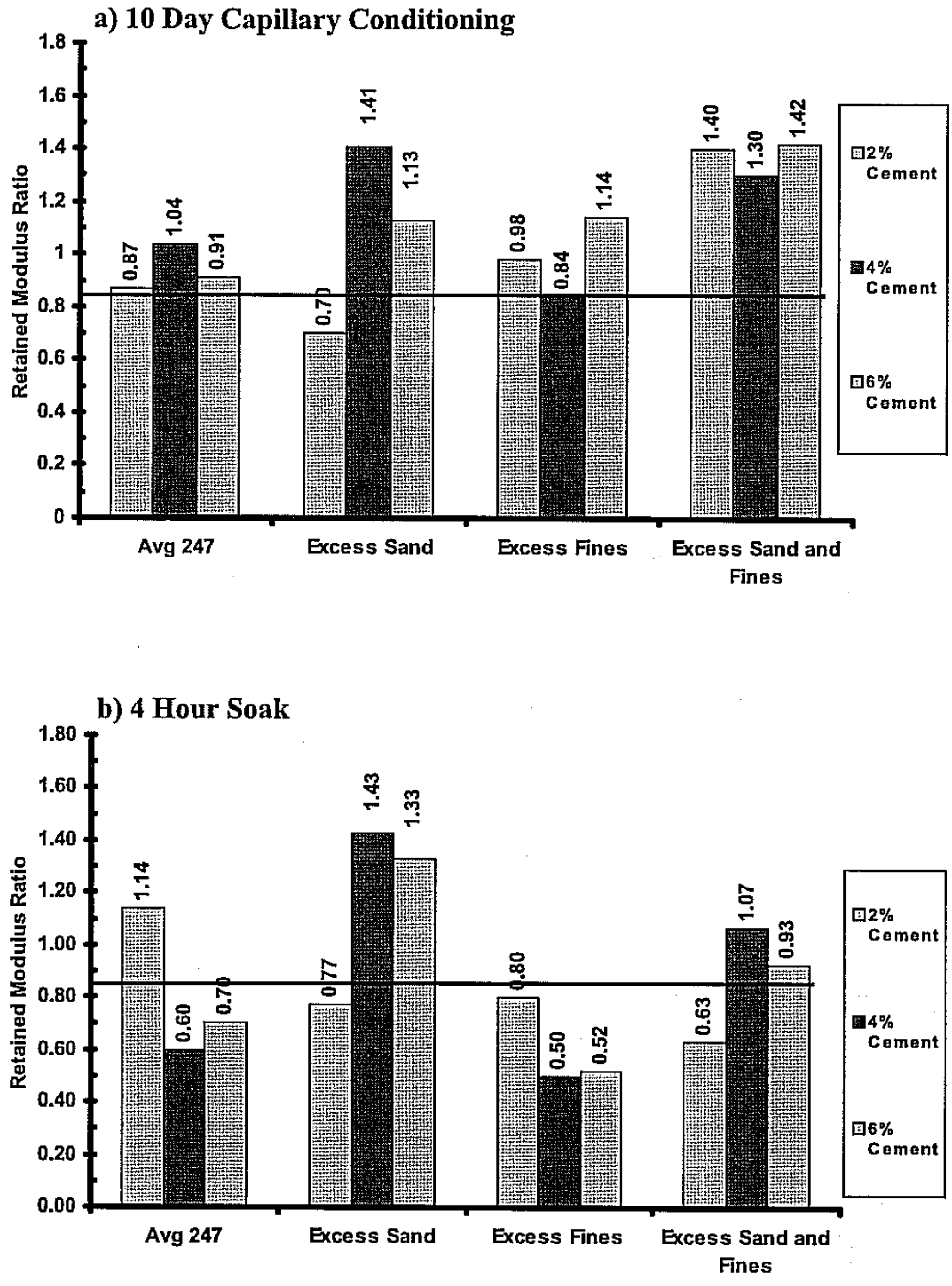
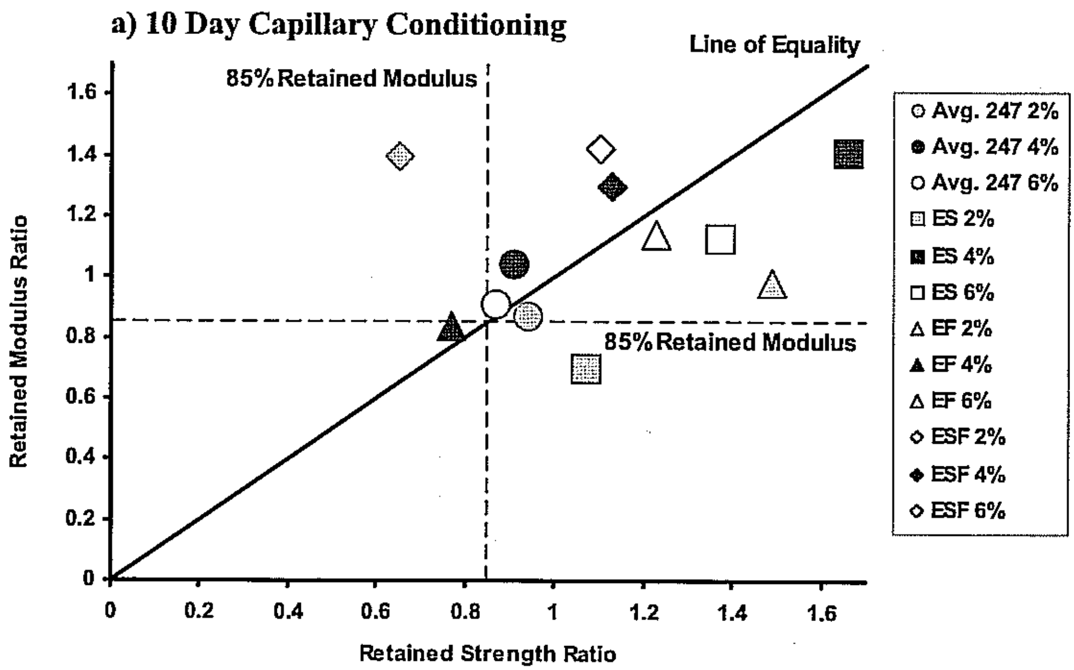


Figure 3.20 - Retained Modulus Ratios for Capillary Moisture Conditioning and 4-Hour Soak



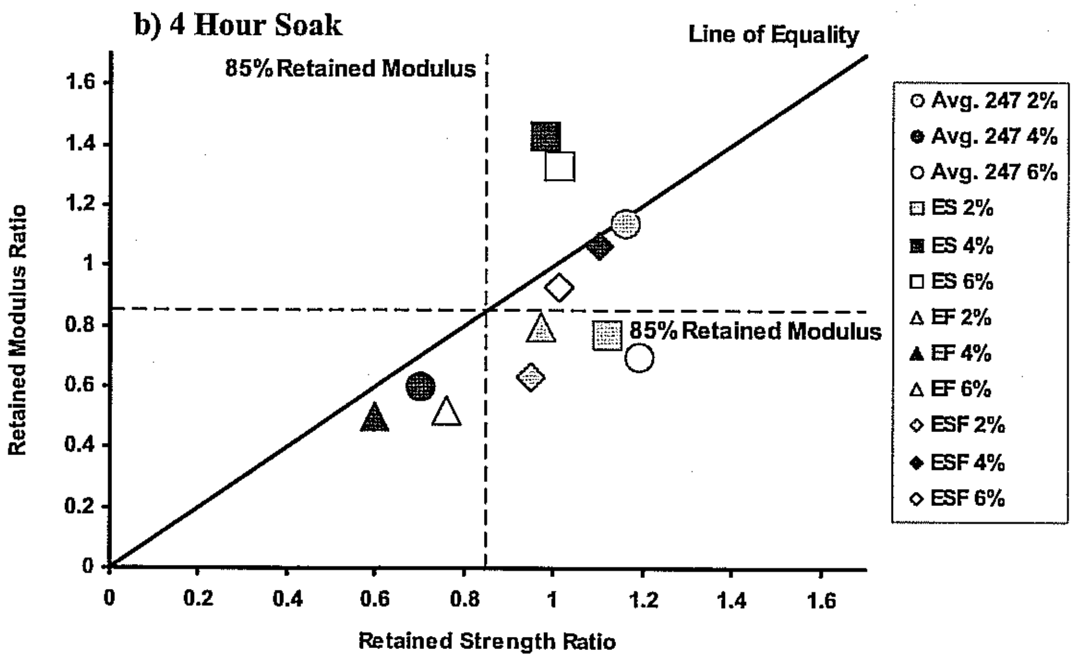


Figure 3.21 - Comparison between Retained Strength Ratio and Retained Modulus Ratio for Cement Specimens

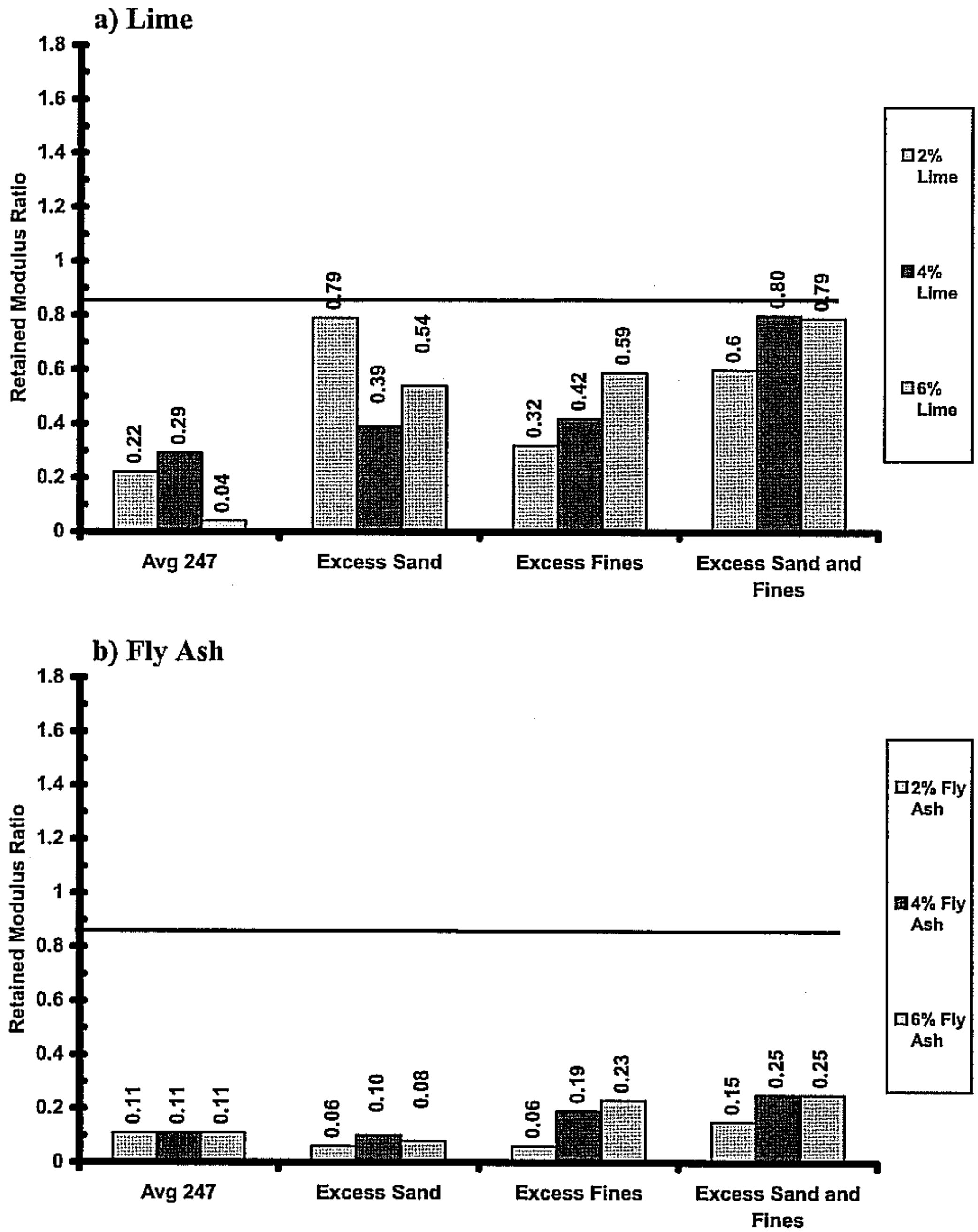
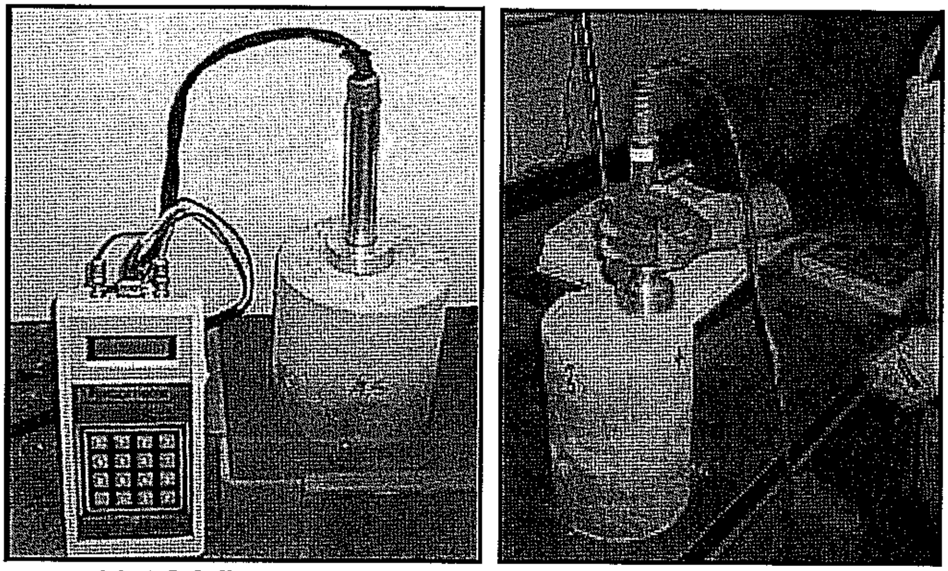


Figure 3.22 - Retained Modulus Ratios for Capillary Moisture Conditioning

these ten days, the FFRC test is performed each day. The average dielectric constant value of the specimen is also determined by taking five readings around the top of the specimen with an Adek Percometer™ (see Figure 3.23). In addition, the specimen is weighed every day to determine the variation in bulk moisture content with time.

Typical variations in dielectric constant, bulk moisture content and seismic modulus on one specimen are shown in Figure 3.24. As the specimen dries for the first 48 hours, the seismic



(a) Adek PercometerTM (b) Taking Dielectric Reading

Figure 3.23 - Utilization of Adek Percometer™

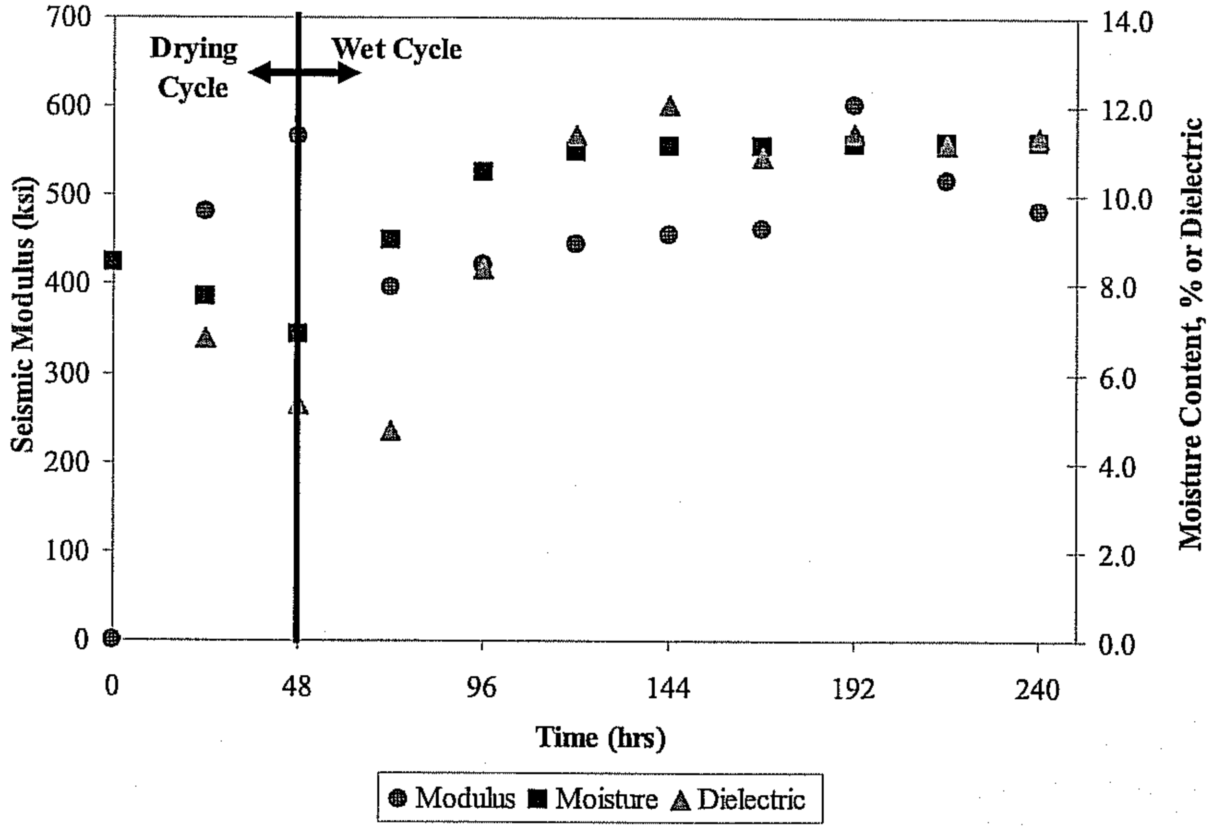


Figure 3.24 – Typical Variations in Moisture Content, Seismic Modulus and Dielectric Constant with Time for Moisture Susceptibility Specimen

modulus increases, while the bulk moisture content and dielectric constant decrease. Upon introduction of water after 48 hours, the seismic modulus decreases first, followed by gradual increase. This eventual gradual increase in stiffness can be attributed to the reaction of the stabilizer with the additional moisture provided during soaking. The bulk moisture content gradually increases as the moisture introduced, but becomes constant after three or four days of soaking. The dielectric constant also increases with the introduction of moisture to the specimen. A time lag between the increase in the moisture and dielectric constant during the first 48 hours of wetting is observed. The dielectric probe is influenced by the change in moisture of the top portion (about 2 in.) of the specimen. It seems that in the first 48 hours of capillary saturation, the moisture content of the top portion of the specimen does not change, even though the specimens absorb moisture.

The results of the dielectric measurements after 10 days of moisture conditioning for different gradations and stabilizer contents are presented in Figures 3.25. Scullion et al. (2003) propose a dielectric value of less than 10 for a material that is not moisture susceptible. For the raw blends (with 0% stabilizer), the dielectric constants are fairly close and less than 10. An unexpected trend is that the two blends with excess fines (EF and ESF) yielded lower dielectric constants than the two coarser blends (Avg. 247 and ES).

For the Avg. 247 blend (Figure 3.25a), the mixes with the 2% and 4% cement provide dielectric constants that are greater than 10, while the mix with 6% cement yield a dielectric value of about 6. The ES and EF blends yield small dielectric constants for all three cement concentrations. However, three mixes with the ESF blend yield values on the order of 10. A comparison of these values may indicate that perhaps a study on the impact of the chemical interaction of the fine and coarse aggregates and cement may be desirable for setting the acceptance level for dielectric values. As shown in Figures 3.25b and 3.25c, the dielectric constants for the lime and fly ash blends are either less or almost equal to 10.

The initial (molding) and final (after completion of moisture conditioning) moisture contents for all blends are shown in Figure 3.26. For cement mixtures, the final moisture contents are similar or higher than the initial moisture content. This indicates that the specimens have significant affinity to moisture, which may have adverse effect on the durability of the mix. When lime was used as the additive, the final moisture contents are again greater than the corresponding molding moisture contents, bringing into question the long-term durability of these mixes. The fly ash mixes also demonstrate great affinity to absorbing moisture in excess of the optimum moisture contents. The results from the raw materials (no additives) in Figure 3.26 indicate that all four blends absorb more or equal moisture after capillary saturation as compared to the molding moisture content. As such, the four raw El Paso blends show strong affinity to the absorption of water. Further work is needed to ascertain whether the affinity of the stabilized mixes to water is directly to the raw material, or whether the additives increase the capillary forces within the mix.

The seismic moduli for the capillary conditioned specimens are presented in Figure 3.27. For the cement stabilized blends (Figure 3.27a), Avg. 247, ES, and ESF with 4% and 6% cement surpassed the 1,000 ksi requirement. The EF blend with 6% cement also exceeded the requirement. The lime and fly ash stabilized blends failed to reach the 500 ksi requirement as

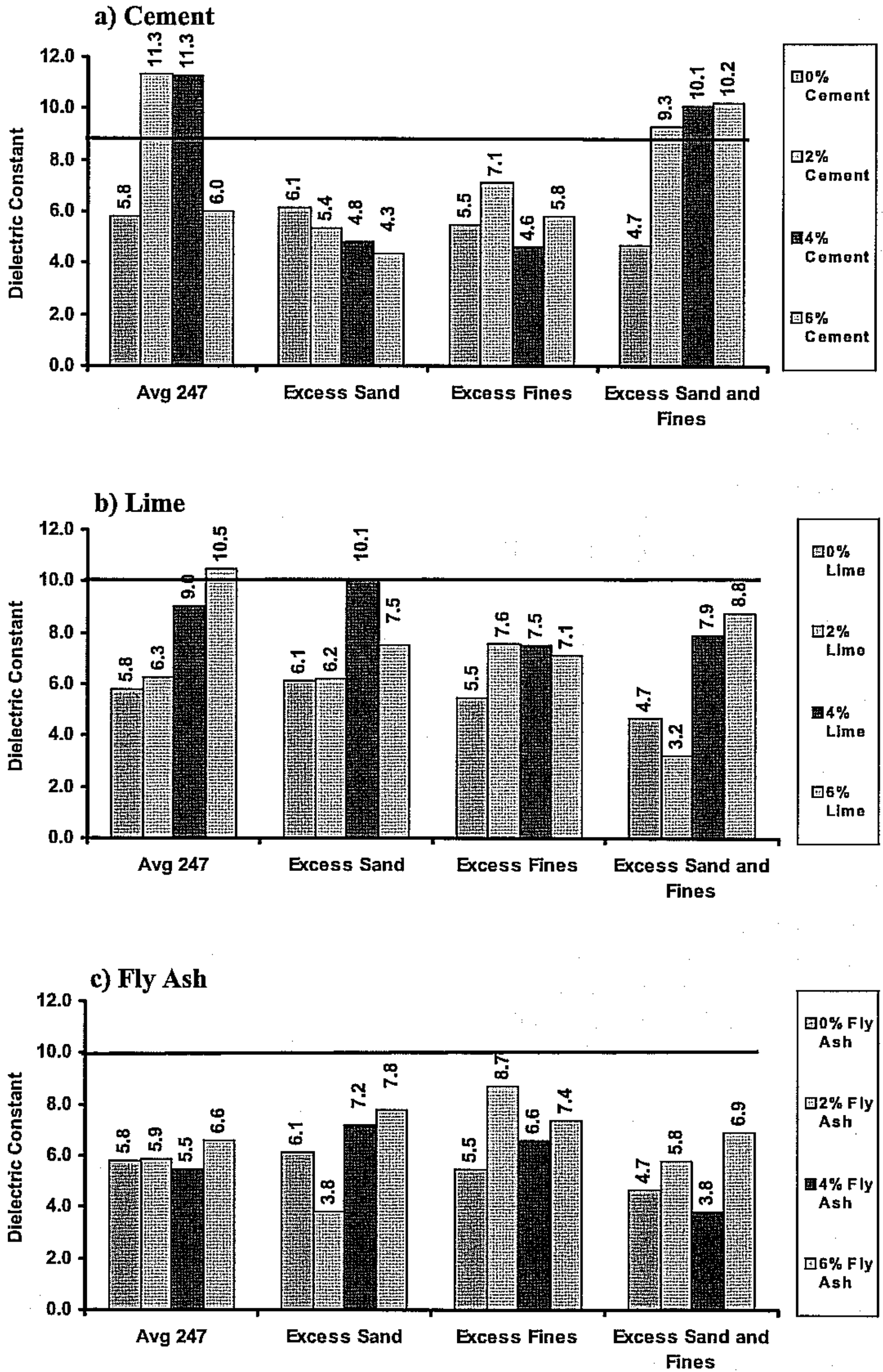


Figure 3.25 – Variations in Final Dielectric Values with Stabilizer Content for Moisture Susceptibility Specimens

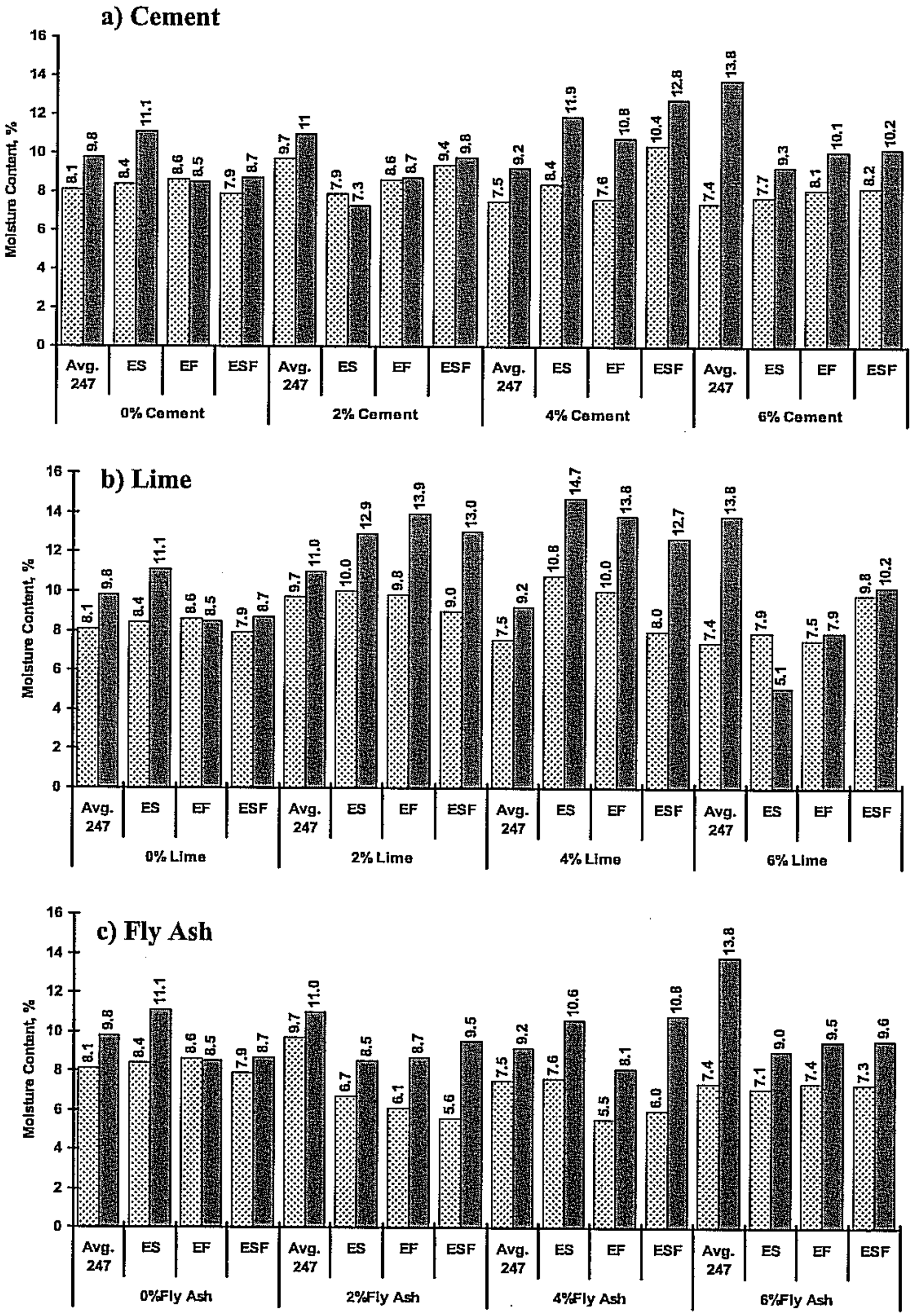


Figure 3.26 - Comparison of Initial and Final Moisture Contents from Moisture Susceptibility Specimens

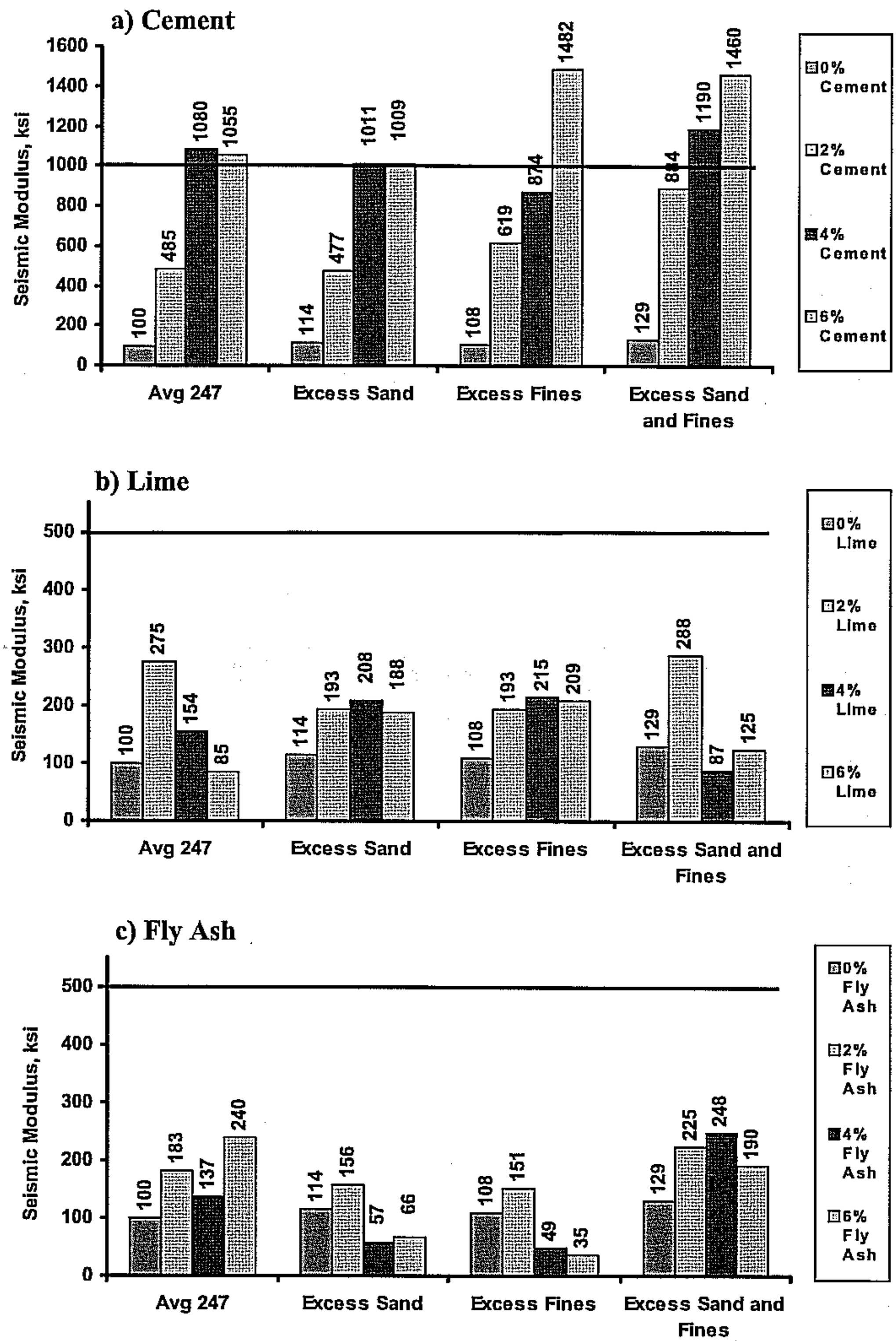


Figure 3.27 – Variation in Seismic Modulus with Stabilizer Content for Moisture Susceptibility Specimens

shown in Figures 3.27b and 3.27c. These low values may be due to the large final moisture contents.

Optimum Stabilizer Content

Based on the unconfined compressive strength, seismic modulus, and moisture susceptibility criteria the optimum stabilizer content can then be determined. Only the cement could provide strength and stiffness values that met the current TxDOT specification. The optimum cement contents for different blends based on UC strength are shown in Table 3.5. The Avg. 247 mix, considered as the control blend, requires about 4% cement, even though the mix would marginally fail the moisture susceptibility and the RSR with 4-hour soak. While the ES and EFS blends require 6% or more cement. On the other hand, the EF blend only requires 3% cement.

Table 3.5 – Selection of Optimum Stabilizer Content for Different Blends

Gradation	Optimum Cement Content, %	Retained Strength Ratio <10?		Dielectric Constant <10?	Seismic Modulus $\geq$ 1,000 ksi?	
		Capillary Saturation	4 Hour Soak		Capillary Saturation	4 Hour Soak
Avg. 247	4	Yes	No	No	Yes	No
Excess Sand	>6	Yes	Yes	Yes	Yes	Yes
Excess Fines	3	Yes	No	Yes	No	No
Excess Sand & Fines	6	Yes	Yes	No	Yes	No

Since the determination of the optimum cement content under the current TxDOT practice is carried out using materials retrieved from the site before pulverization, one can then assume that 4% cement (corresponding to the value obtained from the Avg. 247 blend) should be specified in this case. The implication of change in gradation on strength, stiffness and moisture susceptibility of the mix is summarized in Table 3.6. In that table, the values strength, stiffness and moisture susceptibility parameters associated with all four blends are included.

The ES blend yields a UC strength which about 23% less than the control mix. Similarly, the modulus of the ES blend with 4% cement is about 25% less than the Avg. 247 mix. However, the ES blend with 4% cement performs better under the moisture susceptibility criteria than the Avg. 247 mix. Therefore, if the pulverization increases the sand content of the base, the strength and stiffness will be compromised, but the long-term durability is improved.

The EF blend with 4% cement, on the other hand, provides a stronger (by 15%) mix with similar stiffness when compared with the Avg. 247 blend. The retained strength and modulus ratios are somewhat less favorable for the EF blend with 4% cement as compared to the Avg. 247 blend, but they may be still close to the acceptable levels.

The dielectric constant of the EF blend with 4% cement is significantly less than the Avg. 247 blend. However, as reflected in Figure 3.26, the EF mix absorbs more moisture after 10 days of saturation than the initial moisture content. This may support the lower values of the RSR and RMR as compared to the Avg. 247 mix. The dielectric constant can be lower for the EF blend simply because the moisture had not migrated to the top 2 in. of specimen after 10 days of moisture conditioning. Based on this analysis, if the pulverization process generates significant fines, the pavement may perform better than the mix using the original gradation. However, the long-term durability of the base may be of more concern.

Table 3.6 - Implication of Estimating Cement Content from Materials before Pulverization

Gradation	Unconfined Compressive Strength, psi	Seismic Modulus, ksi	Dielectric Constant	Retained Strength Ratio		Retained Modulus Ratio	
				Capillary Saturation	4 Hour Soak	Capillary Saturation	4 Hour Soak
Avg. 247	323	1034	*11.3*	0.91	*0.70*	1.04	*0.60*
Excess Sand	*250*	*764*	4.8	1.66	0.98	1.41	1.43
Excess Fines	371	1040	4.6	*0.77*	*0.60*	*0.84*	*0.50*
Excess Sand and Fines	*271*	*913*	*10.1*	1.13	1.10	1.30	1.07

For the ESF blend, once again, the strength and stiffness are less than those from the Avg. 247 by about 15%. However, the long-term durability as judged by the retained strength and modulus may be similar or better than the durability of the control blend.

Structural Evaluation

The implications of changes in the modulus of the stabilized base due to change in gradation are summarized in Table 3.6. The required changes in layer thicknesses in order to obtain similar performance from all blends were then studied. The performance of a pavement section is typically estimated based on the fatigue cracking and rutting. Figure 3.28 summarizes the critical stresses and strains that are used to quantify the fatigue cracking and rutting.

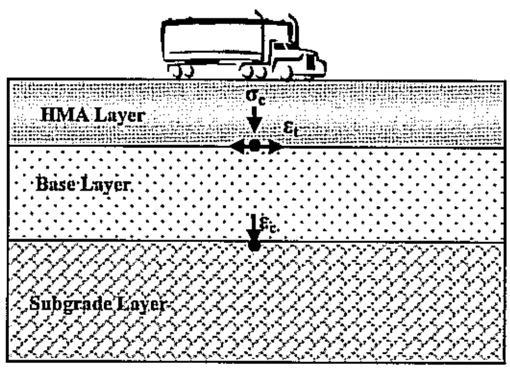


Figure 3.28 – Critical Stresses and Strains in a Three Layer Flexible Pavement System

Fatigue cracking is a function of the tangential strain at the bottom of the HMA layer, and can be estimated from (Huang, 1993):

$$N_f = f_1(\varepsilon_t)^{-f_2}(E_{HMA})^{-f_3} \tag{3.4}$$

where N_f is the allowable number of load repetitions to prevent fatigue cracking, ε_t is the tensile strain at the bottom of the HMA layer, E_{HMA} is the elastic modulus of the HMA layer, and parameters f_1 through f_3 are design constants. As summarized in Huang (1993), a number of different organizations have proposed different sets of values for parameters f_1 through f_3. To avoid the selection of one set of these values, it was assumed that two pavements constructed with the same HMA, will perform similarly as long as the tensile strains at the bottom of their corresponding HMA layers are similar.

In the cases where the stabilized layer is stiffer than the HMA or the HMA layer is not used, the tensile strains at the bottom of the stabilized layers from the two pavement sections should be similar to minimize cracking of the base.

Rutting may occur either in the subgrade or within the HMA layer. The performance of the pavement in terms of subgrade rutting can be estimated from (Huang, 1993):

$$N_d = f_4(\varepsilon_c)^{-f_5} \tag{3.5}$$

where N_d is the allowable number of load repetitions to prevent rutting, ε_c is the compressive strain at the top of subgrade and parameters f_4 and f_5 are design constants. Once again, two pavement sections will perform similarly as long as their compressive strains at the top of subgrade are similar.

Finn et al. (1984) recommend the following relationships for estimating the rutting in the HMA layers less than 6 in. thick:

$$\log RR = -5.617 + 4.343 \log w_0 - 0.167 \log(N_{18}) - 1.118 \log \sigma_c \tag{3.6}$$

If the HMA layer is equal to or greater than 6 in. in thickness:

$$\log RR = -1.173 + 0.717 \log w_0 - 0.658 \log(N_{18}) + 0.666 \log \sigma_c \tag{3.7}$$

where RR is the rate of rutting in micro-inches (1 μin. $=10^{-6}$ in.) per axle load repetition, w_o is the surface deflection in mil (1 mil$=10^{-3}$ in.), σ_c is the vertical compressive stress within the AC layer in psi, and N_{18} is the equivalent 18-kip single-axle load in 10^5 ESALS. In this case, the compressive stress within the HMA layer should be similar for two pavement structures to perform similarly.

To develop the structural equivalency for the bases with different gradations, two pavement sections shown in Table 3.7 were considered. A thin (2 in.) and a thick (5 in.) HMA layers were considered. A modulus of 500 ksi and a Poisson's ratio of 0.33 were assigned to the HMA. The thickness of the pulverized base was assumed to be 12 in. with a Poison's ratio of 0.35. The modulus of this layer varied based on the laboratory results obtained for the four blends with 4% cement. The subgrade was assumed to have a modulus of 10 ksi with a Poisson's ratio of 0.40.

The base modulus from the Avg. 247 blend with 4% cement was used as the control value, since it was presumed that that mixture would represent the material used for determining optimum cement content. The moduli from the other three blends (ES, EF and ESF) with 4% cement were then used to determine the equivalent layer thicknesses for similar performance to the control base.

The flow chart for the determination of the equivalent pavement sections is shown in Figure 3.29. The first step in the analysis was to determine the critical stresses and strains using a linear-elastic layered program (BISAR) for the two pavement sections specified in Table 3.7. The FFRC modulus for the Avg. 247 blend with 4% cement (control mix) after 7 days of moisture curing was 1034 ksi. Since the TTI studied indicated that the resilient modulus of a stabilized layer is approximately equivalent to 70% of the seismic modulus, a modulus of 724 ksi was used in the analysis.

Table 3.7 – Pavement Layer Properties Used in Structural Analysis

Layer	Thickness, in.		Modulus, ksi	Poisson Ratio
AC	2	5	500	0.33
Base	12		Based on FFRC results	0.35
Subgrade	Semi Infinite		10	0.40

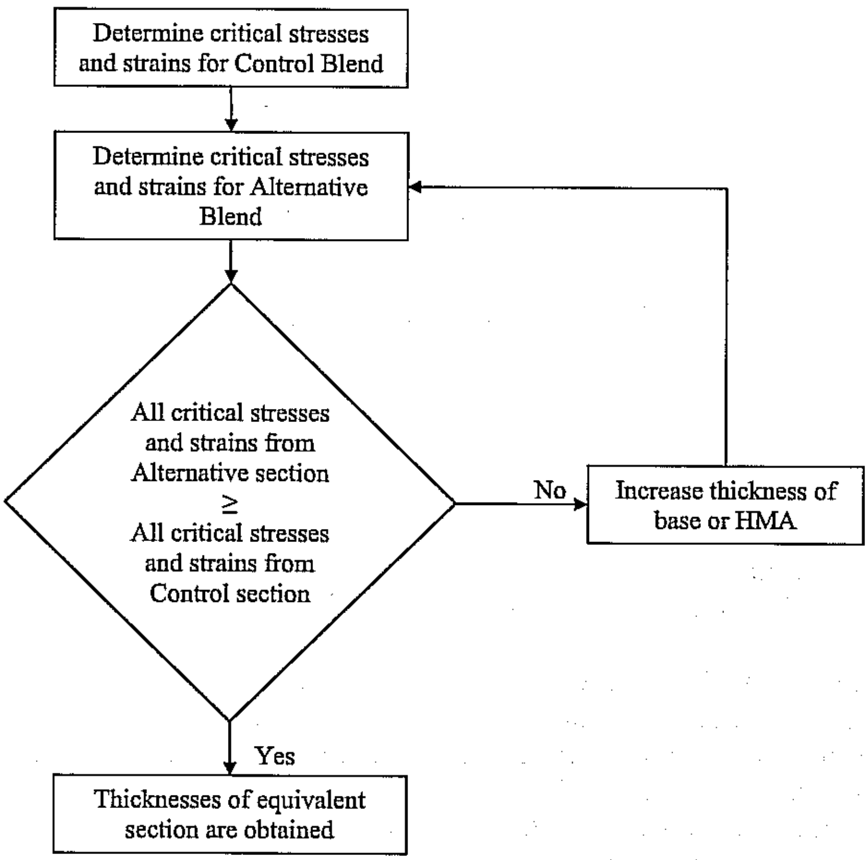


Figure 3.29 - Flow Chart for Determination of Equivalent Pavement Thicknesses

The tangential strain (ε_t) at the bottom of the HMA, compressive stress (σ_c) within the HMA layer and compressive strain (ε_c) at the top of the subgrade layer are reported in Table 3.8.

As an example, the modulus of the base was replaced with the corresponding resilient modulus from 4-hr moisture conditioned specimens. This exercise was carried out to determine the change in layer thicknesses when the base would become wet. The critical stresses and strains for this condition are also shown in Table 3.8. Since this modulus was less than the 7-day moisture-cured modulus, the stresses and strains are generally greater. The tensile strains at the bottom of the HMA when the moisture-conditioned modulus was used are about 40% greater than the control condition. Similarly, the compressive strains at the top of subgrade are about 30% greater. The compressive stresses within the HMA are more or less the same.

Table 3.8 – Critical Strains and Stresses for Avg. 247 with 4% Cement

Specimen Condition	**Micro strain**				**σ_c, psi**	
	ε_t		ε_c			
	2-in. HMA	**5-in. HMA**	**2-in. HMA**	**5-in. HMA**	**2-in. HMA**	**5-in. HMA**
7 day Moisture Cure (Control)	38	30	109	88	79	64
Moisture Conditioned (4 Hour soak)	54	41	145	113	79	60

To determine the equivalent thicknesses between the sections, an optimization algorithm was added to BISAR. The optimization algorithm automatically changes the thicknesses of the base and HMA until the critical stresses and strains from both sections are almost equal. Large combinations of layer thicknesses can be found with this algorithm that would satisfy the equality of the stresses and strains. To keep the program practical only the following two combinations were used: (1) either the thickness of the base was kept constant and the thickness of the HMA was changed until the critical stresses and strains became less than those from the control condition, or (2) the thickness of the HMA was maintained constant and the thickness of the base varied. For example, an HMA thickness of 6 in. (instead of 5 in.) was required so that the performance based on 4-hr soak modulus would be equivalent to the control section. Alternatively, the thickness of the HMA can be maintained as 5 in. and the thickness of the base can be increased to 16 in.

This process was repeated for the blends ES, EF, and ESF; that is the equivalent thicknesses for these three blends were determined so that they would perform similar to the control condition (Avg. 247 blend with 4% cement). The results when the 7-day cured moduli were used are included in Table 3.9a. For the ES blend, a substantial (2.5 in.) thickening of the HMA or a moderate (2 in. to 3 in.) thickening of base is required due to the weakening of the base due to pulverization. For the EF blend, the initial thicknesses are adequate. If the pulverization results in a mix similar to the ESF blend, the HMA thickness should be increased by 1.5 in., or the base layer thickness should be increased to 13 in.

The equivalent thicknesses when the moisture-conditioned moduli are used, instead of the 7-day cured moduli, are presented in Tables 3.9b and 3.9c. Somewhat different results are obtained.

The results from Phase II, where the lab and field moduli are compared, would be needed to determine which sets of moduli are appropriate for consideration.

Table 3.9 – Layer Thicknesses Required for Equivalent Performance when 7-day Cured Laboratory Moduli Used

a) Based on 7-Day Cured Moduli

Blend	HMA Thickness (in.) when Base Thickness Maintained Constant		Base Thickness (in.) when HMA Thickness Maintained Constant	
	2-in. HMA	5-in. HMA	2-in. HMA	5-in. HMA
Excess Sand	4.5	7.5	14	15
Excess Fine	2.0	5.0	12	12
Excess Sand and Fines	3.5	6.5	13	13

b) Based on Capillary Saturated Moduli

Blend	HMA Thickness (in.) when Base Thickness Maintained Constant		Base Thickness (in.) when HMA Thickness Maintained Constant	
	2-in. HMA	5-in. HMA	2-in. HMA	5-in. HMA
Excess Sand	3.0	6.0	12.0	13.0
Excess Fine	4.0	7.0	12.0	14.0
Excess Sand and Fines	2.5	5.0	12.0	12.0

c) Based on 4-hr Soak Moduli

Blend	HMA Thickness (in.) when Base Thickness Maintained Constant		Base Thickness (in.) when HMA Thickness Maintained Constant	
	2-in. HMA	5-in. HMA	2-in. HMA	5-in. HMA
Excess Sand	2.5	5.0	12.0	12.0
Excess Fine	3.5	6.5	12.0	14.0
Excess Sand and Fines	2.5	5.0	12.0	12.0

Chapter 4

Evaluation of Base Materials from Odessa

Introduction

The test protocol outlined in Chapter 3 (Figure 3.1) was utilized to evaluate a base from a project in Odessa District. This site was a section of Interstate Highway (IH) 20 in Ward County. The construction of this project consisted mainly of excavating and discarding the old asphalt concrete pavement (ACP), reclaiming and cement-treating the in-place base down to 6 in., paving the finished based with hot-mix asphalt (HMA) and placing a rubber underseal (see Figure 4.1). This site located on the westbound lane from Station 1602+40 to Station 1554+00.

Base materials from the site were collected prior to and during construction (just after pulverization), and were subjected to a number of tests. In summary, testing of the base material consisted of the following four major steps:

1. Determining moisture density and moisture modulus relationships
2. Determining of strength and stiffness of raw materials
3. Determining the appropriate cement content using materials retrieved before pulverization
4. Comparing the strength and stiffness of pulverized materials with those obtained before pulverization

Additionally, nondestructive tests with the FWD and PSPA were performed on top of the new base to determine the modulus of the base after construction. The results from lab and field tests are presented in this chapter.

The pre-construction and post-construction pavement profiles are presented in Figure 4.2. Prior to construction, the existing base layer was about 18 in. thick. After the construction, the top 6 in. of the base was reclaimed and mixed with 2% cement. The asphalt concrete layer was replaced at its original thickness of 4.5 in.

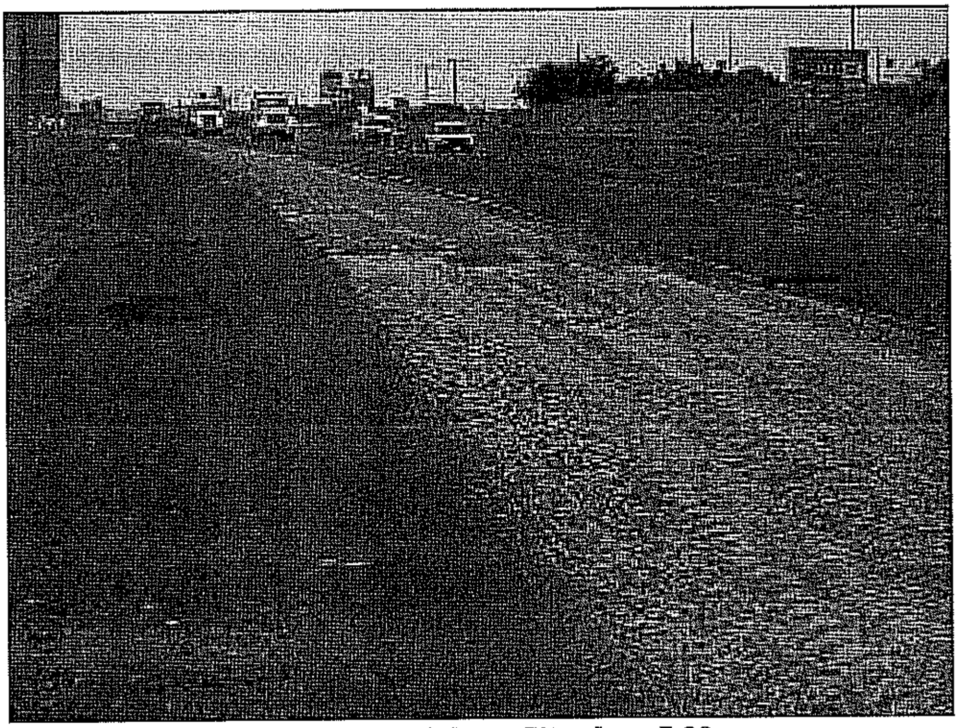


Figure 4.1 – Odessa Site along I-20

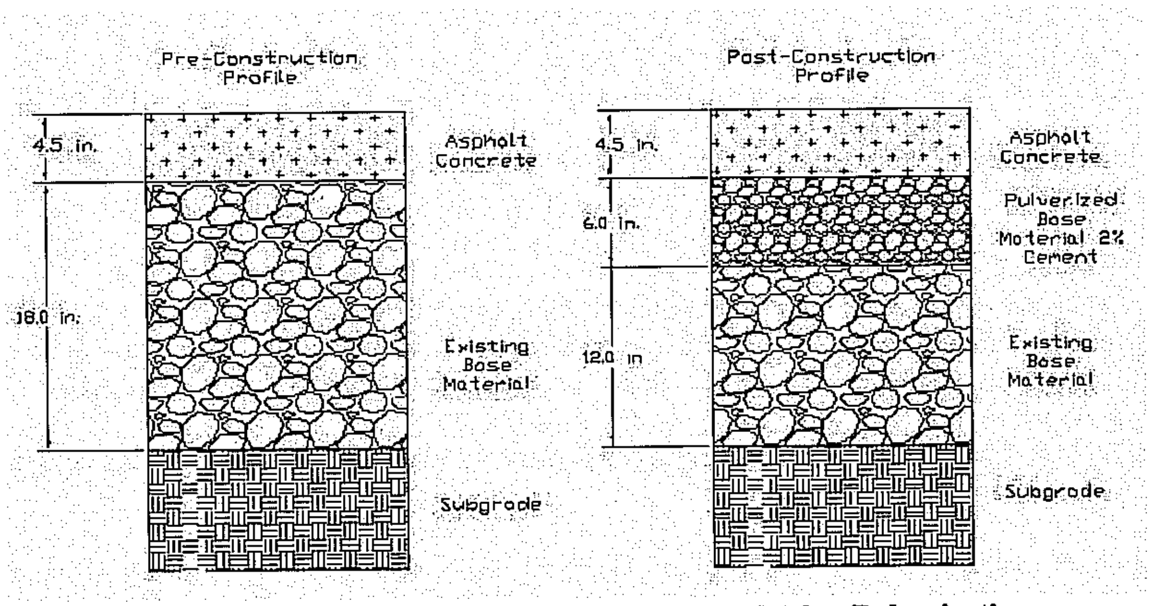
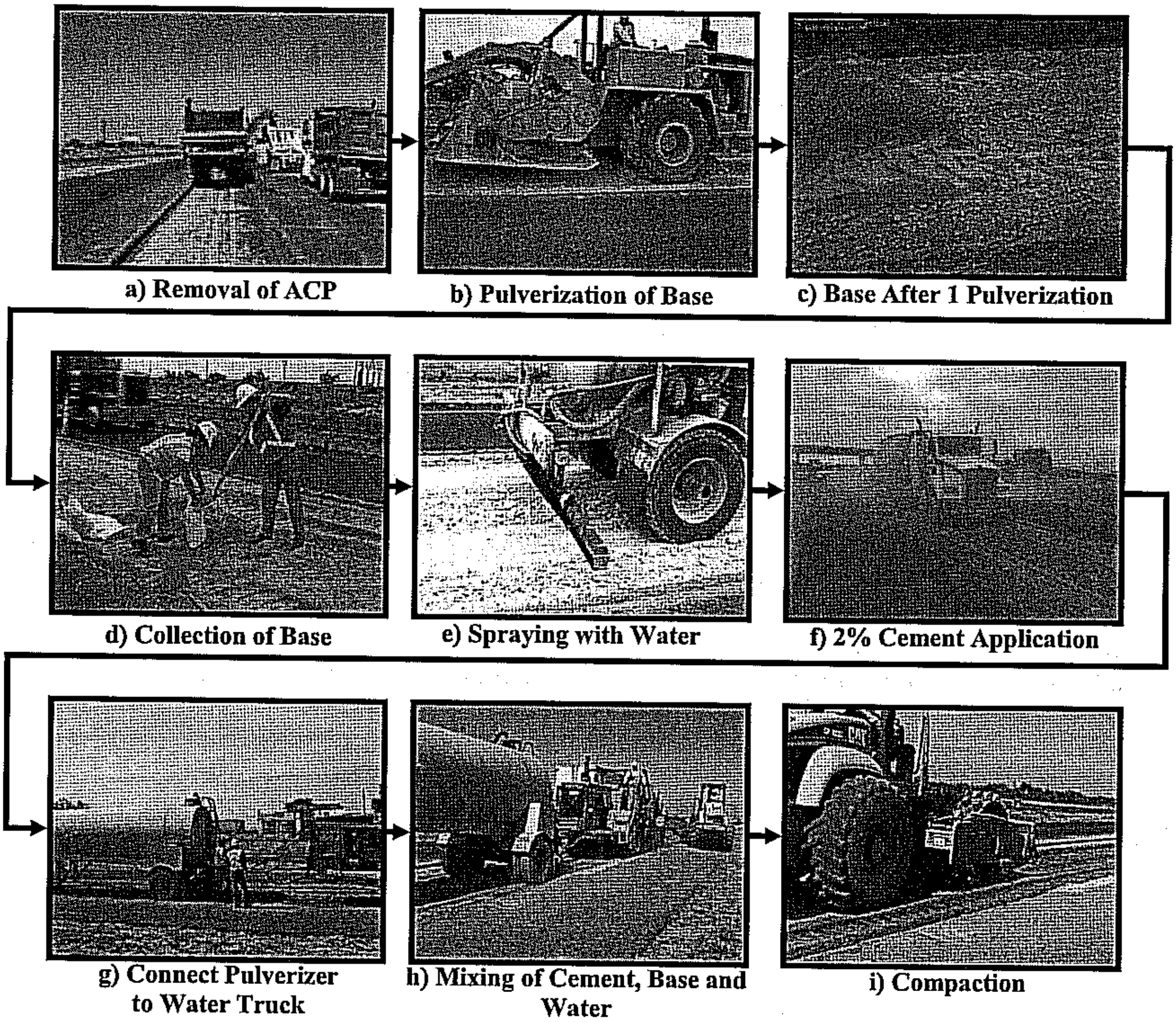


Figure 4.2 – Pavement Cross-section Before and After Pulverization

Construction Activities

The construction sequence is presented in Figure 4.3. The initial step was the removal of the existing asphalt concrete pavement which was hauled off site (Figure 4.3a). The top 6 in. of the base was reclaimed next with a pulverizer (Figure 4.3b). A grader/blade passed over the pulverized base to smooth and grade the surface. A water truck then lightly sprayed the surface in order for the cement to adhere to the base as it was being applied (Figure 4.3e). As shown in Figure 4.3f, the cement was applied to base material. The pulverizer, connected to a water truck, was used to mix the base material, cement, and water (Figure 4.3g and h). As soon as the mixing was completed, a sheep foot roller was utilized to compact the base.



a) Removal of ACP
b) Pulverization of Base
c) Base After 1 Pulverization
d) Collection of Base
e) Spraying with Water
f) 2% Cement Application
g) Connect Pulverizer to Water Truck
h) Mixing of Cement, Base and Water
i) Compaction

Figure 4.3 - Construction Procedure and Base Material Collection at Odessa Site

Testing Activities

Approximately 900 lb of raw base material was collected prior to construction from one location at the site and tested according to the protocol outlined in Chapter 3. This material will be referred to as Odessa Raw to distinguish it from the pulverized materials. The base material was also sampled just after pulverization (Figure 4.3d) which is referred to as Odessa Pulverized.

The collection of pulverized material was performed in the following way:

- Approximately 250 lb was collected at five locations at 1,000 ft intervals
- At one location, the old base was pulverized three times and about 250 lb of material was collected after each pass of pulverizer

Characterization of Raw Base Materials

The test protocol allows for the characterization of a base material by its gradation, strength and moisture susceptibility, etc.; and comparing these qualities to TxDOT construction specifications. The results for Odessa Raw are presented in this section. The results for Odessa Pulverized materials are then reported in the following sections.

Index Testing

The gradation of the raw base is compared to the minimum and maximum limits specified in Item 247 in Figure 4.4. The gradation for this base lies along or outside the maximum allowable limits. The gradation of the raw base is also presented in Table 4.1. The material resembles the Excess Sand (ES) blend studied in Chapter 3. The PI for the base was determined to be 4 as determined from test procedure Tex-107-E. The ACV of the base material was 32, and the dry and wet AIV were 19 and 24, respectively. According to the British standards, the Odessa aggregate can be considered an aggregate with a potential for degradation during construction.

The optimum moisture content and the maximum dry unit weight of the raw material were 9.6% and 124 pcf as shown in Figure 4.5. The seismic modulus at the OMC was 39 ksi.

Strength

The results from Texas Triaxial (Tex-117-E) and standard Triaxial (Tex-143-E) tests are presented in Table 4.2. Based on the gradation and the Texas triaxial classification of 2.9, the material can be classified as a Grade 3.

Verification of Optimum Cement Content

Three cement contents of 1.5%, 3% and 4.5% were considered to establish the optimum cement content. The optimum moisture contents, the maximum dry unit weights, and the seismic moduli at the OMC for the three cement contents are shown in Table 4.3. The OMC varied between 9.6% and 10.4%, indicating once again that the OMC for cement-stabilized mixes does not vary much with cement content. The maximum dry unit weight varied between 114 pcf and 124 pcf with the raw mix once again providing the highest density. The seismic moduli at OMC increased from 39 ksi for the raw material to a minimum of 141 ksi for stabilized materials.

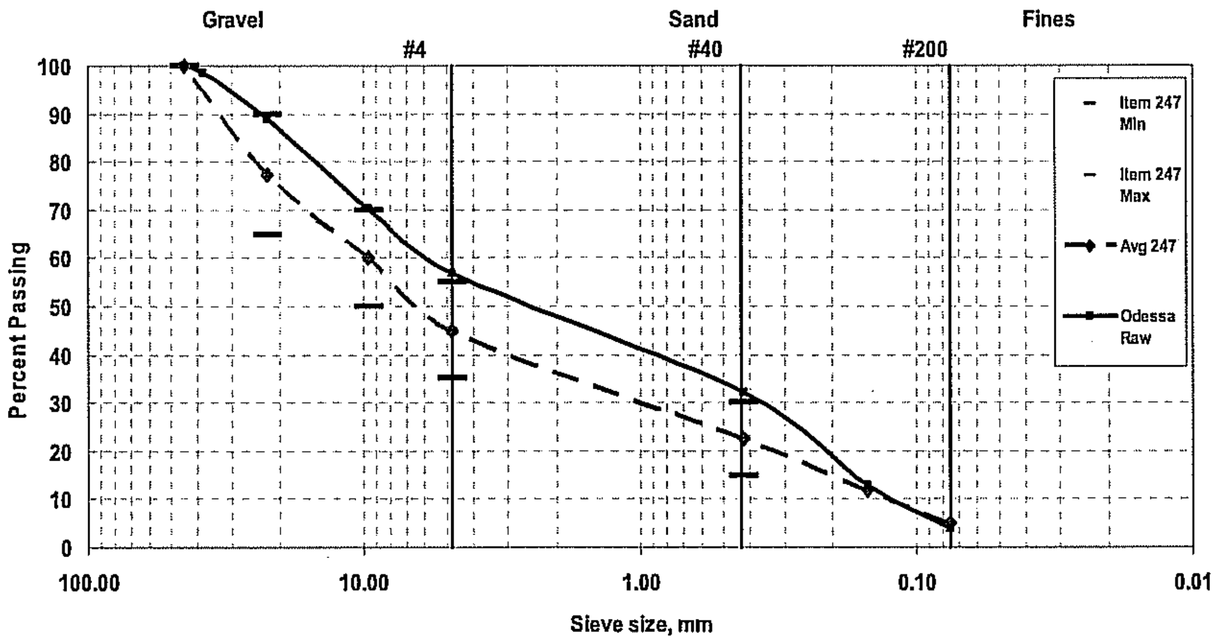


Figure 4.4 - Gradation for Odessa Raw Base Material Compared to Item 247 Limits

Table 4.1 - Gradation of Odessa Raw Material

Sieve Size	Percent Passing	Item 247 Min.	Item 247 Max.
1¾ in.	100	100	100
⅞ in.	89	65	90
⅜ in.	70	50	70
No. 4	57	35	55
No. 40	32	15	30
No. 100	13	-	-
No. 200	4	-	-

Table 4.2 - Results of Triaxial Testing for Odessa Raw Base Material

Parameter	Tex-117-E	Tex-143-E*
Classification	2.9	2.7
Angle of Internal Friction, φ	53	44
Cohesion, c, psi	5.6	18.5
Strength at Zero Lateral Pressure, psi	34	*
Strength at Lateral Pressure of 15 psi, psi	168	*
Grade as per Item 247	3	*
*Not Applicable		

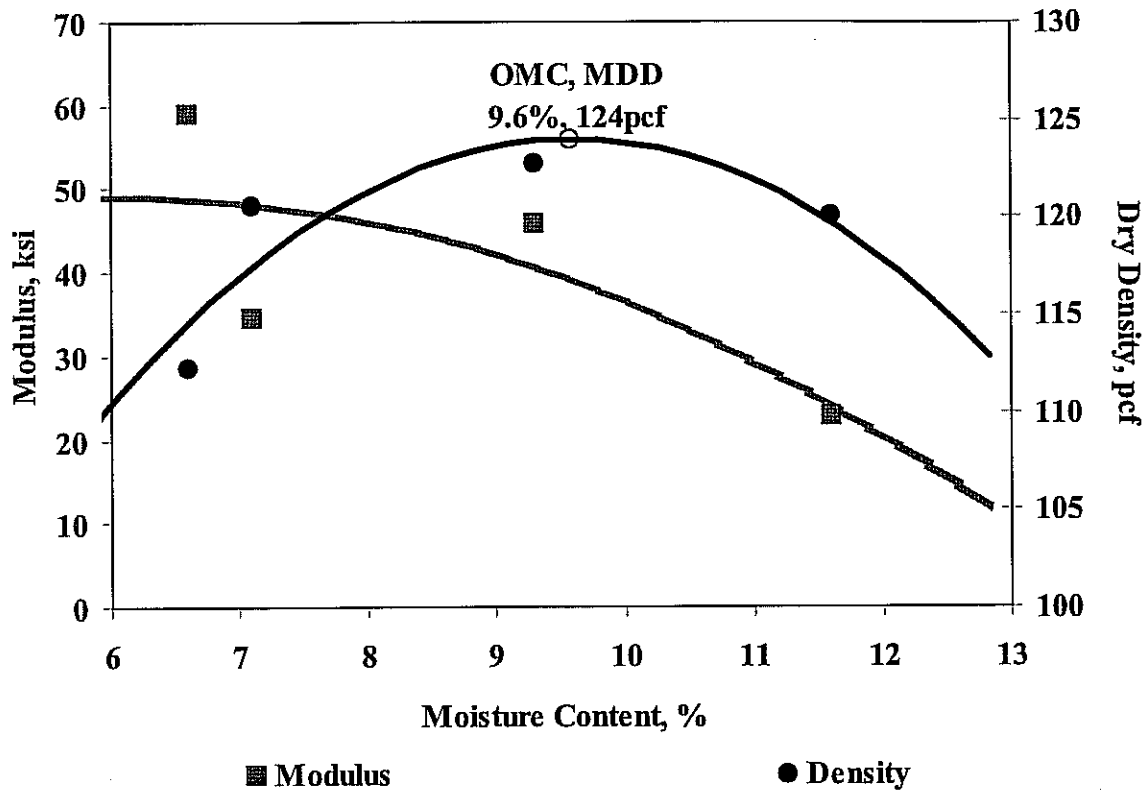


Figure 4.5 – Moisture Density/Modulus Curves for Odessa Raw Base

Table 4.3 – Variation in Optimum Moisture Content, Maximum Dry Unit Weight and Modulus for Raw Base

Parameter	Cement Content, %			
	0	1.5	3.0	4.5
Optimum Moisture Content, %	9.6	10.4	10.4	9.9
Maximum Dry Unit Weight, pcf	124	114	121	120
Seismic Modulus at OMC, ksi	39	273	141	211

The base-cement specimens were then prepared and cured as per Tex-120-E, and tested to obtain the unconfined compressive strengths. Figure 4.6 shows the variation in the unconfined compressive strength with cement content. The strength increases as the cement content increases. All three UC strengths exceeded the 300 psi limit.

Retained Strength Ratio

The 10-day capillary and 4-hr soak moisture conditioned specimens were prepared and subjected to UC strength tests for the three cement contents. These strengths were then compared to the strengths presented in Figure 4.6 to obtain the corresponding retained strength ratios (RSR). The retained strength ratios are presented in Figure 4.7. For the capillary moisture conditioned specimens, the mixes with 3.0% and 4.5% cement exhibited RSRs that were greater than 85%;

yet, none of the specimens with 4-hour soak met this limit. In this case, the 4-hr soak provided harsher moisture conditioning as compared to the capillary saturation.

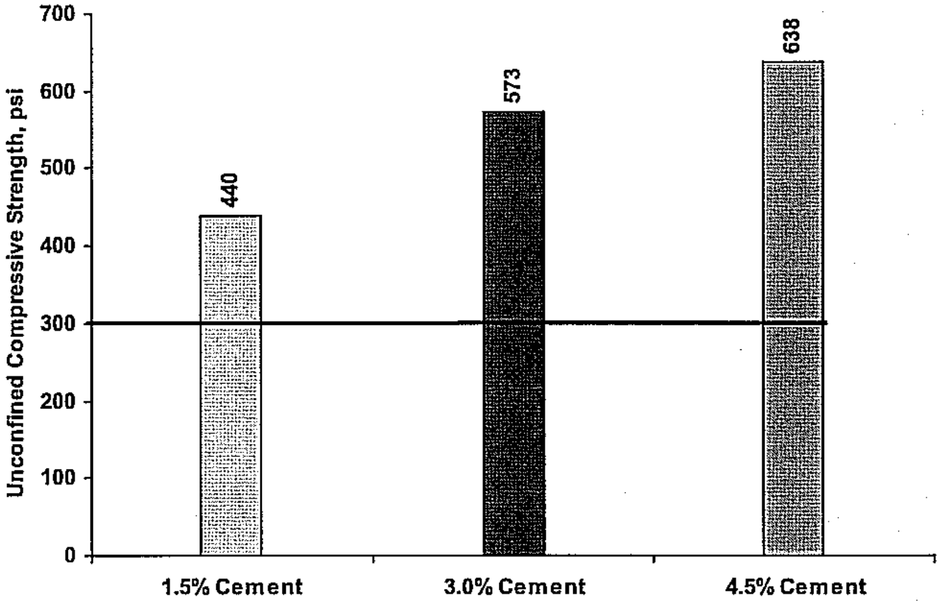


Figure 4.6 – Variation in Unconfined Compressive Strength with Cement Content Using Odessa Raw Material

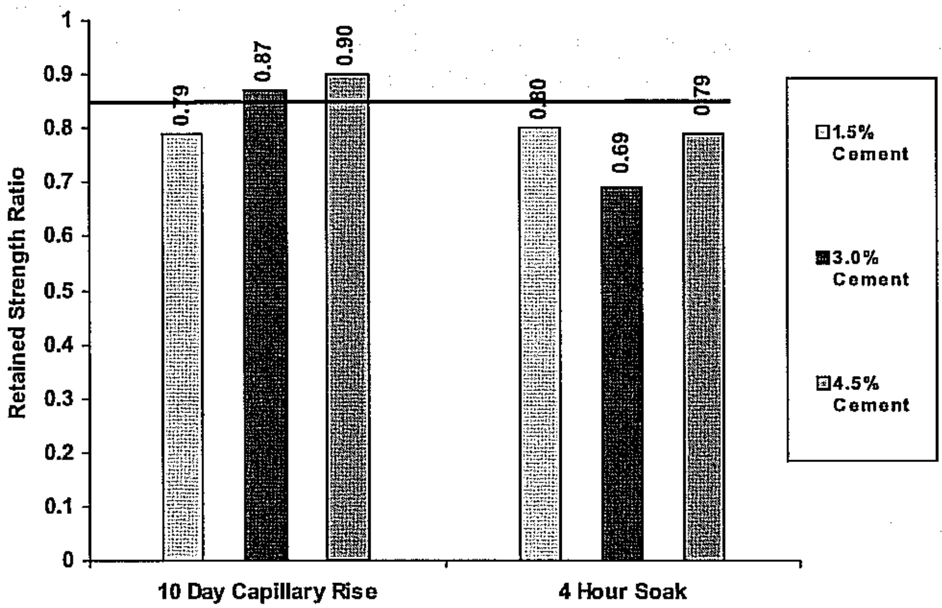


Figure 4.7 – Retained Strength Ratios for Capillary and 4-Hour Soak Moisture Conditioning

Seismic Modulus

The variation in seismic modulus with cement content is presented in Figure 4.8. The results correlate well with the UC results presented in Figure 4.5. The seismic modulus increases as the cement content increases. Once again, the moduli for the three cement contents exceed the value of 1,000 ksi.

Retained Modulus Ratio

The variation in retained modulus ratio (as defined in Chapter 3) with cement content is shown in Figure 4.9. The RMR for both test methods as well as for the three cement contents met and exceeded the 85% limit. By comparing the RMR values from both conditioning methods, the 4 hour soak is more conservative, as shown in Figure 4.10.

Dielectric Constant

The variation in dielectric constants for the capillary moisture conditioned specimens with cement content is presented in Figure 4.10. The mixes with 0% and 1.5% cement contents exceed the limit of 10; but, those with 3.0% and 4.5% cement contents are well below the limit.

Final Moisture Content

The variations in initial (as compacted) and final (after 10 day capillary saturation) moisture contents are reported in Figure 4.11 as a confirmation of the dielectric values. For the 0% and 1.5% cement, the final moisture content is greater than initial, confirming these mixes may be susceptible to moisture. However, for the mixes with 3% and 4.5% cement, the final moisture content is less demonstrating the lack of affinity of the specimens to water.

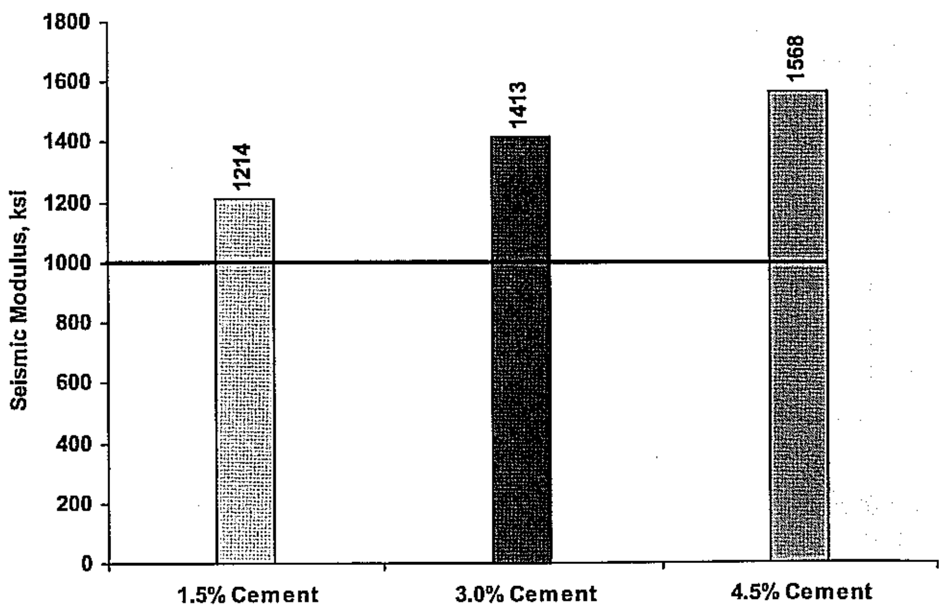


Figure 4.8 – Variation in Seismic Modulus with Cement Content Using Odessa Raw Material

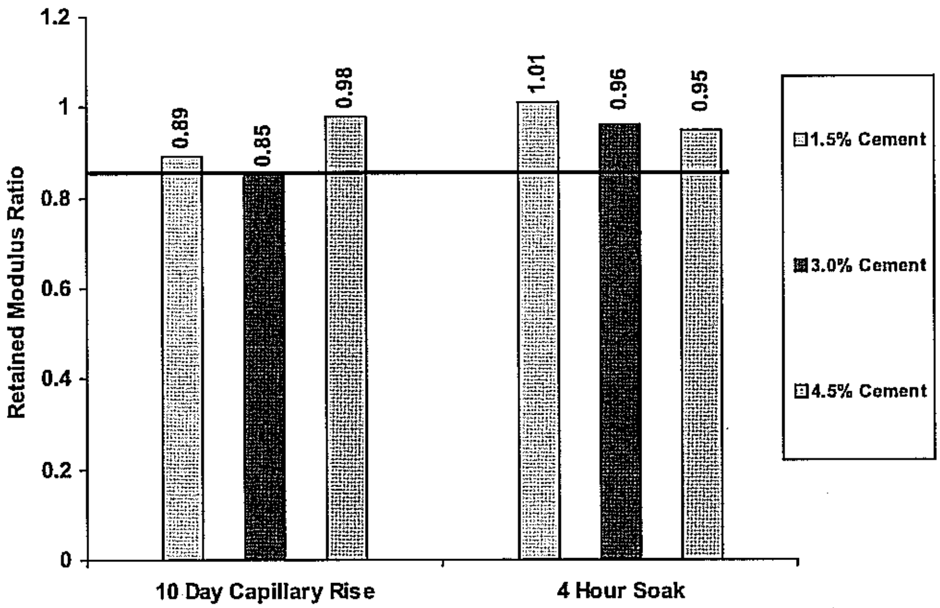


Figure 4.9 – Retained Modulus Ratios for Capillary Moisture Conditioning and 4-Hour Soak

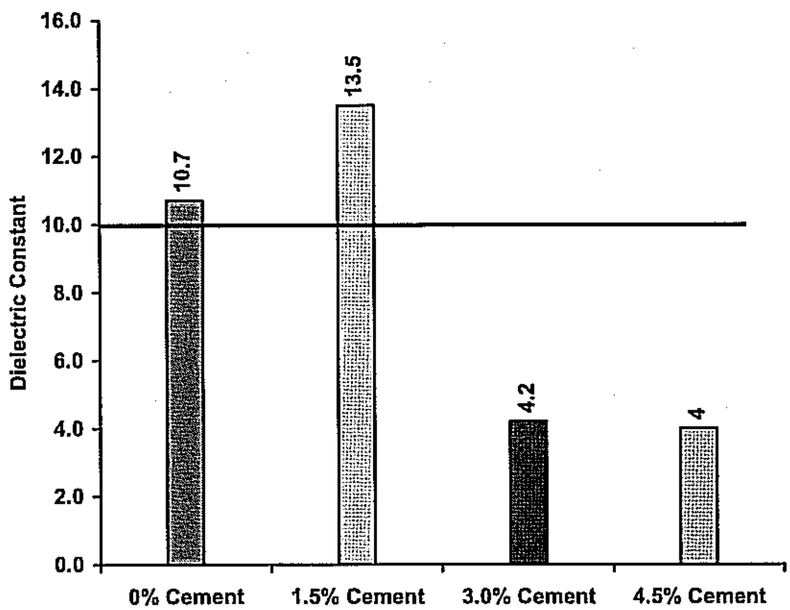


Figure 4.10 – Dielectric Constant for Capillary Moisture Conditioning Specimens

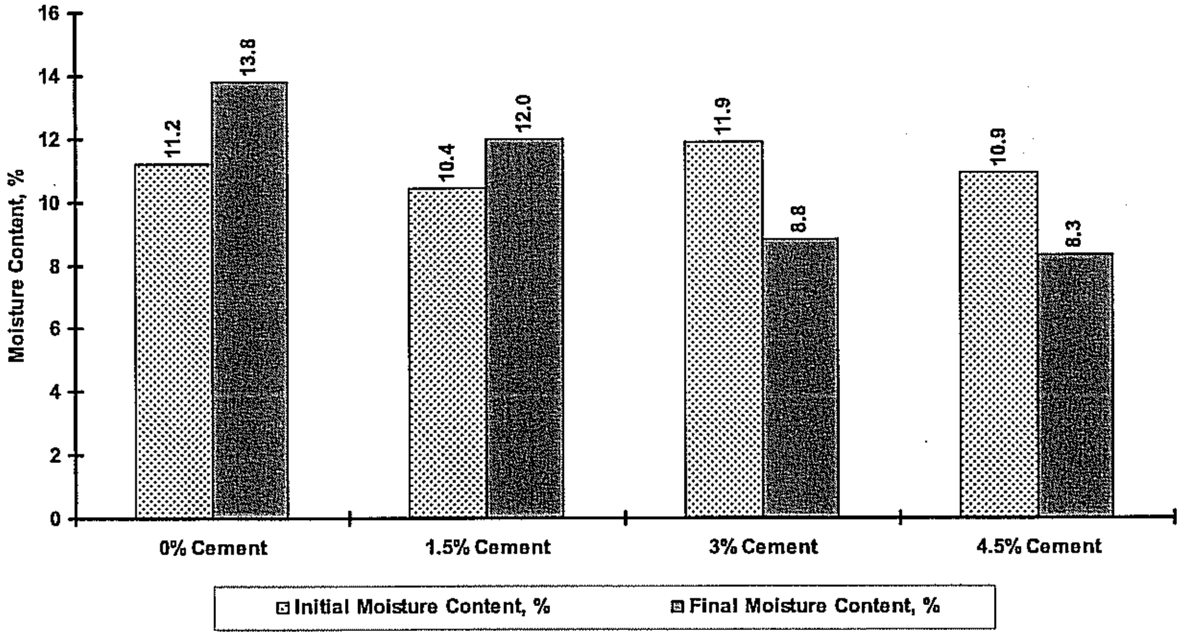


Figure 4.11 – Initial and Final Moisture Content for Capillary Moisture Conditioning

Final Seismic Modulus

The seismic moduli after ten days of moisture conditioning for the specimens subjected to TST tests are presented in Figure 4.12. Once again, the moduli of the cement stabilized materials are above 1,000 ksi.

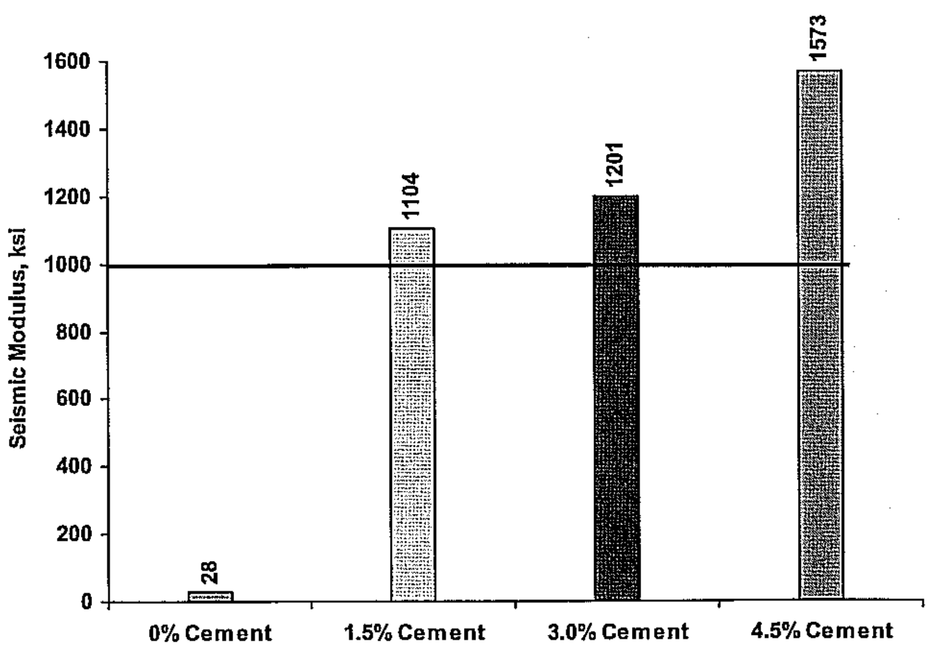


Figure 4.12 – Seismic Modulus for Capillary Moisture Conditioning

Determination of Optimum Cement Content

The determination of optimum cement content is based on the requirements that must be met in terms of strength, modulus, and moisture susceptibility. These requirements are as follows:

- UC strength of 7 day cured specimen ≥ 300 psi
- Seismic modulus of 7 day cured specimen ≥ 1,000 ksi
- Retained strength and modulus ratios ≥ 85%
- Dielectric constant of capillary moisture conditioned specimens < 10

The test results for the Odessa Raw are summarized in Table 4.4. The requirements that are not met for each mix are highlighted. Although, the 1.5% cement content met the UC strength, RMR and seismic modulus criteria, the dielectric constant value exceeded 10, and the RSR was marginally less than 85%. For the 3.0% and 4.5% cement contents, the UC strengths and seismic moduli far exceeds the limits and other requirements were met with the exception of the RSR for the 4 hour soak method. Therefore, 1.5% to 3.0% cement is optimum. To minimize the potential for cracking, perhaps cement content closer to 1.5% is desirable. As such, the 2% cement utilized in the construction seems reasonable. The estimated parameters for 2% cement from other three cement concentrations are also shown in Table 4.4.

Table 4.4 - Results for Odessa Raw to Determine Optimum Cement Content

Criterion		Cement Content, %			
		1.5	**3.0**	**4.5**	**2%***
Tex-120-E	**Unconfined Compressive Strength, psi**	440	573	638	**492**
	Seismic Modulus, ksi	1241	1413	1568	**1300**
Retained Strength Ratio	**Capillary Saturation**	0.79	0.87	0.90	**0.82**
	4 Hour Soaking	0.80	0.69	0.79	**0.74**
Retained Modulus Ratio	**Capillary Saturation**	0.89	0.85	0.98	**0.86**
	4 Hour Soaking	1.01	0.96	0.95	**0.99**
Capillary Moisture Conditioning	**Final Dielectric Constant**	13.5	4.2	4.0	**9.4**
	Final Moisture Content,%	12.0	8.8	8.3	--
	Final Seismic Modulus, ksi	1104	1201	1573	**1106**

*Estimated from other three concentrations

Results for Odessa Pulverized Materials

Gradation

The gradations of the pulverized materials are compared with that of the raw material in Figure 4.13 and are summarized in Table 4.5a. While Odessa Raw gradation fell on or slightly above the Item 247 maximum allowable limits, the pulverized materials are even finer. The gradation curves for the second (Pt. 4-Pass 2) and third (Pt. 4-Pass 3) pulverization passes are above the rest of the gradation curves indicating those materials are slightly finer than the rest.

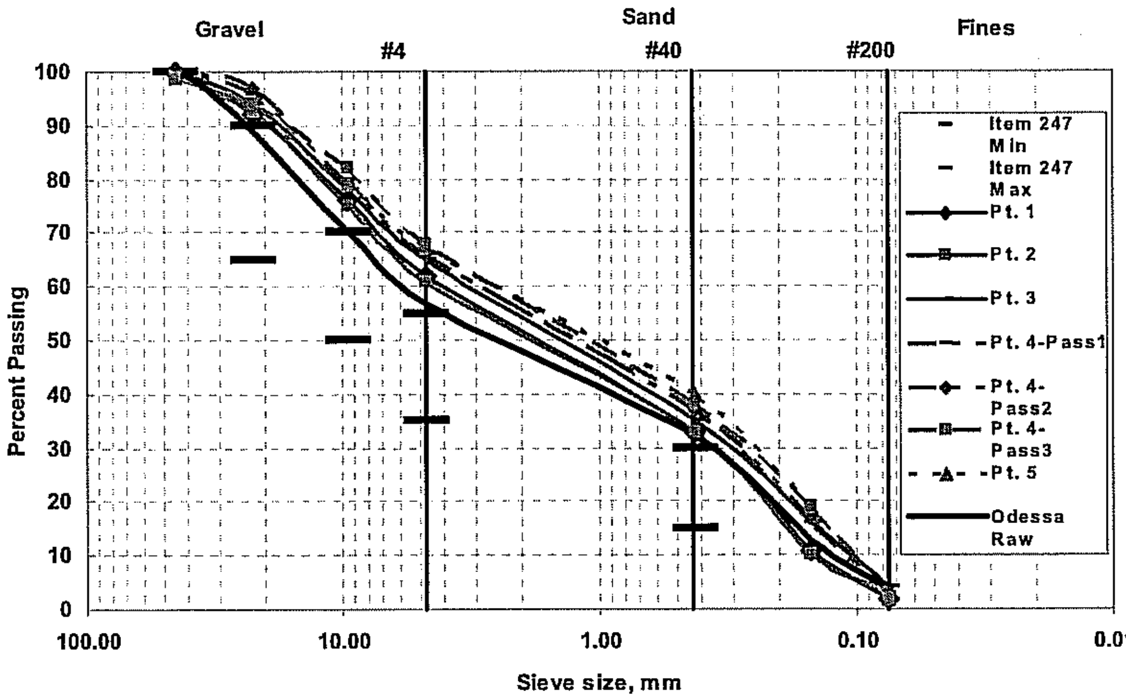


Figure 4.13 - Gradation for Odessa Pulverized Base Material Compared to Item 247 Limits

Table 4.5 - Gradations for Odessa Pulverized Material

a) Sieve Analysis

Sieve Size	Percent Passing							
	Odessa Raw	Odessa Pulverized						
		Pt. 1	Pt. 2	Pt. 3	Pt. 4			Pt. 5
					Pass 1	Pass 2	Pass 3	
1¾ in.	100	100	99	100	100	100	99	99
⅞ in.	89	92	93	94	96	97	94	93
⅜ in.	70	76	75	78	80	79	82	79
No. 4	57	62	61	65	66	66	68	67
No. 40	32	33	33	35	37	37	38	40
No. 100	13	11	10	16	16	17	19	19
No. 200	4	2	3	4	3	2	2	2

b) Change in Material Constituents

Material Constituents	Content, %							
	Odessa Raw	Odessa Pulverized						
		Pt. 1	Pt. 2	Pt. 3	Pt. 4			Pt. 5
					Pass 1	Pass 2	Pass 3	
Gravel	43	38	39	35	34	34	32	33
Coarse sand	25	29	28	30	29	29	30	27
Fine sand	28	31	30	31	34	35	36	38
Fines	4	2	3	4	2	2	2	2

Changes in constituents of the materials due to pulverization are presented in Table 4.5b. As anticipated the gravel content (materials retained on No. 4 sieve) decreased by 4% to 10% for the first pass of the pulverizer. The second and third passes however reduced the gravel content slightly (less than 2%). This indicates that most of the crushing of the gravel size aggregates occurred during the first pass.

The coarse sand content (materials passing No. 4 and retained on No. 40 sieves) increased by 2% to 5% during the first pass, while the second and thirds passes did not significantly change the coarse sand content. The fine sand content (materials passing No. 40 and retained on No. 200 sieves) also increased by 2% to 8% during the first pass; however, the second and third passes increased the fine sand content by 2% to 3%. The fine contents did not seem to change much.

Based on this particular case study, the first pass of the pulverizer is the one that causes most of the changes in the gradation of the in-place materials. The pulverized materials contain less gravel as compared to the in-place material, and more fine sands. The coarse sand and fines do not seem to change much.

Determination of Strength of Pulverized Materials

The results from Texas and Standard Triaxial tests for the pulverized material are compared with those from the raw material in Table 4.6. The classification from Tex-117-E tests ranges from 2.4 to 3.2 (as compared to 2.9 for the raw materials), which, indicates a fair to borderline flexible base material. The angles of internal frictions and cohesions from the raw and pulverized materials are reasonably close to one another. The strengths at zero lateral pressure are less than desirable for all points.

The classification from Tex-143-E tests on pulverized materials ranges from 2.1 to 2.8 (as compared to 2.7 from raw materials) which indicates a fair flexible base material. The angles of internal frictions for the pulverized materials are similar or greater than that from the raw materials. Overall, the soil at each point was classified as Grade 3 based on the comparison to the specifications in Item 247 for gradation and strength at 0 psi and 15 psi lateral pressures.

Stabilized Base Strength

Specimen preparation for the soil-cement specimens using Tex-120-E was based on the optimum moisture and cement contents used in the construction of the base. The unconfined compressive strengths from pulverized points are compared to that of the raw material in Figure 4.14. The strengths from all pulverized materials were less than the 492 psi strength estimated from the raw material with 2% cement. The pulverized materials with 2% cement after 7-day curing exhibited strengths in the range of 300 psi to 460 psi.

Retained Strength Ratio

The retained strength ratios from the pulverized points for the capillary moisture conditioning and 4 hour soak specimens are presented in Figure 4.15. The RSR for raw materials with 2% cement with capillary moisture conditioning is 82% and for 4-hour soak is 74%. In Figure 4.15a,

Table 4.6 - Results of Triaxial Testing for Odessa Pulverized Base Material

Parameters	Tex-117-E							
	Odessa Raw	Odessa Pulverized						
Gradation		Pt. 1	Pt. 2	Pt. 3	Pt. 4 Pass 1	Pt. 4 Pass 2	Pt. 3 Pass 3	Pt. 5
Classification	2.9	3.2	2.9	2.4	2.9	2.4	3.2	2.9
Angle of Internal Friction, φ	53	52	51	56	54	52	52	50
Cohesion, c, psi	6	4	6	7	5	8	4	6
Strength at Zero Lateral Pressure, psi	34	24	26	42	42	37	28	27
Strength at Lateral Pressure of 15 psi, psi	168	126	116	176	173	164	169	131
Grade as per Item 247	3	3	3	3	3	3	3	3
Parameters	Tex-143-E							
Classification	2.7	2.4	2.1	2.1	2.6	2.8	2.7	2.3
Angle of Internal Friction, φ	44	47	50	51	45	43	44	49
Cohesion, c, psi	18	12	8	13	11	17	18	13

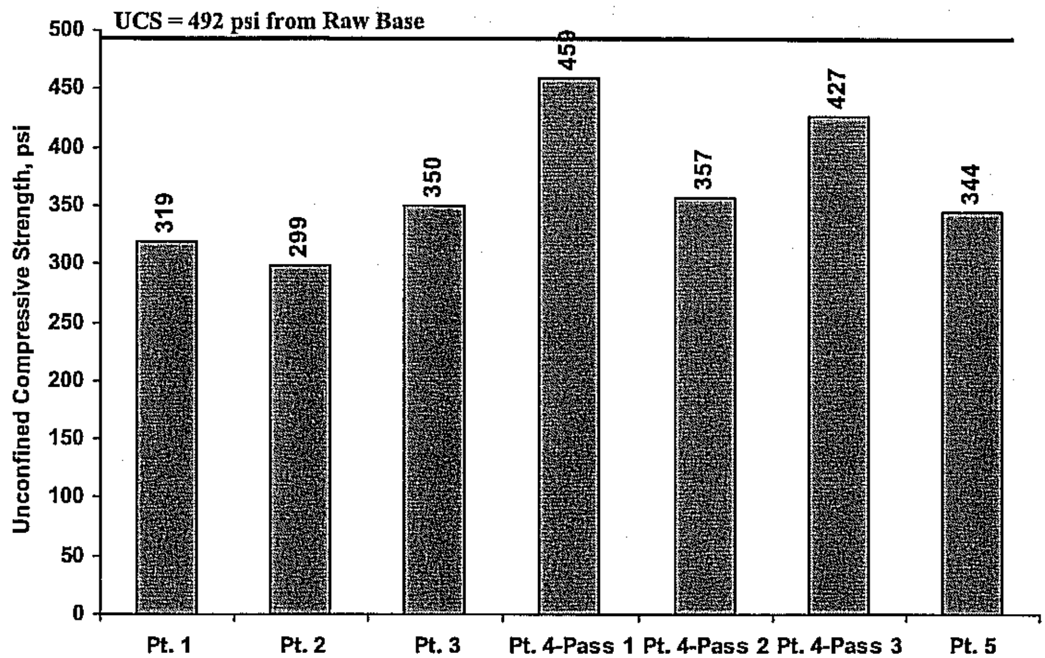


Figure 4.14 – Unconfined Compressive Strength of Soil-Cement Specimens for Odessa Pulverized

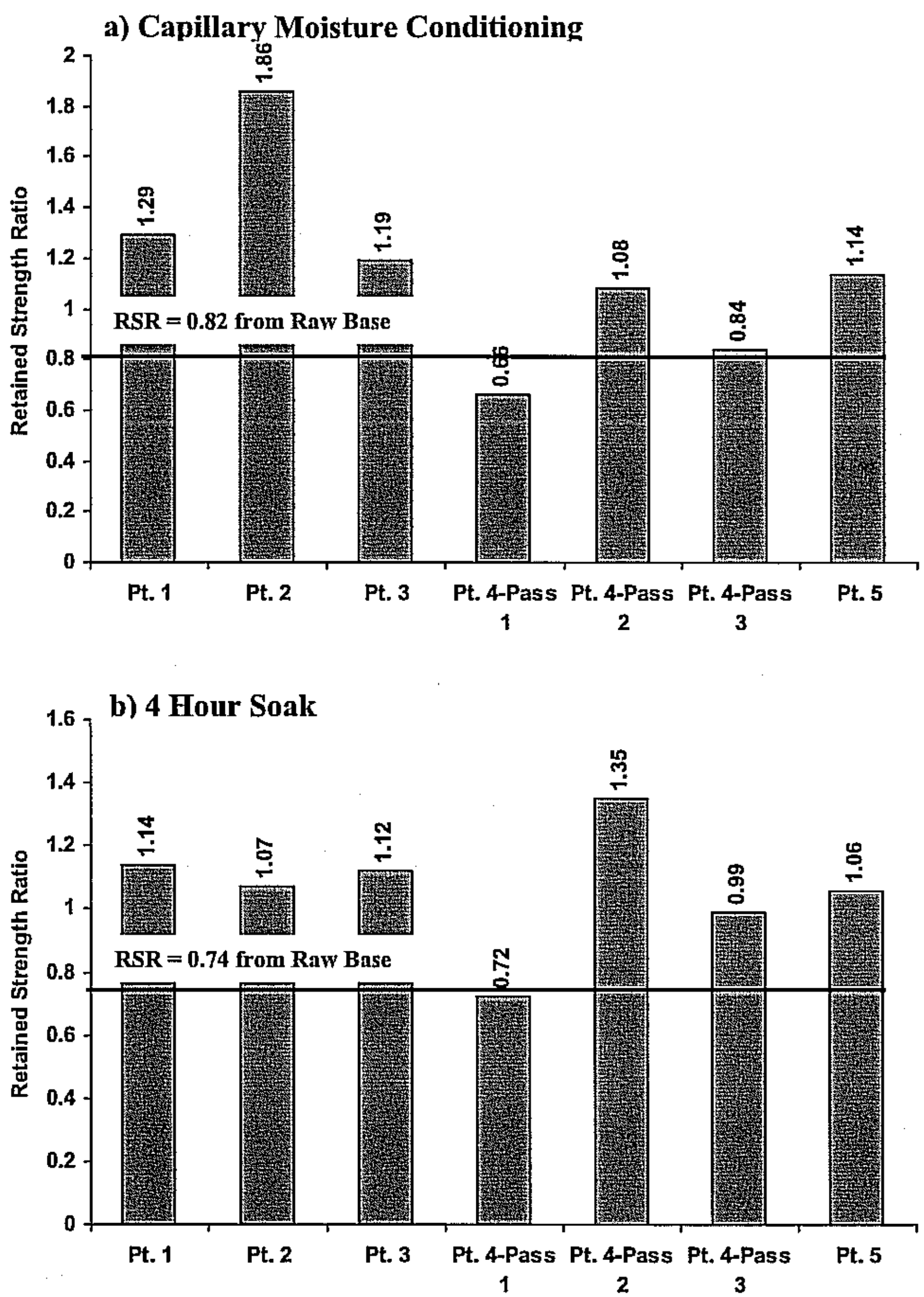


Figure 4.15 – Retained Strength Ratios for Capillary Moisture Conditioning and 4-Hour Soak

one point yields an RSR with less than 82% when the capillary moisture conditioning was used. As shown in Figure 4.15b, the same point also yielded an RSR that was less than that obtained from the raw materials when 4-hr soak was used.

Seismic Modulus

The seismic moduli from the stabilized pulverized materials are compared with the modulus obtained from the raw material in Figure 4.16. All specimens meet or exceed the 1300 ksi modulus obtained from the raw materials. This is contrary to the UC strengths where for all pulverized mixes the strengths were less than that obtained from the raw material (see Figure 4.14). As such, both the strength and stiffness of the mixes should be measured.

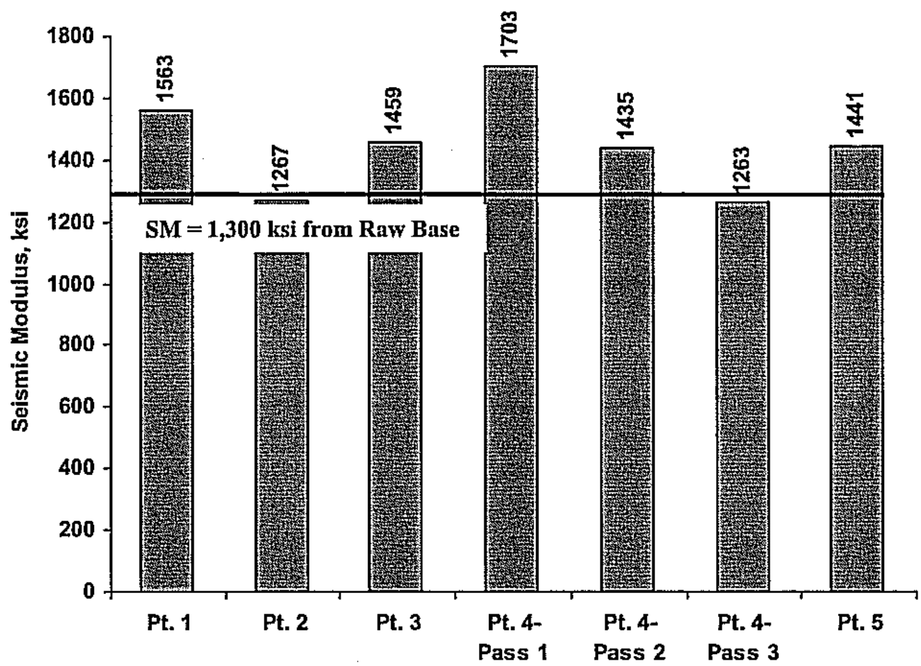


Figure 4.16 – Seismic Moduli for 7 Day Cured Specimens for Odessa Pulverized

Retained Modulus Ratio

The retained modulus ratios for all pulverized mixes are shown in Figure 4.17. As a reference, the RMRs for the capillary moisture conditioned and 4 hour soaked specimens from the raw material were 86% and 99%, respectively. The RMRs of the capillary moisture conditioned pulverized specimens were generally less than the 86% obtained from the raw materials (Figure 4.17a). Similar trend was observed for only three points when the 4-hr soak was used. Based on modulus, the mix may be considered moisture susceptible, despite the fact that the retained strengths were satisfactory in Figure 4.15.

Dielectric Constant

The dielectric constant for each pulverized mix is presented in Figure 4.18. The laboratory dielectric value obtained with the raw material was 9.4. Only two points (Point 1 and 2) do not exceed the limit of 9.4. Except for Points 1 and 2, the dielectric constants are greater than 9.4. The dielectric values are significantly higher for the materials that were pulverized more than once. This trend indicates the possibility of moisture susceptibility of the pulverized materials as suggested by the RMR values.

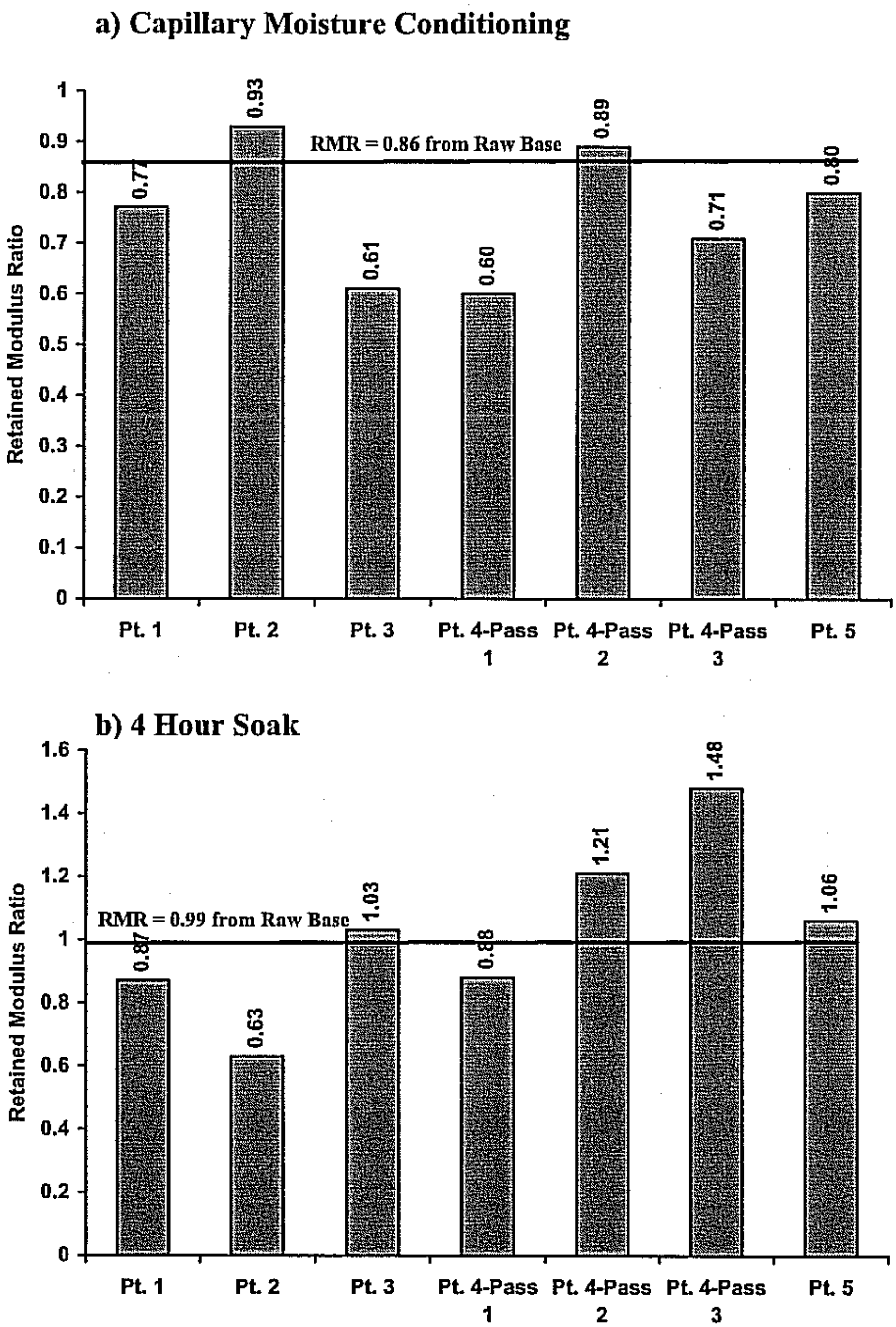


Figure 4.17 – Retained Modulus Ratios for Capillary Moisture Conditioning and 4-Hour Soak for Odessa Pulverized

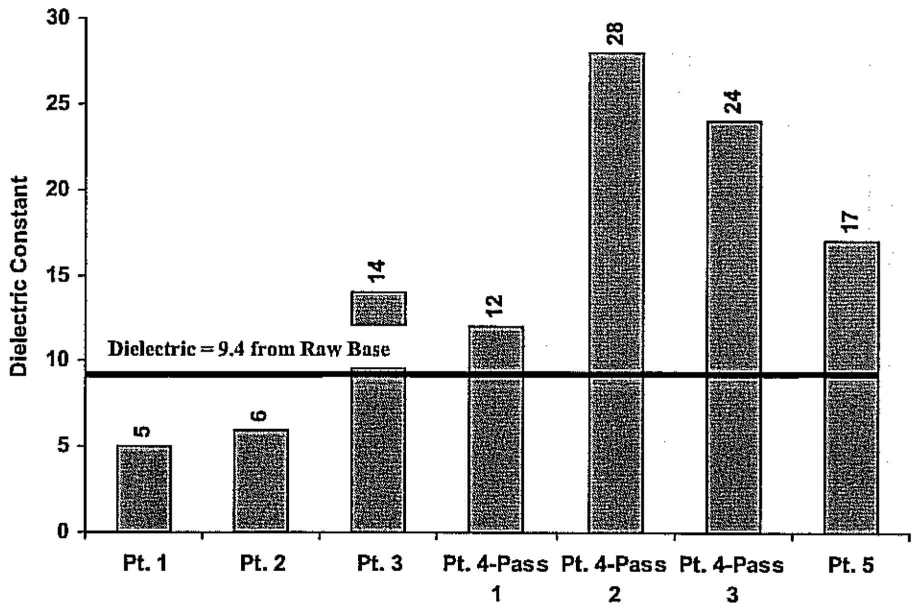


Figure 4.18 – Dielectric Constant for Capillary Moisture Conditioning for Odessa Pulverized

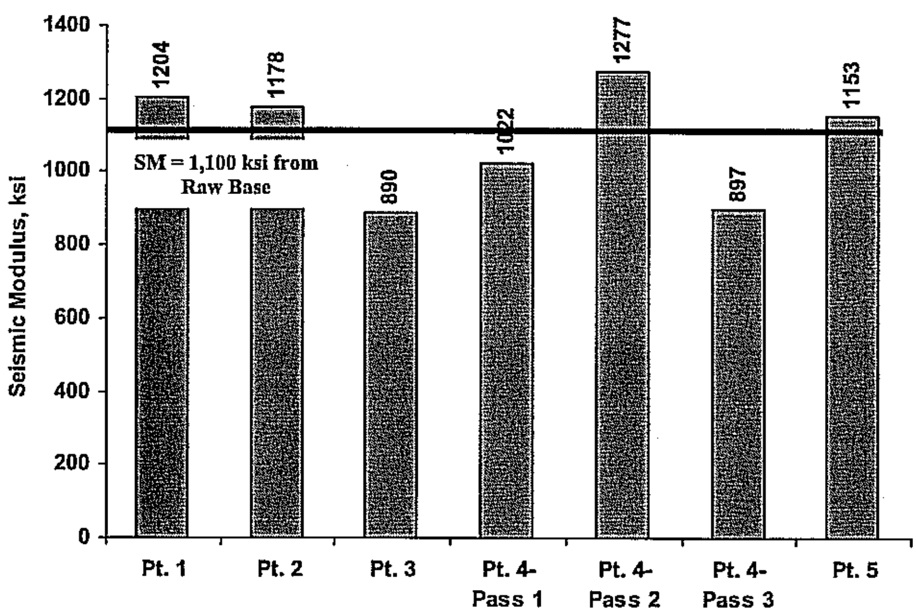


Figure 4.19 – Final Seismic Moduli after Moisture Conditioning

Final Seismic Modulus from Moisture Susceptibility Tests

The seismic moduli after moisture conditioning following the Tube Suction Test protocol (2 day drying, 8 day wetting) are shown in Figure 4.19. Several points yield moduli that are slightly greater than that obtained from the raw material (1,100 ksi), and some yielded lower moduli.

Results from Field Tests

Field tests with NDG, FWD and PSPA were conducted on the base after the construction was completed. The NDG readings were carried out at 10 locations with an interval of 0.1 mile within 24 hours of the completion of the base. The variations in dry density and moisture content from NDG tests at the site are shown in Figure 4.20. The average dry density from NDG tests was 118 pcf with a standard deviation of 2.1 pcf (COV = 2%). The average moisture content from these tests was 7.8% with a standard deviation of 0.7% (COV = 10%).

The FWD and PSPA tests were conducted at 25 stations with an interval of 200 ft three days after the completion of the base. The main goal of FWD and PSPA tests was to characterize the stiffness of the new base. The variations in moduli from FWD and PSPA as well as deflection from sensor 1 (w1) of FWD at the site are shown in Figure 4.21. The average modulus backcalculated from the FWD deflection data was 288 ksi with a COV of 47%. Significant judgment required in backcalculating the moduli of the stabilized base. The average modulus from PSPA direct measurements was 472 ksi with a COV of 21%.

Structural Evaluation

The process discussed in Chapter 3 to determine suitable ACP and base thickness based on moduli was applied to the test results from Odessa material. The control base for this case is the Odessa Raw material. The pavement system at the site is a four-layer system as shown in Figure 4.2. The ACP layer is 4.5 in., cement stabilized base is 6.0 in., original base is 12.0 in., and the subgrade is considered a semi-infinite layer. The moduli and Poisson's ratio used in the calculations are presented in Table 4.7.

Table 4.7 – Pavement Layer Properties Used in Structural Analysis-Odessa Pulverized

Layer	Thickness, in.	Modulus, ksi	Poisson Ratio
AC	4.5	500	0.33
Cement Stabilized Base	6	Based on FFRC, FWD and PSPA results	0.35
Original Base	12	117	0.35
Subgrade	Semi Infinite	33	0.4

Average moduli from FWD and PSPA are compared to the various moduli obtained in the laboratory in Figure 4.22. The ratio of the FWD and PSPA moduli is about 1.6 which is consistent with the result from a previous research that the seismic modulus is about 1.7 times the FWD modulus for a granular base (Nazarian et al., 1996).

All lab moduli were obtained from FFRC tests, which should be compatible to the field moduli obtained with the PSPA. The lab moduli vary significantly depending on the curing and moisture-conditioning. The minimum modulus is obtained from tests on the moisture-density specimens at the traditional OMC (marked at "At OMC" in the figure); while the highest modulus is associated with the specimens cured for 7 days as per Tex-120-E. Allowing for the increase in the strength of the in-place base with time, the most representative modulus seems to be associated with the seismic modulus determined 24 hours after oven drying of the TST specimens. This matter will be comprehensively addressed in the Phase II report, when the field data from all four sites at different ages are obtained.

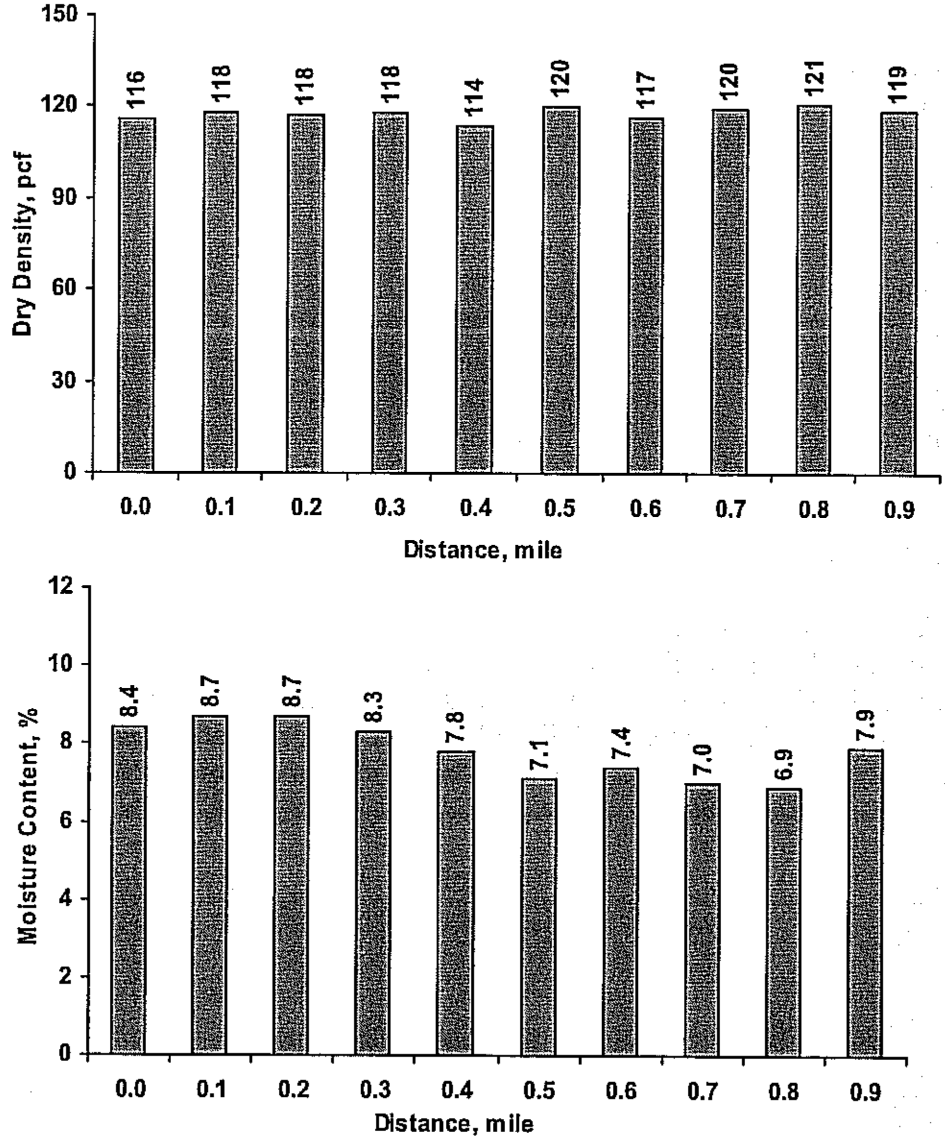


Figure 4.20 – Variations in Dry Density and Moisture Content at Odessa Site

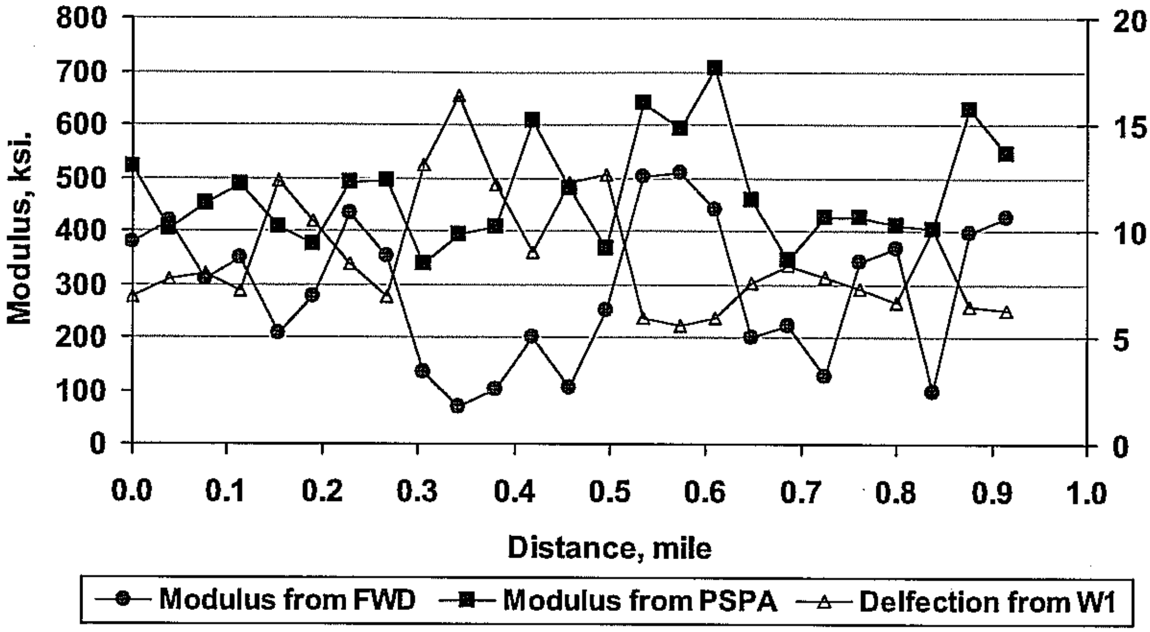


Figure 4.21 – Variations in Seismic Modulus and Deflection at Odessa Site

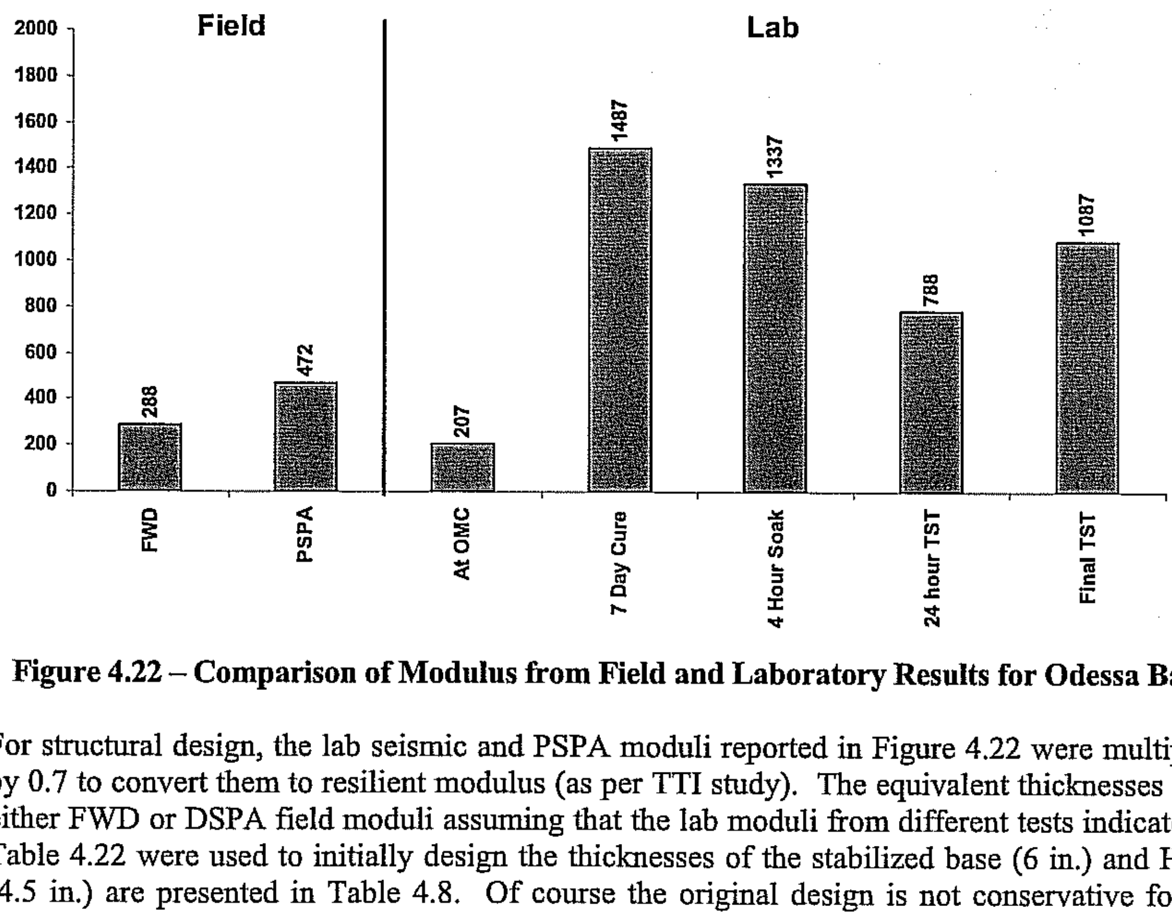


Figure 4.22 – Comparison of Modulus from Field and Laboratory Results for Odessa Base

For structural design, the lab seismic and PSPA moduli reported in Figure 4.22 were multiplied by 0.7 to convert them to resilient modulus (as per TTI study). The equivalent thicknesses from either FWD or DSPA field moduli assuming that the lab moduli from different tests indicated in Table 4.22 were used to initially design the thicknesses of the stabilized base (6 in.) and HMA (4.5 in.) are presented in Table 4.8. Of course the original design is not conservative for the modulus at OMC. Therefore, the thickness of the HMA has to be decreased. Since the modulus from the 7-day-cured specimens and retained moduli from capillary saturation and 4 hour soak are significantly greater than the field moduli, either the HMA thickness should be increased by 2 in. to 3 in. or the stabilized base should be thickened by 2 in. to 4 in. Using the modulus from TST specimens after 24 hours, the equivalent thicknesses are more or less adequate.

It should be emphasized again that this and other sections are under periodical evaluation. More comprehensive recommendations for the appropriate test to determine realistic design moduli will be made in Phase II report.

Table 4.8 – Layer Thicknesses Required for Equivalent Performance when Laboratory Moduli Used in Comparison to Field Results for Odessa Base

Laboratory Moduli	HMA Thickness (in.) when Base Thickness Maintained Constant at 6 in.		Base Thickness (in.) when HMA Thickness Maintained Constant at 4.5 in.	
	PSPA Modulus	FWD Modulus	PSPA Modulus	FWD Modulus
Original Design	4.5		6	
At OMC	5.5	5.5	N/P*	N/P*
7-Day Cured Moduli	7.5	8.0	10	10
Capillary Saturated Moduli	7.0	7.5	9	10
4-hr Soak Moduli	7.5	8.0	10	10
24 hr TST	6.0	6.5	8	8

*** Not a practical solution, since the field modulus is greater than the lab moduli, thickening of the base will not satisfy the fatigue cracking of HMA criterion**

Chapter 5

Summary, Preliminary Conclusions and Future Work

Rehabilitation of highway pavements through FDR is an option chosen by more and more state transportation agencies. This option decreases cost by reducing the use of virgin aggregate for base material. While FDR is a viable alternative, pulverization of the asphalt layer with base material or base material alone may change the strength of the base layer due to the formation of fine materials during the crushing action of the pulverizer. Typically, a stabilizer is used in the FDR process which aids in strength gain for the base layer. The stabilizers mostly used by TxDOT are cement, lime and fly ash. The optimum stabilizer content is currently determined either based on experience or through a series of laboratory tests that evaluates the strength, stiffness and durability of the base-stabilizer mix. For lab testing, base materials are retrieved from the site way before the pulverization activity. The change in gradation due to pulverization can significantly impact the base strength and stiffness.

The main objective of this research project is to evaluate the effects of pulverization on the base properties and to determine the optimum stabilizer content necessary to obtain a reasonably strong, stiff and durable base layer that will perform well for a long time.

The current TxDOT protocols were comprehensively evaluated using a base from El Paso. The impacts of change in gradation, as well as stabilizer type and content, moisture-density curve, modulus, unconfined compressive strength, moisture susceptibility and structural design were studied. Based on the results of this evaluation, preliminary recommendations for modifying the protocol were made. As an example, the existing and recommended protocols were applied to the materials from an actual site in Odessa. The laboratory and field results were compared for that site.

Based on the knowledge gained so far, the following observations were made for the El Paso limestone base:

- The optimum moisture content and the maximum dry unit weight for the cement stabilized materials seem to be close to those obtained from raw materials. For lime and fly ash stabilized material, however, the OMC and MDD were lower than the raw materials.

- The UCS for cement stabilized material consistently increased as the cement content increased. Yet, for lime and fly ash stabilized specimens, the UCS decreased as the stabilizer content increased after the specimen was subjected to moisture conditioning. When specimens were tested prior to moisture conditioning, the lime specimens showed and increase in UCS with an increase in lime content. The fly ash specimens showed a decrease in UCS as the percentage of fine material increased.
- The retained strength ratio (RSR) of 85% for cement stabilized soil was readily achieved regardless of the blend or cement content. The lime and fly ash specimens did not achieve the RSR of 85% for any case. The four-hour soak method for moisture conditioning typically yields greater RSR as compared to 10-day capillary moisture.
- The retained modulus ratio (RMR) trends were similar to the RSR for cement stabilized specimens. For lime and fly ash stabilized material, the RMR was not achieved by any combination of blend or stabilizer content. RSR can perhaps be replaced by RMR to minimize the number of specimens necessary for determining the retained strength and modulus.
- The dielectric constants for the stabilized specimens varied significantly as the percentage of stabilizer was changed.
- The final (10-day) moisture contents from specimens prepared for the tube-suction tests were normally greater than the initial moisture contents.
- As the sand content of the mix increases, the strength and stiffness of the stabilized mix decreases. As such more additives are required if the pulverization turns gravel to sand. When the fines content increases, the strength and stiffness of the mix is slightly compromised. However, the moisture susceptibility of the mix may increase.

After applying the developed protocol to the Odessa base material, the following preliminary findings were made:

- The gradation for the pulverized material contained less gravel, more coarse sand, more fine sand as compared to the raw material. Yet, the fine material (passing No. 200 sieve) did not change significantly.
- The OMC and MDD did not vary significantly for the raw material with different cement contents or for the pulverized materials.
- The UCS of stabilized base from raw materials was greater than those from the pulverized materials, but still acceptable.
- The RSR for raw material with design cement content were generally less than those obtained on pulverized materials with 2% cement.
- The moduli of the mixes with pulverized materials were greater than that from raw materials. But the moisture susceptibility of the pulverized materials in terms of modulus is more severe. The dielectric value from the mixes with raw base was less than those from the pulverized materials.
- Moduli from lab tests after seven day cure are significantly greater than those measured in the field. Field moduli preliminary fall between lab moduli measured after 24 hours with room curing and after 24 hours of oven drying.
- More definitive recommendations will be made in Phase II report where the results from field and lab are comprehensively analyzed.

The information provided in this report is based on the results of the first phase of this study. More work is ongoing at four other sites throughout Texas to better define the impact of pulverization on the short-term and long-term performance of pulverized materials. In the second phase, different types of bases and additives and different construction methods are being considered.

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