# Guidelines for Using Local Materials for Roadway Base and Subbase

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Center for Transportation Infrastructure Systems
The University of Texas at El Paso
El Paso, TX 79968
(915) 747-6925
http://ctis.utep.edu

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16. Abstract

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by

**Biraj Gautam, BSCE**
**Deren Yuan, PhD**
**Imad Abdallah, MSCE**
**And**
**Soheil Nazarian, PhD, PE**

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Biraj Gautam, BSCE
Deren Yuan, PhD
Imad Abdallah, MSCE
Soheil Nazarian, PhD, PE (66495)

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# Abstract

Flexible base materials that meet TxDOT specifications are getting more difficult to purchase in many TxDOT Districts. As a result, high quality materials have to be hauled in long distances, sometimes from other States. This act would significantly increase the costs associated with the construction of roads and subsequent maintenance and rehabilitation of them. Out-of-specification local materials are normally available. If through appropriate modifications of the materials (adjusting the gradation or/and chemical treatment) or structural design (specifying thicker layers of base and/or hot mix) the use of the local materials can be permitted, the construction can be accelerated and significant monetary benefits can be realized. Under the current TxDOT specification (Item 247), a material can be considered out-of-specification (low-quality) for a variety of reasons such as inadequate gradation, inadequate plasticity and inadequate strength. In many cases, the local flexible base supplies miss the standard specifications by a small margin. Since the criteria set in Item 247 are experienced-based, some of the parameters used to classify a base may be less significant than others. In this report, guidelines and test protocols for the use of out-of-specification base materials in low volume roads are recommended on the basis of test results of ten materials from five TxDOT districts.

# Implementation Statement

In this report a number of recommendations have been made to improve the use of out-of-specification materials as base and subbase layers in low volume roads with guidelines and test protocols. The recommendations are based on the test results of ten materials from five TxDOT districts.

At this time, the recommendations should be implemented on a number of new and ongoing projects to confirm their applicability and to adjust the limits and/or criteria recommended. As part of the implementation, a guide should be developed to decimate to the TxDOT staff.

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# Chapter 1

# Introduction

The use of high-quality base materials is generally required for pavement construction and rehabilitation to comply with conventional specifications. The source of these high-quality materials can be a long distance from the construction site, resulting in high transportation costs. The use of local sources of marginal materials or low-quality materials is not allowed if they do not comply with the existing specifications. Since the reserves of high-quality materials are diminishing in some regions, it is necessary to use local sources of unbound granular materials in pavements.

If through appropriate chemical treatment and/or gradation modification of the low-quality materials or proper structural design (specifying thicker layers of base and/or hot mix), the use of the low-quality materials for the purpose of low-volume roads can be permitted, the construction can be accelerated and the significant monetary benefits can be realized. A thorough evaluation of out-of-specification base materials from different local sources is essential to provide guidelines and test protocols for using these materials for roadway base and subbase.

## Objective

The main objective of this research project is to evaluate the out-of-specification/marginally low-quality base materials from local sources and develop comprehensive guidelines and test protocols for the use of such materials in the construction of low-volume roads.

To achieve this objective, a number of tasks were proposed and completed. A flow chart of the progression of these tasks is shown in Figure 1.1. The first step of the process was to identify those TxDOT districts that benefit the most from using local materials and to investigate the methods criteria that the districts currently use to incorporate local materials in their construction. The local bases from the districts that might benefit the most from this study were selected for comprehensive testing and evaluation.

The second step of the process consisted of extensive performance-based laboratory tests to determine whether the local materials can be used as-is. If a local material did not meet the

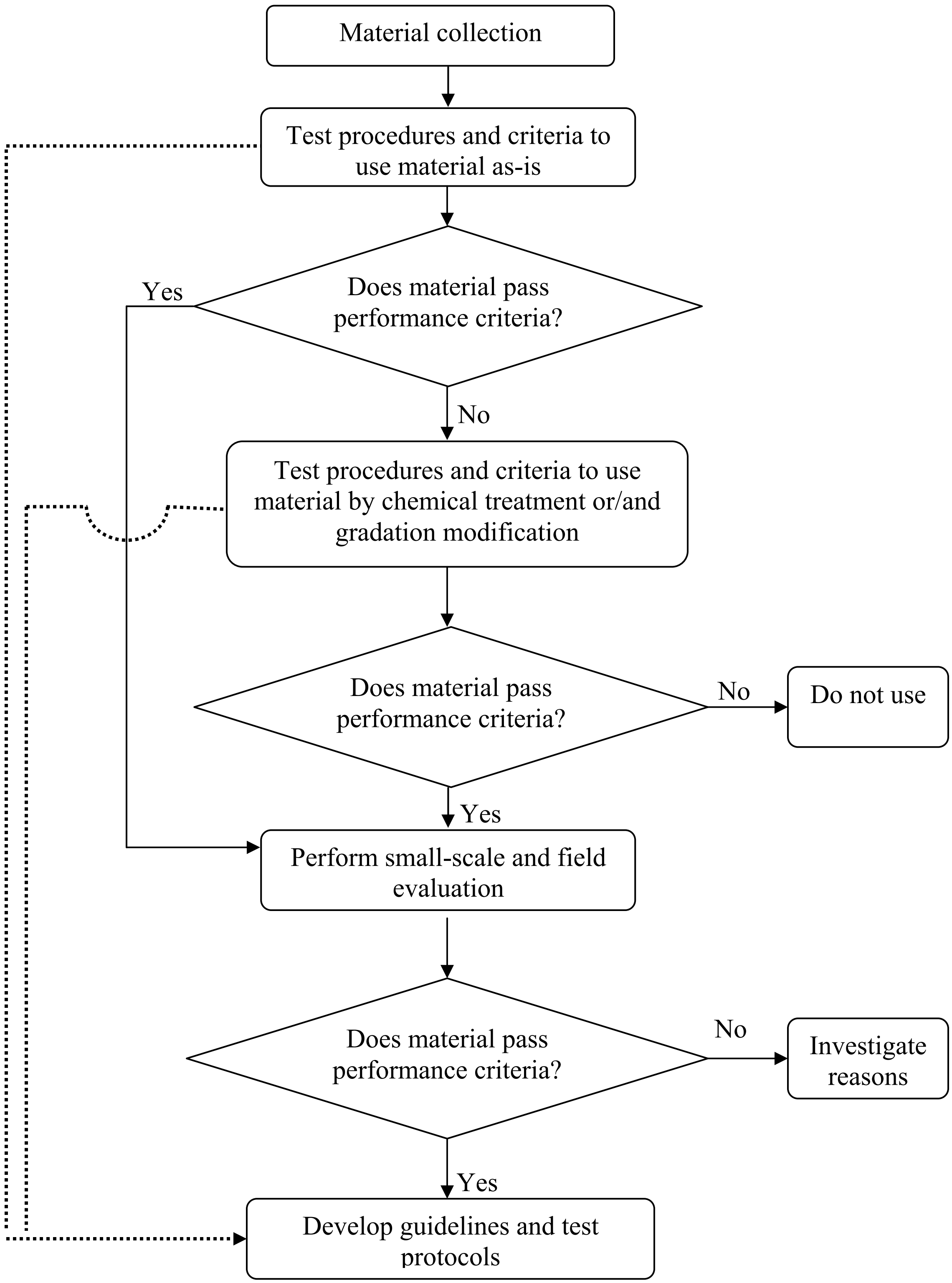



**Figure 1.1 - Flow Chart of Research Activities**

performance-based criteria, the feasibility of treating the material with minimum amount of stabilizer was pursued.

The next step consisted of the validation of the outcomes of the previous step. Small-scale simulation and field tests were used to validate the effectiveness of proposed remediation process under different environmental conditions. Finally, the results from all laboratory and field tests were analyzed and used to develop the guidelines and test protocols.

## Organization of Report

This report consists of seven chapters. Chapter 2 contains the background and information searched from the previous work done on locally available base materials for the purpose of road construction.

Chapter 3 outlines the research and test procedures for characterizing low-quality base materials. The topics discussed in that chapter are procedures for different laboratory testing programs, which include index property tests, compression tests, repeated load triaxial tests, free-free resonant column tests, tube suction tests and small scale tests.

Chapter 4 presents information and results of common base materials from different parts of Texas. The topics discussed in that chapter are the description of the materials investigated, laboratory testing programs mentioned Chapter 3, the reasons for the materials being considered low-quality or marginal and the remedial measures with evaluation. Also, included in that chapter are the results from small scale tests and field monitoring tests.

Chapter 5 presents the structural analysis and cost evaluation. For structural analysis, the topics include the sensitivity study in order to understand the parameters that influence the performance of the pavement in a low-volume road the most, and the determination of equivalent base thickness for low quality materials. For cost evaluation, several factors such as material cost, construction cost and transportation cost are discussed to comprehensively evaluate costs between the uses of the low-quality material from a local pit and the high-quality material from a distance source.

Chapter 6 provides the guidelines and test protocols for using low quality materials on low-volume roads.

Chapter 7 contains the summary and conclusions of the research as well as recommendations for changing TxDOT policies and future study.

# Chapter 2

# Background and Information Search

## Introduction

The performance of a pavement depends on many factors such as the structural adequacy of the pavement, the properties of the materials used, traffic loading, climate conditions and the construction method. For flexible pavements, the quality of base material is one of the most important factors. Previous research has found that much of the distress that flexible pavements experience can be traced to problems encountered in the base materials. Local materials may be out-of-specification with respect to the standard specifications for roadway base/subbase. Under the current TxDOT specification (Item 247), a base material can be considered out-of-specification for a variety of reasons (inadequate gradation, inadequate plasticity, inadequate strength etc.). In many cases, local base supplies miss the standard specification by a small margin. Since the criteria in Item 247 are experienced-based, some of the parameters used to classify a base material may be less significant than others. With appropriate treatment or structural design, many of these out-of-specification materials can perform adequately for low-volume roads (Arora et al., 1986; Greening and Rolt, 1997; Cook and Grourley, 2003). These materials should be capable of providing low-cost base and subbase in roads that are subjected to low traffic levels but high axle loads (Bullen, 2003).

As the first task of the research project, an extensive literature review on the use of locally available materials for roadway base or subbase was conducted. The results from the information search are documented in this chapter.

## History and Current Status of Using Local Materials

During the past two decades, a large number of research projects have been conducted throughout the world to utilize the locally available base materials. Table 2.1 shows examples of the use of non-standard local materials in low volume roads in different places in the world.

**Table 2.1 - Examples of the Use of Non-Standard Materials in Low-Volume Roads (Cook and Gourley, 2003)**

| Material | Location | Climatic Environment | Material Characteristics | Utilization |
|---|---|---|---|---|
| Calcrete | Botswana | Semi-arid | Low particle strength<br>Low compacted strength<br>Poor grading<br>High plasticity | Roadbase: Revised specifications developed for both sealed and unsealed shoulder designs. Successfully used as roadbase with acceptable performance (0.3 x 106 ESAL) for materials with soaked CBR > 35 % and PI<30 if shoulders are sealed. |
| Laterite | Malawi | Seasonally wet tropical | Low particle strength<br>Low compacted strength<br>Poor grading<br>High plasticity | Roadbase: Construction procedure modified to allow traffic to run on roadbase for one rainy season before proof rolling, shaping and sealing in the following dry season. All sites well drained and with crown-height at least over 1m. |
| Marl | Belize | Wet humid tropical | Low particle strength<br>Poor grading | Roadbase and sub-base: Embankment construction (600-750 mm of fill) used throughout due to seasonally high watertable. Only non-plastic or slightly plastic materials selected. Controlled heavy compaction used to lock material and achieve >98% MDD. Good maintenance regime adopted including regular clearing of drains and unsealed shoulder maintenance. |
| Basalt | Botswana | Sub-tropical | Crushed material (with added fines) passed specification criteria; but had demonstrably poor in-service durability | Roadbase: Addition of plastic (active) fines to improve the grading along with modification using too low a percentage of lime (lime also suspect i.e. inactive) led to early failure due to moisture interaction/volumetric change in the road base material. Unsealed shoulder design. |
| Weathered Basalt | Botswana | Sub-tropical | Ripped weathered (Grade III +) basalt selected. Grading out of recommended specification; PI < 12 and soaked CBR >55 | Roadbase: Normal construction methodology adopted. 1m embankment and sealed shoulders. |
| Coral | Papua New Guinea | Wet humid tropical | Low particle strength<br>Poor grading<br>( including oversize)<br>High plasticity | Roadbase: Modified specification based on the requirement of high compaction giving dense layers (max. 150mm). Selection of appropriate compaction plant vital (a function of grading and PI). |
| Cinder Gravels | Ethiopia | Semi Arid | Low particle strength and high porosity<br>Poor grading | Roadbase: Procedures developed to control selection; mechanical stabilization with ash fines and selection of appropriate compaction plant vital. |

Studies conducted by the UK Department for International Development (DFID) and others have shown that, with appropriate design, the use of local materials can play a crucial role in terms of cost saving, pavement performance, resource management and environment protection (Cook and Gourley, 2003; Bullen, 2003).

The Ministry of Works and Communications of Botswana (MOWC) and the UK Transport Research Laboratory (TRL) carried out a research program on the performance of calcrete (caliche) road base materials in the Kalahari region of southern Africa between 1978 and 1993 (Greening and Rolt, 1997). Based on the results from that study, Cook and Gourley (2003) proposed an evaluation procedure as a decision making tool in the context of using sub-standard materials for low volume roads.

Potential calcrete sources were defined as being highly variable in character and frequently out of standard specification in terms of gradation, plasticity, particle strength, and moisture susceptibility. A capacity to breakdown under compaction was also noted. Since there were no similar calcrete roads existed to use for performance data gathering at that time, additional laboratory tests were conducted to quantify the moisture susceptibility and its impact on compacted strength. The evaluation showed significant uncertainty as to the long-term performance due to sensitivity of fine calcrete to wetting. In the light of this significant risk, it was decided that mechanical and chemical stabilization options could be included in further studies. A recommendation for long-term trials was made, incorporating both stabilized and un-stabilized calcrete bases. The trials comprised of four sections with un-stabilized calcrete and one section each of lime, cement and mechanically stabilized fine calcrete. Construction and in service performance were monitored for 13 years. It was found that the four types of calcrete as road base could be recommended within the defined road environment, such as natural environment factors, project-related factors and design response factors. Appropriate specifications and guidelines for use were drawn up by Cook and Gourley (2003).

Another study was also conducted on the same stretch of road to develop performance models. The primary measure of pavement performance considered was the volume of traffic that the pavement was able to carry before reaching a defined "failure" condition at which rehabilitation was required. The design traffic for the road was taken as 0.5 million ESALs (equivalent single axle loads). But in the experiment, many of the trial sections did not reach the terminal condition. The performance models were developed relating rut depth to traffic volume (Greening and Rolt, 1997). A survey of seven contracts in that region indicated that the cost of constructing a calcrete base was about 85% less as compared with hauled-in crushed stone. The use of sand as subbase material also resulted in considerable savings. Most of the calcrete materials were suitable for use in the subbase but the use of the abundant sources of Kalahari sand resulted in savings in haulage costs. Savings of approximately £34,000 per kilometer was expected when local material was available adjacent to construction sites in the region (Greening and Rolt, 1997).

A research study was conducted on a highway in northern Belize to investigate the suitability of local calcareous materials, known as marls, for road bases (Woodbridge, 1999). The marls comprised of high-purity carbonate materials containing mainly silt-sized particles and fall outside the gradation, plasticity, and strength specifications normally required for bases. Despite the good performance of existing marl pavements in Belize and Mexico, there were concerns about the low wet strength, poor gradation, and relatively high plasticity of the marls. Subsequently, the TRL carried out a full laboratory investigation followed by a field trial in 1978, using three marls substituted for the crushed stone base used in a major project.

The gradation of the stockpiled marls was outside the recommended gradation envelope for mechanically stable natural gravel. Gradations determined on marl samples taken after compaction were even finer grained than the stockpiled marl samples, and therefore, further outside the recommended gradation envelope. The marls contained a high proportion of fines, but did not show high plasticity. The plasticity index of the material exceeded 6, and the liquid limit exceeded 25. Under these criteria, the marls were of marginal quality as base materials. The un-soaked California bearing-ratio (CBR) values of the marls were very high and comparable to the crushed stone but their soaked CBR values were much lower. Considering the cost savings, the Santa Cruz marl was selected for stabilization with 5% cement. The performance of the cement-stabilized marl base was excellent. A number of cracks developed in the early years did not increase. The low values of rut depth and deflection testified to the high strength of the base. In 1992, field samples yielded unconfined compressive strength (UCS) values averaging 1300 psi (9 MPa) for the fresh stabilized marl. It was also found that the material became more water resistant. This result indicated that it was possible to use a much wider range of marls if they were stabilized.

Another example concerning cost benefit is from Northeastern Thailand where the most available local materials were lateritic soil, and gravel and silty sand. The place had encountered the problem of material deficiency for many years, especially crushed rock for base and subbase course. A research was conducted on the use of local materials as base course for low volume road design and construction (Ruenkrairergsa, 1980). For the relatively low traffic volume of most routes and for the problem of material shortage associated with the financial status of the country, an approach using the local materials was applied to the design and construction of the road network in this area. The approach mainly included:

- Conducting researches on local materials and their stabilization to develop the more suitable specifications.
- Determining how to use different local materials on the basis of the new specifications
- For fined grained or high PI lateritic soil, 3% to 5% of cement is adopted to achieve an unconfined compressive strength of 250 psi which is related to a CBR value of more than 100% in this case.

In the United States, a large number of studies have been conducted on low volume roads. The first comprehensive guideline for low volume road design and construction was developed by Arora et al. (1986) for the Federal Highway Administration (FHWA). Considerations to traffic, environmental factors, subgrade preparation, compaction and curing were also provided in that guideline. Hall and Bettis (2000) summarized and compared all available design procedures used in the United States for low volume roads. Their study indicated that in many cases, “normal” design methods often provide substantial, and perhaps unwarranted, structural sections for low volume roads, and comprehensive low-volume road design procedures were needed.

In Texas, more than 60% of base materials used in roadway construction during the period from September 1, 2005 to August 31, 2006 are classified as Grade 4 defined by Item 247. Even though in some instances Grade 4 was used to strengthen the specifications, in most cases the Grade 4 was used to relax some of the requirements of Item 247. These locally available materials have been used not only for low volume roads but also for major roads in some districts.

Perceived difficulties of using sub-standard locally available materials still exist. A number of factors combine to pose a major challenge to the implementation of them. These factors include:

- **Standards and Specifications**. Insufficient research has been carried out to justify changes in the current standards and specifications which are usually conservative and seek to establish material property limits which will provide materials of undoubted quality. Where research has been carried out, limited funding is made available for effective dissemination and implementation of changes is often inadequate.

- **Engineering Uncertainty**. There is still reluctance to use sub-standard materials and related mix/structural design and construction technology because of a perceived risk of problems or even failure.

Comprehensive guidelines or specifications for the use of sub-standard locally available materials for roadway base and subbase need to be developed. These guidelines or specifications should cover the issues on material characterization, stabilizer selection and application, cost-benefit analysis, construction QA/QC and initial road performance monitoring as well as pavement design incorporating the concept, defined road environment, as called by Cook and Gourley (2003).

## Factors Affecting Strength, Stiffness and Permanent Deformation

The structural integrity of a flexible pavement section is controlled by several parameters. In most classical structural design programs (such as FPS19 or Texas Triaxial), the design thickness of the layers are (directly or indirectly) estimated based on the criteria that the stresses at the interfaces of the hot mix and base and the base and subgrade are low enough so that the cracking and rutting will not be an issue. The traffic volume is also a major consideration. For a given traffic condition, the thicker the layers overlying the base, the thicker the base layer and the stiffer the subgrade are, the lower the base layer stresses will be. This indicates that not only the quality of base should be considered in the decision to use local materials, the stiffness of the subgrade and the thickness of the hot mix should also be considered.

Assuming that the untreated local materials have lower moduli than those hauled-in, several strategies can be followed to replace hauled-in materials with local ones. These strategies include:

1. Use the local base materials but as a thicker layer,
2. Use the same thickness of local base but thicken the hot mix asphalt and/or improve the stiffness of the subgrade layer,

3. Place the local material on top of the subgrade but cover it with a thinner layer of hauled-in, high-quality base, and
4. Mix the local material with limited amount of imported high quality aggregate.

By adding chemical additives to the local material, its strength, stiffness, resistance to permanent deformation, can be improved provided it is placed on the appropriated subgrade. An extremely strong mix is not desirable because of potential for cracking, and a weak mix will impact the structural capacity of the road. As the concentration of appropriately selected additives increases, the strength and modulus of the stabilized material generally increase as well. Different materials are impacted differently with the type and concentration of the additives. The optimum moisture content (OMC) and the maximum dry density (MDD) are also impacted by the addition of the stabilizers. A thorough investigation of the behavior of the mixtures with the change in material and type of stabilizer should be considered.

Since the economy is of big concern in this project, most probably the best solution is to add just enough additives to improve the local material to act similar to the imported higher-quality base, rather than resorting to a strong and stiff stabilized layer.

**Aggregate Shape and Size**

The aggregate particle shape is characterized by three different properties: angularity, form, and surface texture. Angularity expresses the sharpness or roundness of the aggregate corners. Form expresses the dimension of the aggregates, and texture refers to the small scale asperities. Lekarp et al. (2000) showed that gravel (rough particle) had a higher resilient modulus than the crushed limestone. But many other researchers believe that crushed aggregate with angular to sub-angular shaped particles provides better load spreading properties and a higher resilient modulus than uncrushed gravel with sub-rounded or rounded particles. Barksdale and Itani (1989) studied the influence of aggregate shape and surface characteristics on aggregate rutting. They concluded that blade shaped crushed aggregate is slightly more susceptible to rutting than the other types of crushed aggregate.

**Compaction**

The resilient response of a base material is affected by the degree of compaction, degree of saturation, moisture content during compaction, and method of compaction (Nazarian et al., 1996). Thompson (1989) stated that for a given degree of saturation, soils compacted to the maximum dry density yield higher resilient moduli. Resilient moduli are greater on the dry side of optimum than on the wet side. If allowed to rest before testing, the specimens compacted at higher degree of saturation exhibit a significant increase in strength due to the thixotropic effect. This effect is significant on specimens compacted on the wet side, as compared to the dry side of the optimum. As such, the degree of saturation plays a major role in the resilient response of granular materials subjected to repeated loading (Nazarian et al., 1996).

## Dry Density

Hicks and Monismith (1971) found the effect of density to be greater for partially crushed than for fully crushed aggregates. They found that the resilient modulus increased with relative density for the partially crushed aggregate tested, whereas it remained almost unchanged when the aggregate was fully crushed. They further reported that the significance of changes in density decreased as the fines content of the granular material increased.

Barksdale and Itani (1989) reported that the resilient modulus increased markedly with increasing density only at low values of mean normal stress. At high stress levels, the effect of density was found to be less pronounced. At densities above the optimum value, the resilient modulus is not very sensitive to density.

Resistance to permanent deformation in granular materials under repetitive loading appears to be highly improved as a result of increased density. Barksdale (1972) studied the behavior of several granular materials and observed an average of 185% more permanent axial strain when the material was compacted at 95% instead of 100% of maximum compaction density. Allen (1973) reported an 80% reduction in total plastic strain in crushed limestone and a 22% reduction in gravel as the specimen density was increased from Proctor to modified Proctor density. For rounded aggregates, this decrease in strain with increasing density is not considered to be significant, as these aggregates are initially of a higher relative density than angular aggregates for the same compaction effort.

## Fines Content

Studies demonstrating the variation in response of granular materials subjected to repeated axial stresses indicate that the fines content (percent passing No.200 sieve) can also affect the resilient behavior. Hicks and Monismith (1971) observed some reduction in resilient modulus with increasing fines content for the partially crushed aggregates tested, whereas the effect was reported to be the opposite when the aggregates were fully crushed. The variation of fines content in the range of 2-10% was reported to have a minor influence on resilient modulus. Yet, a dramatic drop of about 60% in resilient modulus was noted by Barksdale and Itani (1989), when the amount of fines increased from 0 to 10%.

## Gradation and Grain Size

Kolisoja (1997) showed that for aggregates with similar grain size distribution and the same fines content, the resilient modulus increased with increasing maximum particle size. As the size of the particle increases, the particle to particle contact decreases resulting in less total deformation and consequently higher stiffness. Thom and Brown (1988) concluded that uniformly graded aggregates were only slightly stiffer than well-graded aggregates. They further indicated that the influence of gradation on the permanent deformation depends on the level of compaction. Lekarp et al. (2000) argued that the effect of gradation on permanent deformation was more significant than compaction, with the highest plastic strain resistance for the densest mix.

## Moisture Content

The amount of moisture present in most untreated granular materials has been found to influence the resilient response of the material in both the laboratory and the in-situ conditions. Studies of the behavior of granular materials at high degrees of saturation have showed that the resilient modulus is highly dependent on moisture content, with the modulus decreasing with growing saturation level (Lekarp et al., 2000).

Dawson et al. (2000) studied a range of well-graded unbound aggregates and found that below the optimum moisture content stiffness tends to increase with increasing moisture level, apparently due to development of suction. Beyond the optimum moisture content, as the material becomes more saturated and excess pore water pressure is developed, the effect changes to the opposite and stiffness starts to decline fairly rapidly. As moisture content increases and saturation is approached, positive pore pressure may develop under rapid applied loads. Excessive pore pressure reduces the effective stress, resulting in diminishing permanent deformation resistance of the material. The combination of a high degree of saturation and low permeability due to poor drainage leads to high pore pressure, low effective stress, and consequently, low stiffness and low deformation resistance.

Thompson and Naumann (1993) reported the results from repeated load triaxial tests on the crushed stone from the AASHTO Road Test at varying degrees of saturation. In all cases, the samples experienced a substantial increase in permanent deformation after soaking. It was suggested that one reason for the observed increase was development of transient pore pressures in the soaked samples.

## Stress State

The resilient modulus increases considerably with increasing confining pressure and sum of principal stresses (Lekarp et al., 2000). An increase of about 50% in resilient modulus was observed by Smith and Nair (1973) when the sum of principal stresses increased from 10 psi to 20 psi.

Compared to confining pressure, deviator or shear stress is said to be much less influential on resilient modulus of the material. The accumulation of axial permanent strain is directly related to deviator stress and inversely related to confining pressure. Several researchers have reported that permanent deformation in granular materials is principally governed by some form of stress ratio consisting of both deviator and confining stresses.

Lekarp and Dawson (1997) argued that failure in granular materials under repeated loading is a gradual process and not a sudden collapse as in static failure tests. Therefore, ultimate shear strength and stress levels that cause sudden failure are of no great interest for analysis of material behavior when the increase in permanent strain is incremental.

## Material Characterization

Any material not wholly in accordance with the specification but can be used successfully either in special conditions, or because of climatic characteristics, or recent progress in road techniques or after having been subject to a particular treatment is defined as non-standard and nontraditional material (Cook and Gourley, 2003).

Currently, in Texas, any base material that does not meet the requirements of TxDOT Item 247 considered out-of-specification. These requirements are presented in Table 2.2. Besides soil gradation, the main requirements are the liquid limit, plasticity index (PI) and compressive strength.

**Table 2.2 – 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 | |
| [illegible] in. | | 10-35 | - | - | |
| 3/8 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<br>Lateral pressure 0 psi<br>Lateral pressure 15 psi | | 45<br>175 | 35<br>175 | -<br>- | As shown on the plans |

A nationwide study commissioned by the Federal Highway Administration (NCHRP, 2000) indicates that rutting and fatigue cracking of flexible pavements can be attributed to the poor performance of base and sub-base layers (see Table 2.3).

**Table 2.3 - Contributing Factors to Base Related Distress**

| Type of Distress | Contributing Factor | Possible Related Test Parameter |
|---|---|---|
| Fatigue Cracking (Alligator Cracking) | Low modulus<br>Improper gradation<br>High fines content<br>High moisture level<br>Lack of adequate particle angularity and surface texture.<br>Degradation under repeated loads | Resilient modulus<br>Gradation and fines content<br>Moisture susceptibility<br>Density |
| Rutting | Low modulus and strength<br>Low density of base material<br>Improper gradation<br>High fines content<br>High moisture level<br>Lack of adequate particle angularity and surface texture.<br>Degradation under repeated loads. | Resilient Modulus<br>Permanent Deformation<br>Triaxial Testing-angle of internal friction, cohesion<br>Gradation<br>Fines content |

Marginal materials that could be considered for use in the base or sub-base layers can be grouped within a five tier systems (TRL, 2002):

- Group I - Hard Rocks: usually comprising materials that require crushing and processing but retaining properties that result in the material does not fully meeting the requirements of a crushed stone base.
- Group II - Weak rocks: materials derived from weakly cemented, poorly consolidated or partially weathered parent deposits.
- Group III: Natural Gravels: transported and residual soils and gravels not meeting the minimum material standards for natural gravel base.
- Group IV: Duricrusts: indurated or partially indurated soils not meeting the minimum material standards for natural gravel base.
- Group V: Manufactured materials: include a range of man-made materials that could effectively be re-processed as granular pavement materials.

A study conducted by Cook and Gourley (2003) gave examples of materials that would commonly be associated with each of the groups and provided a summary review of typical nonstandard aspects within each group. The potential characteristics of naturally occurred nonstandard granular materials (Group III as defined above) and their likely problems are shown in Figure 2.1.

| | | Non-Standard Natural Granular Material Groups | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Alluvial Sands | Alluvial Clayey Sand Deposits | Aeolian Sand Deposits | Colluvial Deposits | Alluvial Gravel Deposits | Volcanic Pyroclastics | Residual Clayey Sand Deposits | Residual Gravel Deposits |
| Primary Specification Criteria | High PI Fines | | ■ | | ■ | | | ■ | |
| | Low Particle Strength | | | | | | ■ | | |
| | Poor Grading | ■ | ■ | ■ | ■ | | ■ | ■ | ■ |
| | Poor Durability | | | | | | ■ | | ■ |
| | Poor Particle Shape | ■ | ■ | | | ■ | | ■ | |
| Additional Impacting Criteria | High Mica Content | | | | | | | | |
| | High Water Absorption | | | | | | ■ | | |
| | High Variability | | | | ■ | ■ | | | |
| | In-service Deterioration | | | | | | | | |
| | Low PI Fines | | | ■ | | ■ | | | |

■ Potential likely problem

**Figure 2.1 - Non-Standard Material Groups and Their Likely Problems (Cook and Gourley, 2003)**

Besides the consideration on the fundamental parameters shown in Tables 2.2 and 2.3, determination of the aggregate toughness and the changes in gradation due to dynamic and static loads are also of importance to characterize the base materials. These changes can be measured in the laboratory under the British test procedures (British Standard 812-112:1990) of Aggregate Crushing Value (ACV) and Aggregate Iimpact Value (AIV). The ACV gives a relative measure of the resistance of air-dried aggregates to crushing under a gradually applied compressive load, whereas the AIV gives the relative strength of aggregates against impact loading. Both the tests, ACV and AIV, are carried out on aggregates passing ½ in. sieve and retained on 3/8 in. sieve. A detailed discussion about the tests is presented in Chapter 3.

Figure 2.2 illustrates a test protocol employed to assess the current TxDOT test procedures. The first step, Preliminary Testing, consists of establishing the gradation, index properties and the hardness of 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 (Geiger et al., 2006).

## Stabilizer Selection

In order to achieve specified properties, raw base materials usually require treatment or stabilization with calcium-based additives such as cement, lime and fly ash. Each of these materials must be properly designed to determine the appropriate additive to achieve the desired improvement or modification. The major properties and functions of these three additives are given in Table 2.4.

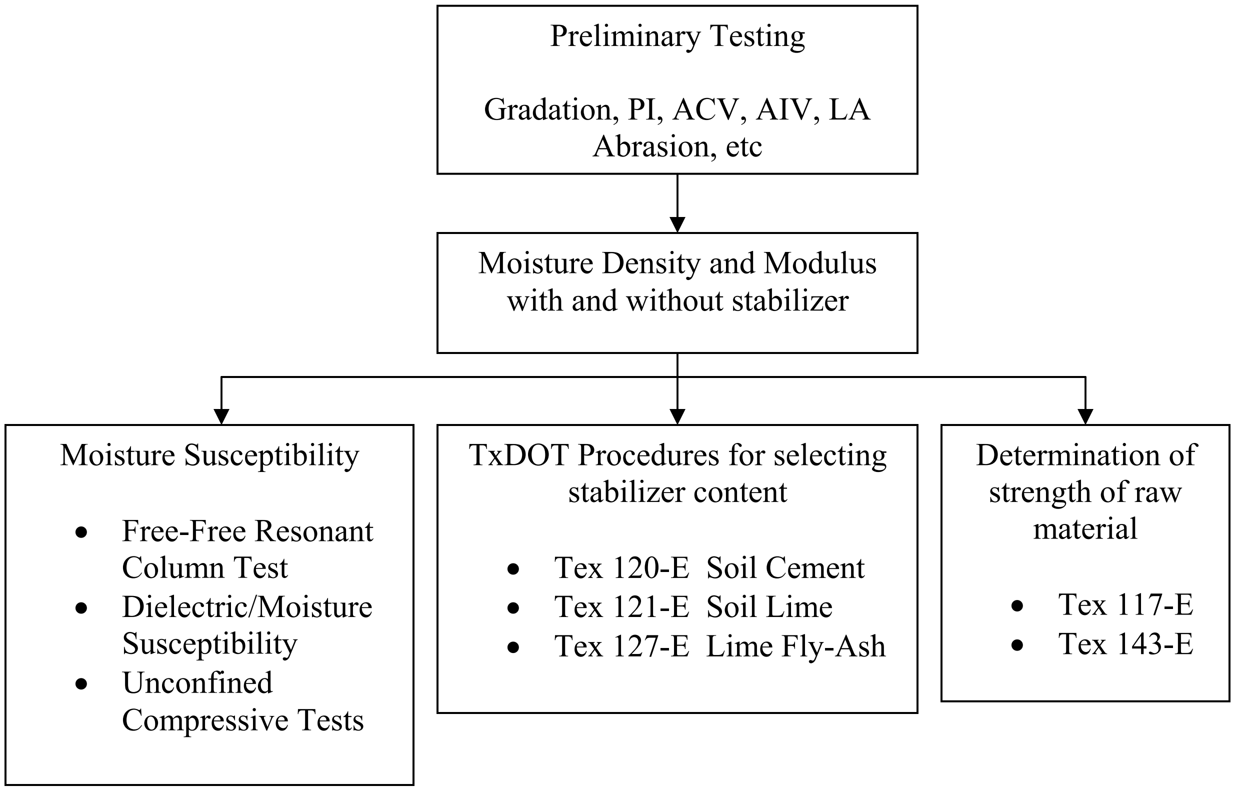



**Figure 2.2 - Test Protocol for Characterizing Base Materials (Geiger et al., 2006)**

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 most common improvements achieved through stabilization include (Army TM 5-822-14, 1994):

- Reducing plasticity index
- Reducing swelling potential
- Increasing durability and strength
- Waterproofing the soil
- Drying of wet soils
- Conserving aggregate materials
- Reducing cost of construction

The selection of the type and determination of the amount 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. Relatively larger amount of additives are used to significantly improve the strength, stiffness and durability.

**Table 2.4 - Summary of Conventional Stabilizers (Yoder and Witzcak, 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 $\geq$ 12) | Plastic clay soils |
| Maximum Dry Density | Varies | Decreases | Increases, however delay compaction time decreases density |
| Optimum Moisture | Varies | Increases | Decreases |
| Plastic Index | Decreases | Decreases | Decreases |
| Plasticity | Decreases | Decreases | Decreases |
| Strength | Increases | Moderate increment | Increases, however curing temperature and delay time affects strength |

The selection of stabilizer type also depends on the type of material present and their location in the pavement structure (Terrel et al., 1979). Table 2.5 provides varying stabilization methods for different materials.

**Table 2.5 - 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 |

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 2.3 demonstrates a 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.

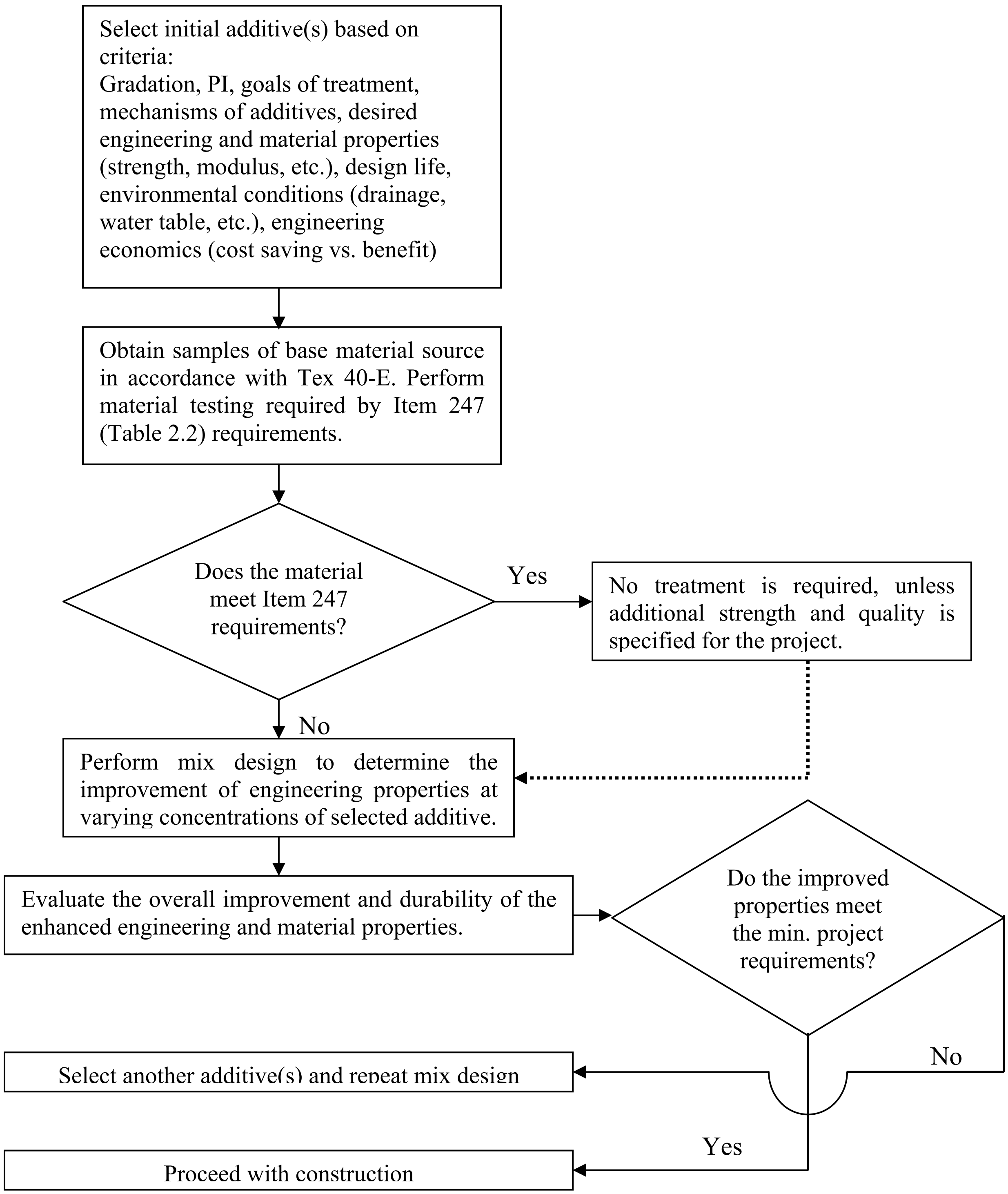



**Figure 2.3 - Flowchart for Base Treatment (TxDOT, 2005)**

## Performance Evaluation with Small Scale Test

Since actual field testing is expensive, and since the level of control in placing the section cannot be practically achieved in the field, it is necessary to have a method that can verify the outcomes from standard lab tests and link these outcomes to those from the field tests. A small-scale laboratory testing system (see Chapter 3 for details) can be inexpensive alternative to obtain realistic performance results for comparison. This system is easier to control the quality of the subgrade, and to vary the moisture content from the as-built or as-compacted condition in the laboratory than in the field.

Amiri (2004) used this small-scale testing system to evaluate the performance of a number of bases under different moisture conditions to verify the Texas Triaxial Method. In that study, it was demonstrated that the results from field sections can be reasonably well simulated with this system. The results from different experiments can be compared to demonstrate the effectiveness of the remediation proposed based on lab testing on each local base material under different environmental conditions.

After the specimen is prepared, the load applied and the resulting displacement are monitored, measured, digitized, and saved for further analysis. Three moisture conditions are considered: 1) Base and subgrade at optimum moisture content, 2) Base under optimum condition and the subgrade saturated, and 3) Base and subgrade are saturated. These moisture conditions should cover the best and most severe conditions that a pavement is subjected to. Typical test results anticipated in terms of load-induced permanent deformation are shown in Figure 2.4. Typical results for several common base materials in Texas are shown in Table 2.6 and Table 2.7. These results demonstrate the utility of these tests.

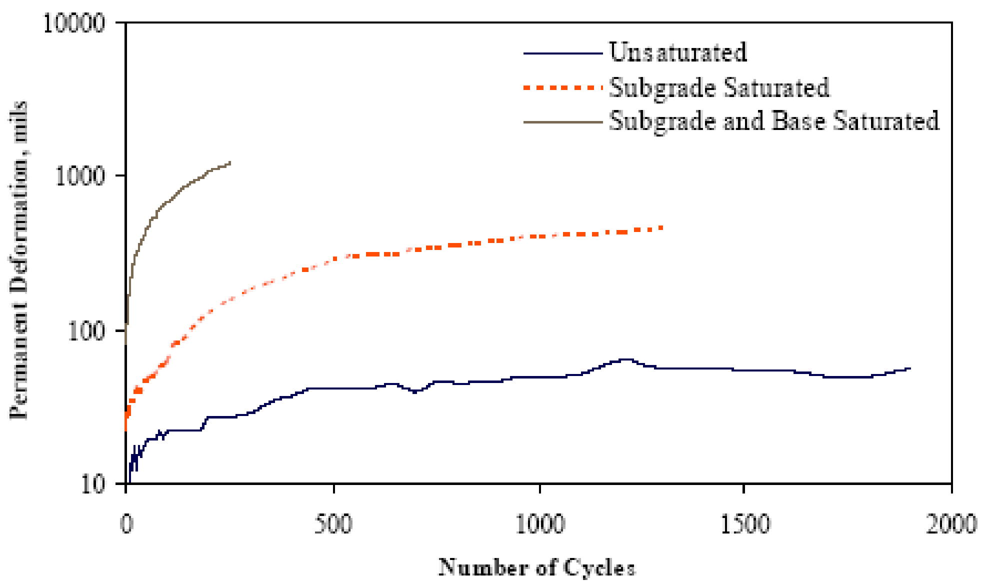



**Figure 2.4 - Variations of Rutting with Number of Load Cycles**

**Table 2.6 - Load Carrying Capacity of Different Bases on Sandy Subgrade**

| Moisture Condition | Deformation, mils | Load, lbs | | | | | |
|---|---|---|---|---|---|---|---|
| | | Caliche | Limestone | | Sandstone | Springdale | Uncrushed Gravel |
| | | | Grade 1 | Grade 2 | | | |
| Optimum | 50 | 1475 | 1060 | 863 | 741 | 1057 | 2265 |
| | 100 | 2475 | 2058 | 1612 | 1503 | 2050 | 4292 |
| | 150 | 3000 | 2995 | 2247 | 2286 | 2980 | 6079 |
| Subgrade Saturated | 50 | 746 | 958 | 578 | 842 | 685 | 833 |
| | 100 | 1447 | 1961 | 1105 | 1729 | 1604 | 1545 |
| | 150 | 2103 | 3008 | 1579 | 2660 | 2756 | 2137 |
| Base Saturated | 50 | 428 | 423 | 507 | 491 | 582 | 293 |
| | 100 | 910 | 878 | 978 | 1021 | 1287 | 554 |
| | 150 | 1448 | 1365 | 1413 | 1590 | 2113 | 783 |

**Table 2.7 - Load Carrying Capacity of Different Bases on Clayey Subgrade**

| Moisture Condition | Deformation, mils | Load, lbs | | | | | |
|---|---|---|---|---|---|---|---|
| | | Caliche | Limestone | | Sandstone | Springdale | Uncrushed Gravel |
| | | | Grade 1 | Grade 2 | | | |
| Optimum | 50 | 1178 | 1514 | 920 | 656 | 1152 | 892 |
| | 100 | 2410 | 2873 | 1831 | 1277 | 1958 | 1803 |
| | 150 | 3694 | 4079 | 2735 | 1863 | 3842 | 2734 |
| Subgrade Saturated | 50 | 469 | 974 | 634 | 251 | 546 | 828 |
| | 100 | 810 | 1597 | 1098 | 427 | 974 | 1323 |
| | 150 | 1023 | 1868 | 1393 | 530 | 1285 | 1484 |
| Base Saturated | 50 | 232 | 435 | 378 | 284 | 479 | 250 |
| | 100 | 461 | 803 | 785 | 505 | 903 | 484 |
| | 150 | 687 | 1106 | 1221 | 664 | 1272 | 701 |

## Cost Benefit

Walls et al. (1998) developed a process for evaluating the total economic worth of a usable pavement project segment by analyzing initial costs and discounted future cost, such as maintenance, user, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment.

The cost of construction including the future maintenance and rehabilitation of pavement largely depends upon the type of material used beneath the road surface and the volume of traffic flow. This is especially true in the case of roads constructed with low-quality marginal base materials. The study conducted by Greening and Rolt (1997) revealed that the haulage costs for high quality base materials are the main factors for increasing the project cost. The study also revealed that the cost of a constructed natural gravel base (calcretes) was about 85% less as compared with hauled-in crushed stone. A study conducted by Woodbridge (1999) stated that the potential savings in costs are realized if locally available materials are used instead of crushed stone for base. To emphasize the use of locally available, low quality materials for realizing the cost benefit, Arora et al. (1986) compared the cost analysis of treated road materials with the non-stabilized materials. They concluded that soil stabilization treatments are the best technique to realize potential benefits

The materials used for base and subbase in western Qeensland, Australia which were deemed nonstandard and later performed satisfactorily in service were Winton sandstone, Silcrete, Kopi limestone, Ferricrete, Calcrete, Loams (clayey), Ridge gravels, loams (sandy), and White rock. The use of white rock on the Cunningham Highway was not thought to be possible, as grading specifications could not be met and the white rock degraded under compaction. The material breakdown, however, allowed the manufacture of a well-graded, strong, impermeable product that performed well in service. The use of the locally available aggregate instead of a river gravel blend was reported to have saved more than Aust$ 15,000/km on the basis of the savings in transport costs alone. As a result, specifications were written to allow better use of the material.

# Chapter 3

# Testing Procedures

## Introduction

The suitability of using marginal materials in base construction is often assured by performing different laboratory tests which help in determining their physical and engineering properties, as well as compaction characteristics. This chapter gives a brief description of the laboratory tests that were performed to assess the quality of these materials.

## Index Properties

Soil index tests are conducted to determine the reasons for the material being considered out-of-specifications/low-quality. The similar information about higher-quality hauled-in or processed material is also obtained for comparison.

### Sieve Analysis and Gradation

The particle size analysis is conducted as per Tex 110-E to determine the composition of particles in a material sample.

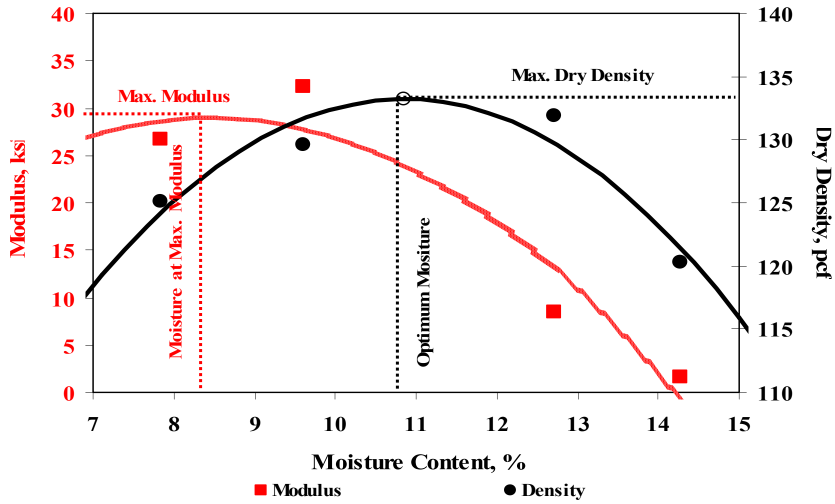
### Atterberg Limits

The liquid limit, plastic limit and plasticity index of soils are used with other soil properties to correlate with engineering behavior such as compressibility, permeability, compactibility, shrink-swell and shear strength. These tests are conducted as per Tex 104-E, Tex 105-E and Tex 106-E.

### Moisture Density

Tex-113-E procedure is carried out to determine the relationship between water content and the dry mass (density) of base materials. At least four 6 in. (diameter) by 8 in. (height) specimens are prepared to determine the optimum moisture content at which maximum dry unit weight can be achieved.

To estimate the variation in modulus and strength with moisture, a specimen is first tested with the Free-Free Resonant Column device (FFRC, proposed Tex-149-E) for modulus and then subjected to Unconfined Compressive Strength (UCS) tests. Typical relationships between moisture content and dry density (unit weight), modulus and UCS are shown in Figure 3.1. These relationships are useful to estimate the impact of moisture on strength and stiffness.



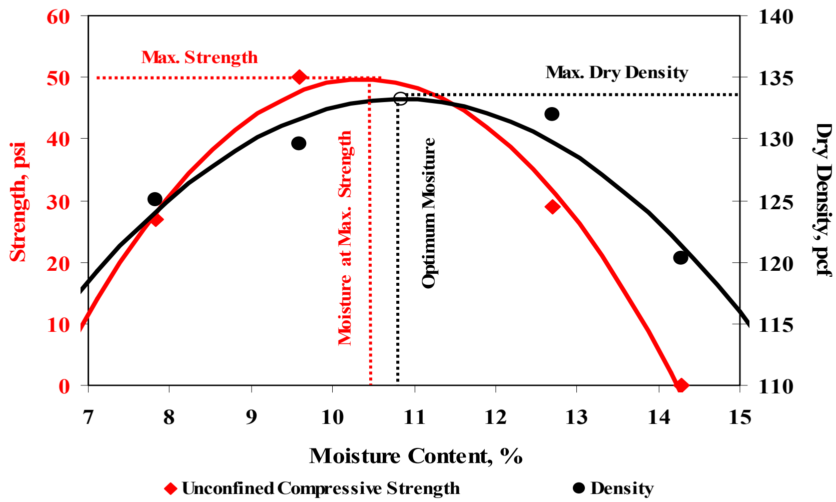




**Figure 3.1 - Relationships between Moisture Content, Dry Density, Modulus and Strength**

## Aggregate Quality Assessment

The aggregate toughness and the changes in gradation due to dynamic and static loads are measured in the laboratory under the British test procedures using Aggregate Impact Value

(AIV) and Aggregate Crushing Value (ACV). The AIV (See Figure 3.2a) of aggregates is performed as per BS 812-112 and the ACV (See Figure 3.2 b) is conducted as per BS 812-110.

a) AIV Test

b) ACV Test

**Figure 3.2 - Aggregate Impact Value and Aggregate Crushing Value Test Setups**

For AIV, a coarse aggregate sample contained within a mold is used to perform the test. 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 No. 8 sieve (2.36 mm); and, expressed as a percentage of the initial sample weight. The AIV is given by the following equation:

$$\mathrm{AIV} = \frac{M_2}{M_1} \times 100\% \tag{3.1}$$

Where $M_1$ is the mass of test specimen and $M_2$ is the mass of the specimen passing No. 8 sieve.

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 aggregates passing ½ in sieve and retained on 3/8 in. sieve 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 (400 kN) 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 No. 8 sieve (2.36 mm) sieve, is represented as a percentage of the original mass. This percentage is the aggregate crushing value (ACV). The ACV is also calculated by using Equation (3.1).

**Triaxial Compression Test**

A conventional triaxial test using more than three specimens is normally used to determine the shear strength parameters of soil. The test comprises shearing a single specimen to failures at several elevated lateral pressures, while measuring the corresponding deviator stresses at which the failures occur. Two types of compression tests are advocated to determine the strength parameters: Texas Triaxial Test (Tex-117-E) and Standard Triaxial Test (Tex-143-E).

Tex-117-E is a well-known procedure in Texas, in which the specimens are subjected to capillary wetting prior to being tested. Procedure Tex-143-E is a revised Tex-117-E adopted by TxDOT. The major difference of Tex-143-E from Tex-117-E is that the specimens are cured only for 24 hours at room temperature without subjecting capillary conditioning.

**Repeated Load Triaxial Test**

Besides traffic load and environmental conditions, pavement performance is related to the resilient modulus and permanent deformation of each layer in a pavement. One of laboratory methods advocated for determining these two parameters is the repeated load triaxial test as per AASHTO T-307. The procedure and setup for this test are shown in Figures 3.3 and 3.4.

The testing system consists of a loading frame with a crosshead mounted hydraulic actuator. A load cell is attached to the actuator to measure the applied load. The specimen is housed in a triaxial cell where confining pressure is applied. As the actuator applies the repeated load, specimen deformation is measured by a set of Linear Variable Differential Transducers (LVDT's). A data acquisitions system records all data during testing.

***Resilient Modulus***

`

The resilient modulus determined from the repeated load triaxial test is defined as the ratio of the repeated axial deviator stress to the recoverable or resilient axial strain:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{3.2}$$

Where $M_r$ is the resilient modulus, $\sigma_d$ is the deviator stress, and $\varepsilon_r$ is the resilient (recoverable) strain in the vertical direction (see Figure 3.5).

The load cycle duration, when using a hydraulic loading device, is 1 second that includes 0.1 second load duration and a 0.9 second rest period.

The test is started by applying 1000 repetitions of a load equivalent to a maximum axial stress of 15 psi at a confining pressure of 15 psi. This is followed by a sequence of loadings with varying confining pressures and deviator stresses. A combination of confining pressures of 3, 5, 10, 15 and 20 psi and deviatoric stresses of 1, 2, 3, 5, 6, 9, 10, 15, 20, 30, and 40 psi are used. To utilize the results in design, a model can be described in the form of:

$$E = k_1 \sigma_c^{k2} \sigma_d^{k3} \tag{3.3}$$

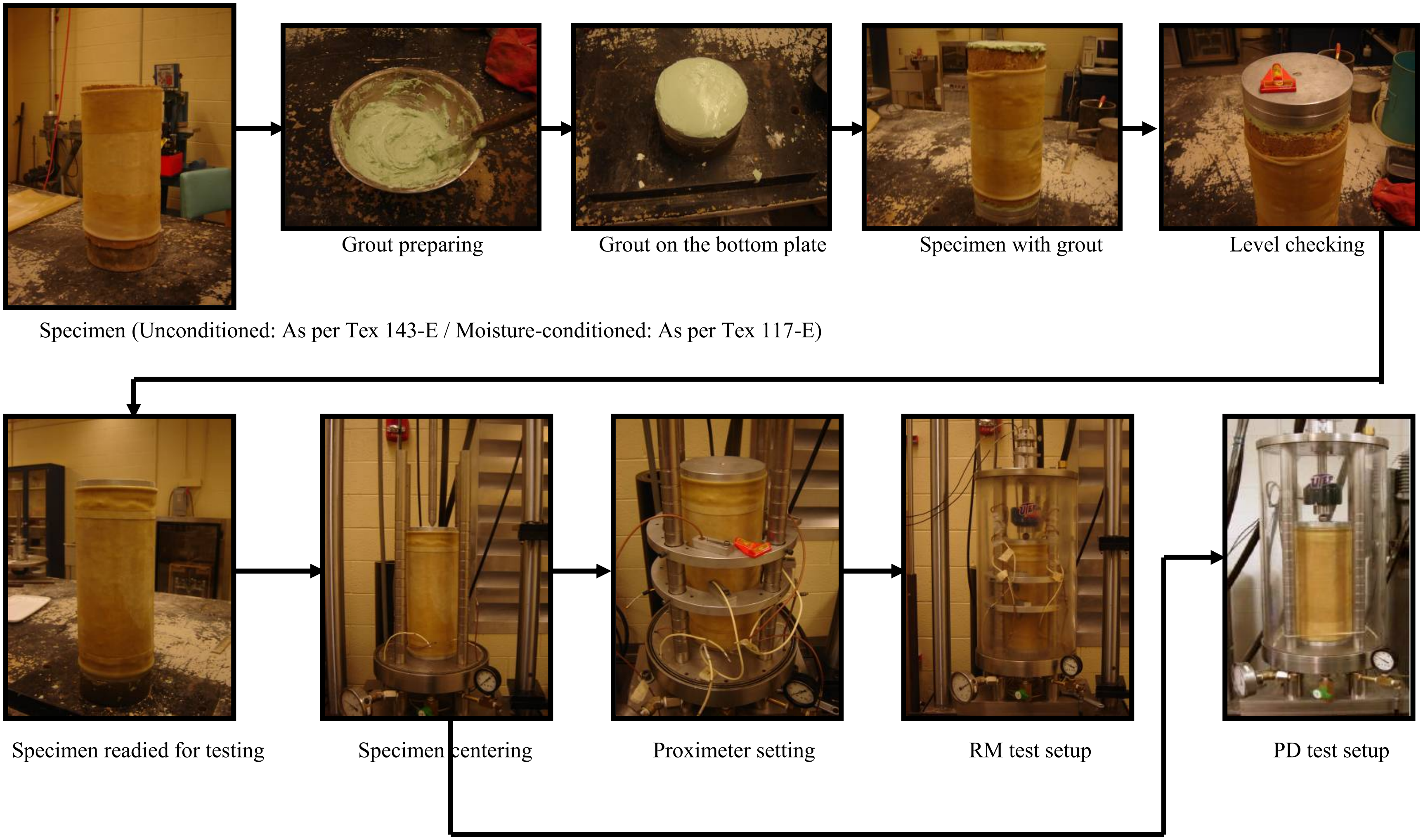



**Figure 3.3 - Procedure for Repeated Load Triaxial Test**

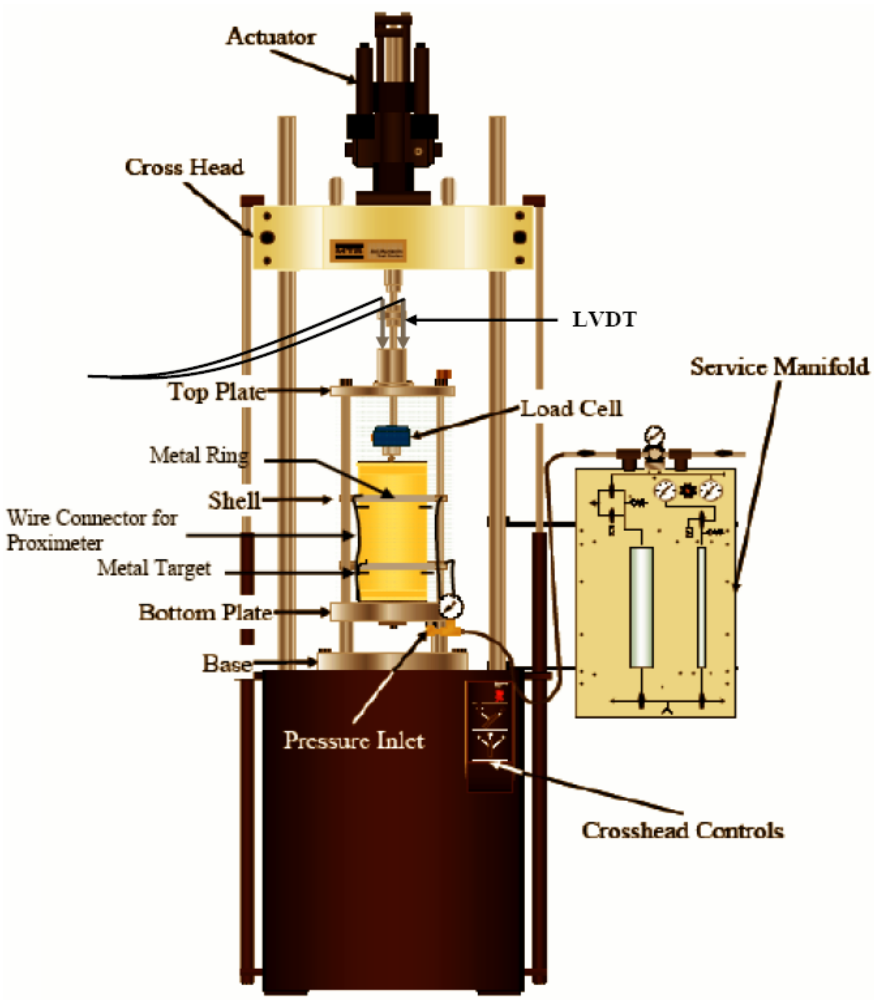



**Figure 3.4 - Repeated Load Triaxial Load Schematic**

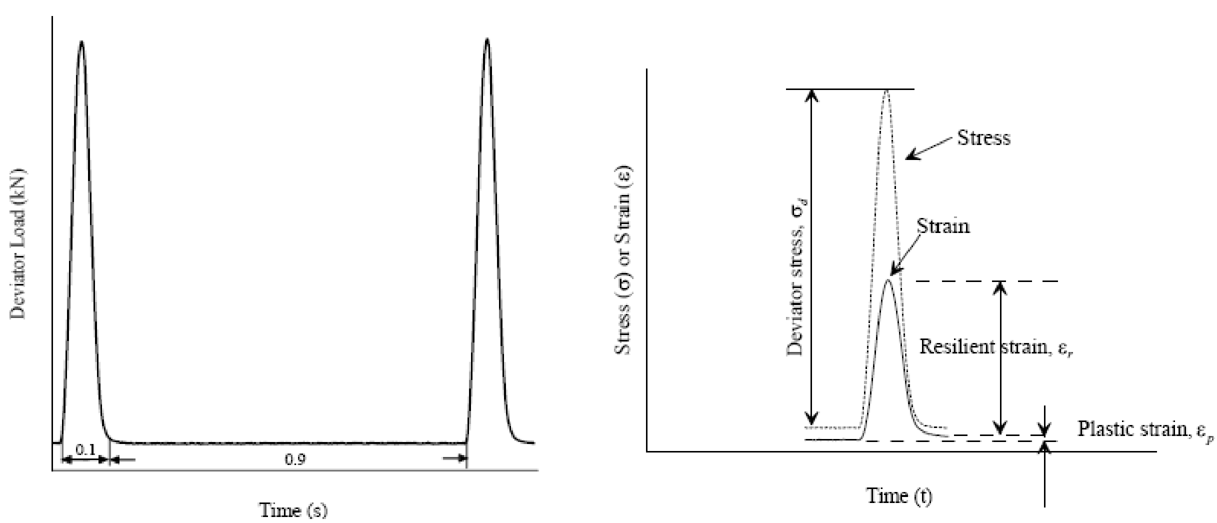



**Figure 3.5 – Stress and Strain of One Load Cycle**

where $k_1$, $k_2$ and $k_3$ are coefficients determined from laboratory resilient modulus tests and $\sigma_c$ and $\sigma_d$ are the confining pressure and deviatoric stress, respectively. The advantage of this type of models is that it is universally applicable to fine-grained and coarse-grained materials.

A typical result from one test is shown in Figure 3.6. For base materials in Texas, the resilient modulus increases as the confining pressure increases and decreases as the deviatoric stress increases.

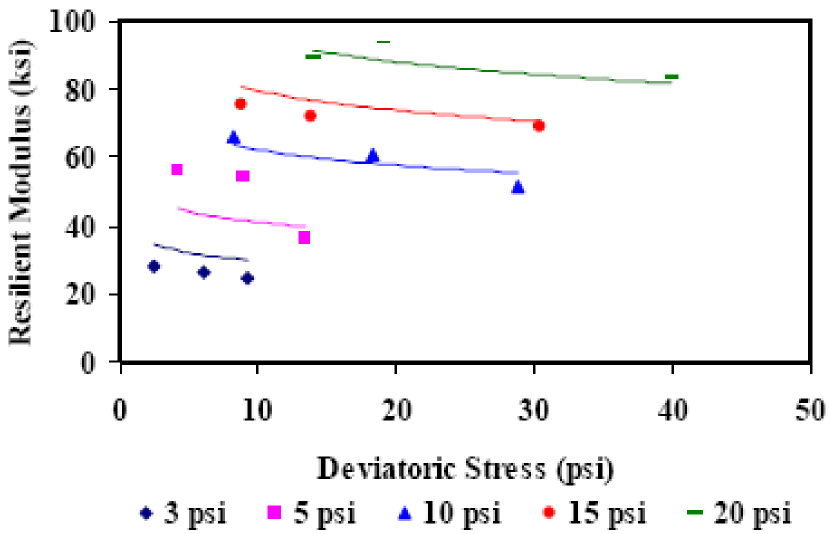



**Figure 3.6 - Typical Result Obtained from Resilient Modulus Test**

The resilient modulus is carried out at two different conditions: (a) Unconditioned (specimen prepared as per Tex 143-E) and (b) Moisture-conditioned (specimen prepared as per Tex 117-E). The results obtained from resilient modulus tests are discussed in Chapter 4.

***Permanent Deformation***

The prediction of permanent deformation or rutting for material characterization is usually based on the assumption that the permanent strain is proportional the resilient strain (Huang, 2004) by

$$\varepsilon_p(N) = \mu\, \varepsilon_r\, N^{-\alpha} \tag{3.4}$$

where $\varepsilon_p(N)$ is the plastic or permanent strain due to a single load application, e.g., at the N$^{th}$ application; $\varepsilon_r$ is the elastic or resilient strain at the 200th repetition; N is the load application number; $\mu$ and $\alpha$ are the permanent deformation parameters. The total permanent strain can be obtained by integrating Equation 3.4.

$$\varepsilon_p = \int_0^N \varepsilon_p(N)\, dN = \varepsilon_r\, \mu\, \frac{N^{1-\alpha}}{(1-\alpha)} \tag{3.5}$$

Theoretically, Equation 3.5 indicates that a plot of log ($\varepsilon_p$) vs. log (N) results in a straight line. So that the slop of the straight line $S = 1 - \alpha$, or $\alpha = 1 - S$. The intercept of the straight line at N = 1, $I = \varepsilon\mu/(1 - \alpha)$, or $\mu = IS/\varepsilon$. Figure 3.7 shows an example for the relationship from an actual permanent deformation test.

A confining pressure of 15 psi and a deviatoric stress of 15 psi are adopted during the test. These stress levels are selected based on a stress analysis conducted for the base layer in a flexible pavement model. The permanent strain is determined at the 1,000th or 10,000th repetition, depending upon the number of cyclic stress levels employed. The resilient strain is obtained at the 200th repetition. The parameter $\alpha$ is determined from the slope of the straight segment from the 200th repetition to the 1,000th repetition as shown in Figure 3.7 and $\mu$ is determined from the intercept the segment (not the entire line) at N =1.

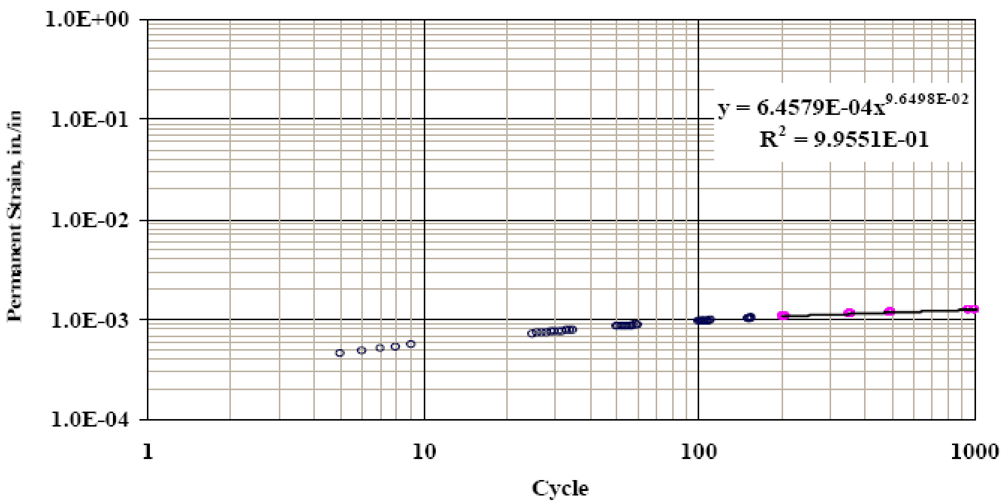



**Figure 3.7 -Typical Result Obtained from Permanent Deformation Test**

**Free-Free Resonant Column (FFRC) Tests**

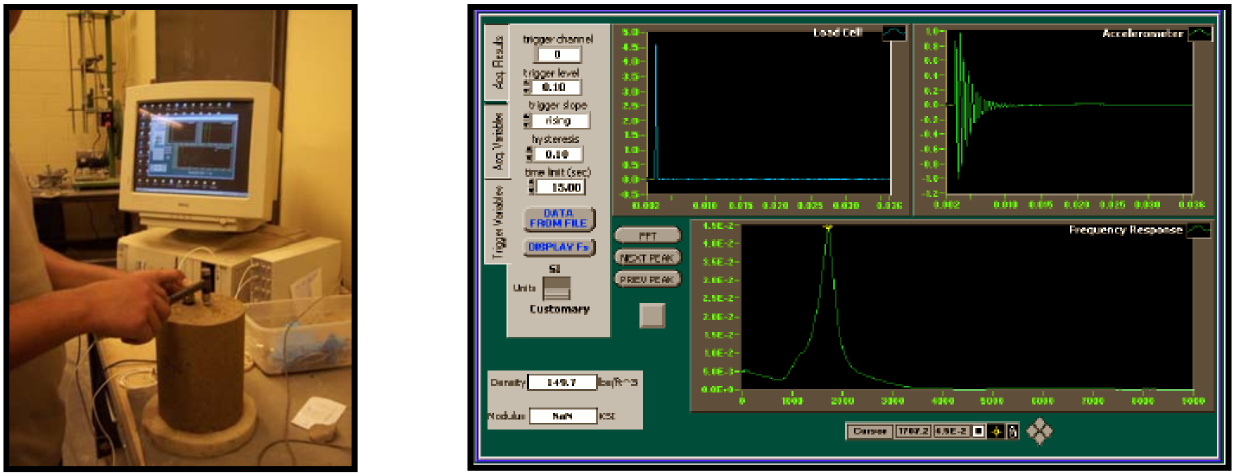
Unlike the resilient modulus test, the FFRC test is nondestructive and easy for day-to-day use in the laboratory. The FFRC test is applied to all cylindrical specimens used in this study.

The principle of the FFRC method is based on the determination of the fundamental resonant frequency of longitude vibration of a specimen. From the resonant frequency, Young's modulus the specimen can be calculated by ((Richard et al., 1970)

$$E = \frac{\gamma}{g}\left(2\, f_c\, L\right)^2 \tag{3.6}$$

Where L is the length of the specimen, $f_c$ is the resonant frequency of the fundamental-mode vibration of the specimen related to compression waves, $\gamma$ are the weight of the specimen and g is the gravity acceleration.

A setup for FFRC test and typical records from a FFRC test are shown in Figure 3.8




a) FFRC Test Setup b) Records from a FFRC Test

**Figure 3.8 - Free-Free Resonant Column (FFRC) Test**

**Moisture Susceptibility**

Tube Suction Test (TST, proposed Tex-144-E) is used for assessing the moisture susceptibility of materials. The moisture susceptibility is evaluated based on the mean surface dielectric value of a compacted specimen after a 10-day capillary soak in the laboratory. A percometer is employed to measure the dielectric values of specimens (see Figure 3.9). The surface dielectric value of a ten-day capillary-conditioned specimen is dependent on the suction and permeability of the aggregate layer and the state of bonding water that accumulates within the aggregate matrix. Permeability is an especially important issue in moisture damage mechanisms, such as frost heave, where water must be able to rapidly respond to changes in suction within the pavement structure. The state of bonding of water describes the structuring of the water molecules within the soil or aggregate matrix.

a) Adek Percometer b) Measuring Dielectric Value

**Figure 3.9 - Tube Suction Test**

To perform the test, a specimen prepared at the optimum moisture content is prepared and placed in a 140°F oven for 48 hours. The specimen is then placed on top of a porous stone placed in a water bath for additional eight days. The variations in bulk moisture content and dielectric constant of the specimen with time are also shown in the Figure 3.10. Normally, the moisture content and dielectric constant decrease for the first two days and then increase. The specimen is also tested with the FFRC so that the variation in modulus with moisture can be observed as shown in Figure 3.10.

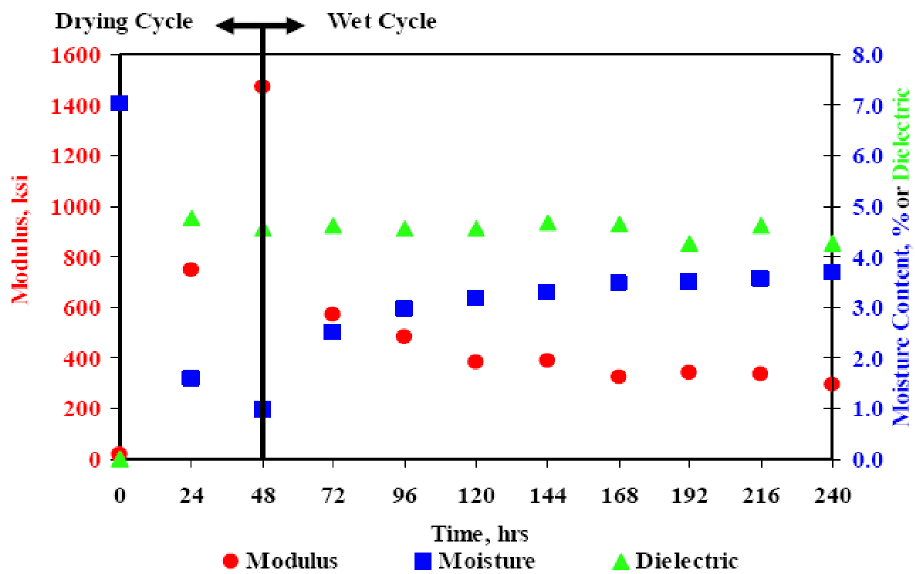



**Figure 3.10 - Typical Variation in Modulus, Moisture and Dielectric Constant with Time**

**Small Scale Test**

To verify the outcomes from the standard laboratory tests presented above, a series of small-scale laboratory tests is carried out on different base materials. Since the level of control in a pavement section is difficult to achieve under the field condition, the small-scale test is an economical alternative to obtain realistic performance results for comparison. For example, it is easy to control the quality of the subgrade, and to vary the moisture content.

The schematic picture of a specimen for small-scale tests is shown in Figure 3.11. Three materials are placed in the tank. From the bottom are pea gravel, subgrade and base.

Each specimen is compacted in a 36 inch diameter cylindrical tank. The tank is a polyethylene sewage pipe with a wall thickness of one inch. The pipe is reinforced with helical loops on the outer surface to counteract the lateral deformation that the pipe undergoes during testing. At the bottom of the tank, a thin layer of silicon and 6-mil thick polyethylene sheet is lined to make the tank water-proof. The wall of the tank should be smooth with a minimum amount of friction.

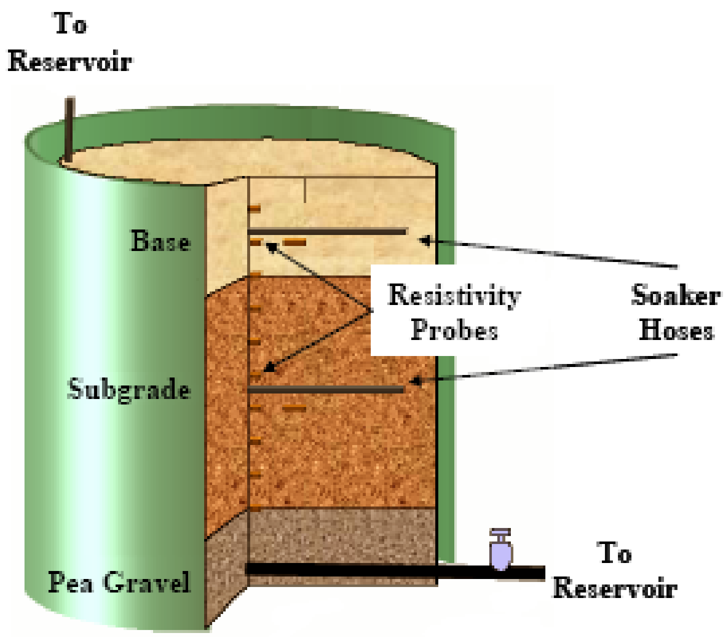
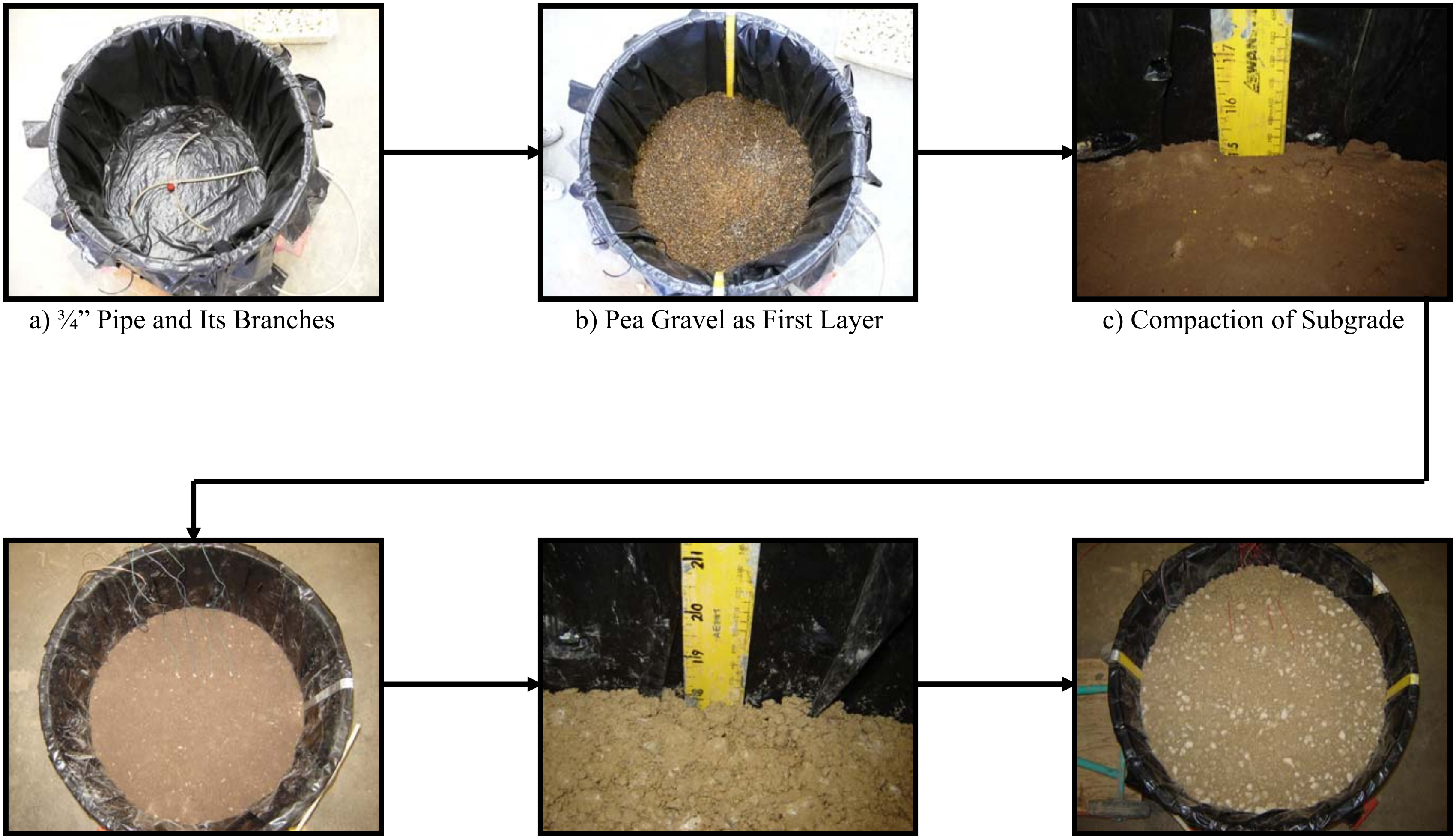



**Figure 3.11 - Schematic Picture of Tank (Amiri, 2004)**

A ¾ in PVC pipe with branches is installed at the bottom of the tank for introducing water to the specimen. The tank is filled with a 3-in.thick layer of pea gravel which can be easily saturated. A 14 in. layer of subgrade is then compacted on top of the gravel layer by placing in 2 in. thick lifts. For each lift, the amount of soil and water necessary to achieve the appropriate moisture and density are calculated. The instrumentation is placed at pre-selected depths in the tank. Before the next lift of soil is placed, the top of each lift is scarified to ensure a continuous mass. A 6-in. thick layer of the base with the desired density is then placed on top of the subgrade. Soaker hoses are placed within the base and the subgrade layers so that water can be introduced to them if necessary. A step-wise procedure to carry out the test is summarized in Figure 3.12.

Modulus and permanent deformation tests are performed on each small-scale specimen on the following dates: (1) three days after construction, (2) after saturation of the subgrade and (3) after saturation of both the base and the subgrade. The modulus tests are carried out with a Portable Seismic Pavement Analyzer (PSPA) and a Dynamic Cone Penetrometer (DCP) as shown in Figure 3.13.

The permanent deformation test is carried out with a 220-kip MTS system. The small-scale specimen is placed right under the frame of a 220-kip MTS system (see Figure 3.14). Two types of loading, cyclic ramp and sinusoidal, are applied to the specimen with the MTS system. The cyclic ramp load is applied to the specimen to measure its strength by increasing load at a rate of 500 lb per minute to a peak and decreasing at the same rate and then maintaining constant for 1 minute. The maximum cyclic load is varied between 700 lb and 11000 lb. The sinusoidal loading is applied with amplitude of 2000 lbs and a frequency of 1 Hz to measure the permanent deformation. As a sample, the two load patterns and the corresponding deflections are shown in Figure 3.15 and Figure 3.16.



a) ¾” Pipe and Its Branches

b) Pea Gravel as First Layer

c) Compaction of Subgrade

d) Resistivity Probes in the Subgrade

e) Compaction of Base

f) Resistivity Probes in the Base

**Figure 3.12 - Different Steps of Filling Tank**

(a) PSPA Test

(b) DCP Test

**Figure 3.13 - Modulus Test on a Small Scale Specimen**


**Figure 3.14 – Permanent Deformation Test on a Small Scale Specimen**

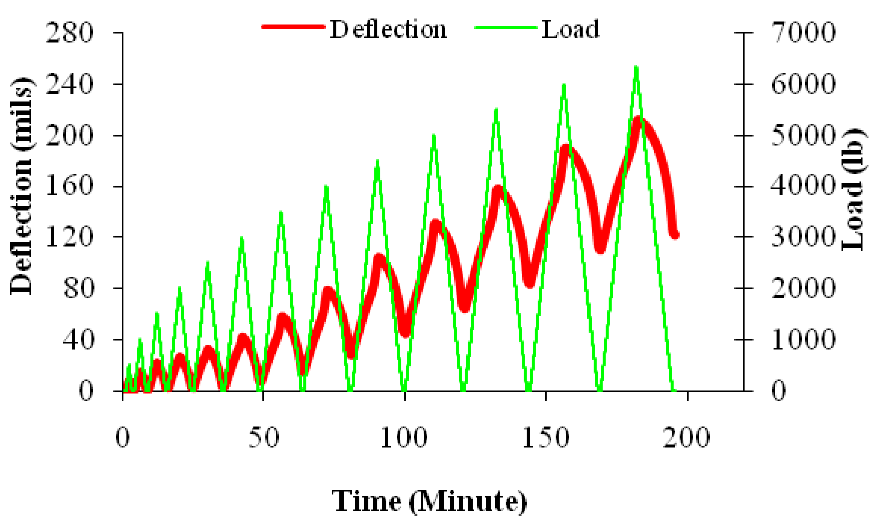



**Figure 3.15 - Typical Pattern and Corresponding Deflections for Cyclic Ramp Loading**

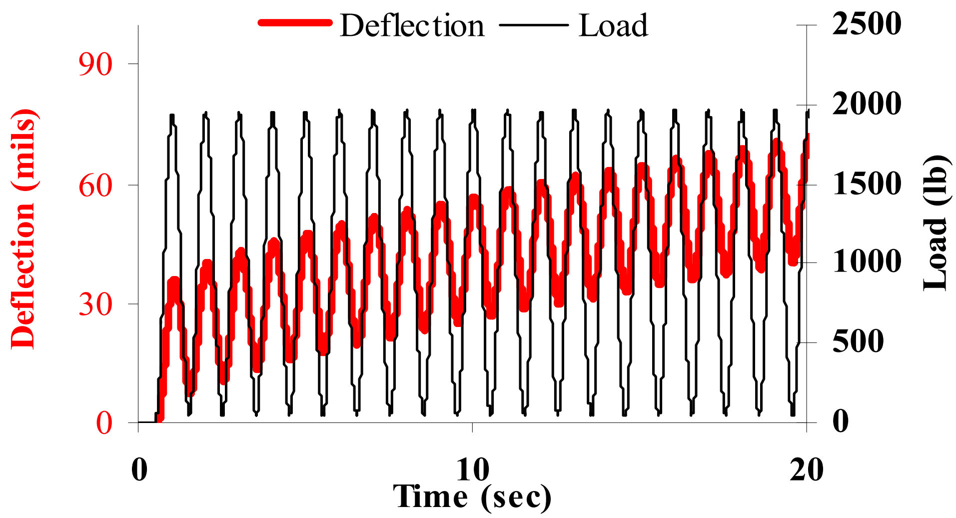



**Figure 3.16 – Typical Pattern and Corresponding Deflections for Sinusoidal Loading**

# Chapter 4

# Results from Laboratory and Field Tests

## Introduction

The laboratory test program as discussed in Chapter 3 was conducted on representative materials collected from different districts. Materials were subjected to various tests to determine their index properties, compaction characteristics, strength, resilient modulus and permanent deformation. Results from these tests are presented in this chapter. In addition, results from field monitoring tests are also included to validate the results from the laboratory tests.

## Material Selection

A survey was conducted to understand the extent of the use of local base materials and to identify the districts that could benefit from the outcome of this study. Responses were received from 19 districts. Beaumont, Lufkin and Houston do not use their local materials at all, since suitable base materials do not exist in these districts. Sixteen districts have used local materials for roadway base/subbase construction. Out of the 16 districts, 14 districts have used local materials both for low volume roads and for major roads. In seven districts, Abilene, Brownwood, Dallas, Lubbock, Odessa, San Angelo and San Antonio, all roadways are constructed with local base materials.

Based on the interaction with the districts and the PMC of the project, nine materials from four districts were selected and used to develop guidelines for using local materials for roadway base construction. These districts are concentrated in the central and north Texas. In addition, El Paso limestone, as our local material, was also tested to help in developing appropriate test protocol and procedures. The sources and rock types of these materials are summarized in Table 4.1.

**Table 4.1 - Sources and Rock Types of Materials Selected**

| District | Rock Type | Quarry |
|---|---|---|
| Abilene | Limestone | Black Lease |
| | Limestone | Old Bobby Noble |
| Brownwood | Limestone | Prater (Medium) |
| | Limestone | Prater (Good) |
| | Limestone | Vulcan |
| El Paso | Limestone | Cemex |
| Lubbock | Rhyolite Tuff | Caddell (High PI) |
| | Rhyolite Tuff | Caddell ( Low PI) |
| San Angelo | Limestone | Lumpkin |
| | Limestone | Turner |

## Laboratory Test Results of Raw Materials (Materials As-Is)

### Index Properties

Table 4.2 summarizes the index parameters for the nine base materials. The gradation curves from all the materials are shown in Figure 4.1 along with the Item 247 limits for acceptable gradation for a Grade 1 base. The gradations and Atterberg limits as well as the classifications of the materials as per Unified Soil Classification System (USCS) and AASHTO are shown in Table 4.2. The San Angelo material is slightly finer than and the Lubbock and the Abilene materials are slightly coarser than the specifications, especially for Sieve No. 40.

Even though TxDOT does not have a requirement for fine content (materials passing Sieve No. 200), the fine content is known to impact the long-term performance of bases. According to findings of TxDOT Project 0-4348 (Gandara et al., 2005), the fine content should be between 5% and 10%. Brownwood "Good" and San Angelo bases contain more than 10% fines and Brownwood 'Medium" and Lubbock less than 5%.

The Liquid Limits (LL) and Plasticity Indices (PI) of the base materials are also shown in Table 4.2. The LL of the Lubbock material is marginally below the limit of 35 required in Item 247. The PI of the Lubbock material is almost twice the level of 10 required by Item 247 while the PI's of the Brownwood Good and San Angelo are close to 10.

### Moisture Density and Related Tests

Moisture-density tests as per Tex-113-E were carried out on the nine materials. The optimum moisture contents (OMC) and maximum dry densities (MDD) as well as UCS and modulus at OMC for the base materials are summarized in Table 4.3. The quality of material impacts their OMC's and MDD's and subsequently result in changes in strength and modulus. The MDD's are higher for the materials that have lower OMC's.

**Table 4.2 – Gradations, Classifications and Atterberg Limits**

| Material Source | | Gradation, % | | | Classification | | Atterberg Limits | | |
|---|---|---|---|---|---|---|---|---|---|
| District | Quarry | Gravel | Sand | Fines | USCS | AASHTO | LL | PL | PI |
| ABL | Black Lease | 62 | 34 | 4 | GW | A-2-4 | 18 | 12 | 6 |
| | Old Bobby Noble | 65 | 34 | 1 | GW | A-2-4 | 16 | 8 | 7 |
| BWD | Prater (Medium) | 55 | 44 | 1 | GW | A-1-a | 20 | 15 | 5 |
| | Prater (Good) | 53 | 33 | 14 | GM | A-2-4 | 26 | 17 | 9 |
| | Vulcan | 50 | 47 | 3 | GW | A-2-4 | 15 | 11 | 4 |
| ELP | Cemex | 55 | 40 | 5 | GW | A-2-4 | 27 | 19 | 8 |
| LBB | Caddell (High PI) | 66 | 33 | 1 | GW | A-2-6 | 34 | 14 | 20 |
| | Caddell (Low PI) | 66 | 33 | 1 | GW | A-2-4 | 26 | 18 | 8 |
| SJT | Lumpkin | 50 | 31 | 19 | GM | A-2-6 | 29 | 18 | 11 |
| | Turner | 63 | 36 | 1 | GW | A-1-a | 7 | 2 | 5 |

**Table 4.3 – Results from Moisture-Density and Related Tests**

| Material Source | | Optimum Moisture Content (OMC), % | Maximum Dry Unit Weight, pcf | UCS Strength at OMC, psi | Modulus at OMC, ksi |
|---|---|---|---|---|---|
| District | Quarry | | | | |
| ABL | Black Lease | 7.3 | 143 | 46 | 28 |
| | Old Bobby Noble | 6.2 | 138 | 46 | 92 |
| BWD | Prater (Medium) | 10.8 | 133 | 49 | 24 |
| | Prater (Good) | 13.3 | 125 | 29 | 27 |
| | Vulcan | 6.6 | 142 | 20 | 22 |
| ELP | Cemex | 6.4 | 143 | 28 | 16 |
| LBB | Caddell (High PI) | 11.6 | 124 | 19 | 9 |
| | Caddell (Low PI) | 10.2 | 114 | 16 | 11 |
| SJT | Lumpkin | 12.7 | 122 | 40 | 28 |
| | Turner | 6.7 | 131 | 34 | 18 |

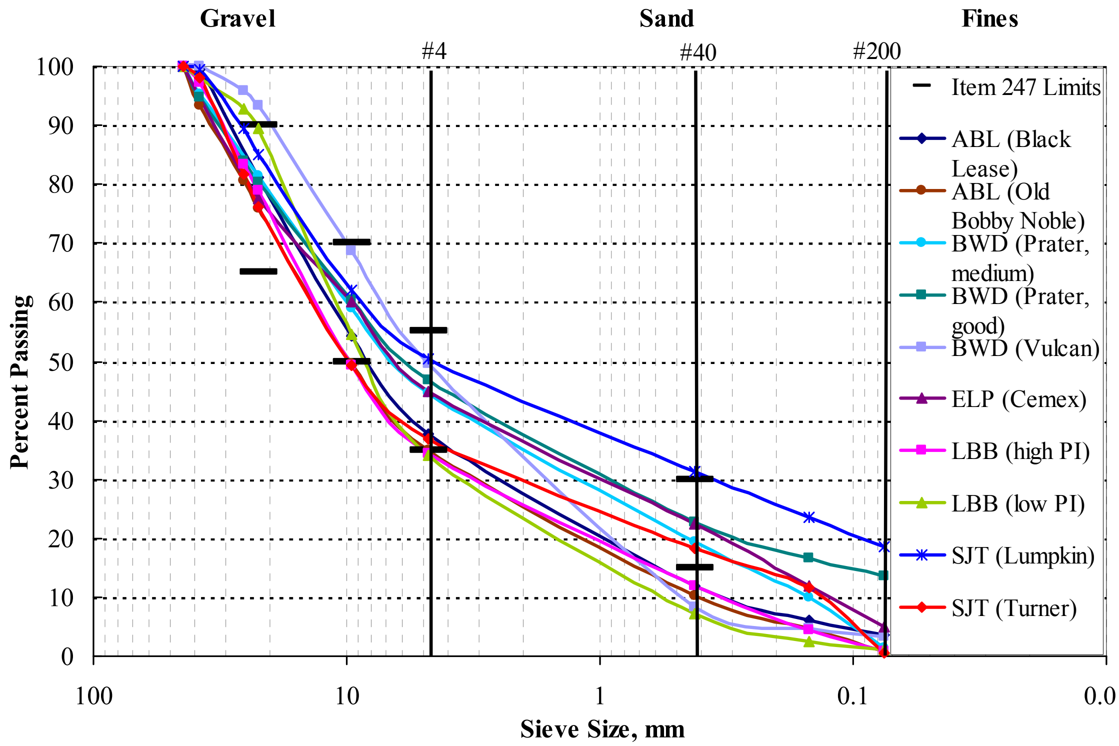



**Figure 4.1 - Gradation Curves from Different Base Materials**

## Triaxial Strength Test

The strengths and the Texas Triaxial classifications of the materials, obtained following Tex-117-E and Tex- 143-E procedures, are summarized in Table 4.4. The newly-developed Tex-143-E procedure is quite similar to Tex-117-E with the major exception that the specimens are cured only for 24 hours and they are not subjected to the moisture capillary saturation.

Except for the materials from Black Lease (Abilene), Turner (San Angelo) and Cemex (El Paso), the rest seven materials are classified as Class 2 or 3. Two other requirements for the material to be classified as Grade 1 are the compressive strengths at the zero and 15 psi lateral pressures as per Tex-117-E). Based on these two requirements, materials from Black Lease (Abilene), Caddell (Lubbock, High PI), and Cemex (El Paso) can be classified as Grade 1.

The strengths from procedure Tex-143-E are also shown in the Table 4.4. For the materials from Cemex (El Paso), Caddell (Lubbock, both High PI and Low PI), Lumpkin and Turner (San Angelo) and Prater-Good (Brownwood), the unconditioned strength (Tex-143-E) are less than or equal to those from moisture conditioned (Tex-117-E), indicting that moisture capillary does not have a detrimental impact on the strength of these materials. On the other hand, for the materials from Black Lease and Old Bobby Noble (Abilene) and Prater Medium and Vulcan (Brownwood), the unconditioned strengths of the specimens with Tex-143-E are greater than those from moisture conditioned specimens (Tex-117-E), indicating the possibility of moisture susceptibility.

**Table 4.4 - Triaxial Compression for Different Base Materials**

| **Base Material** | | **Angle of Internal Friction, degree** | | **Cohesion, psi** | | **Texas Triaxial Class** | | **Strength at Zero psi Lateral Pressure, psi** | | **Strength at 15 psi Lateral Pressure, psi** |
|---|---|---|---|---|---|---|---|---|---|---|
| District | Quarry | Tex 117-E | Tex 143-E | Tex 117-E | Tex 143-E | Tex 117-E | Tex 143-E | Tex 117-E | Tex 143-E | Tex 117-E |
| ABL | Black Lease | 58.8 | 59.8 | 7.9 | 9.6 | 1.0 | 1.0 | 54 | 103 | 255 |
| | Old Bobby Noble | 46.8 | 48.6 | 4.5 | 10.8 | 3.6 | 2.3 | 34 | 80 | 130 |
| BWD | Prater (Medium) | 48.9 | 55.8 | 6.2 | 4.0 | 2.9 | 1.0 | 23 | 52 | 117 |
| | Prater (Good) | 47.6 | 50.3 | 7.2 | 10.5 | 2.9 | 2.1 | 29 | 22 | 120 |
| | Vulcan | 54.8 | 56.8 | 7.0 | 6.9 | 2.4 | 1.0 | 32 | 46 | 180 |
| ELP | Cemex | 58.2 | 59.8 | 9.7 | 7.8 | 1.0 | 1.0 | 62 | 28 | 230 |
| LBB | Caddell (High PI) | 54.9 | 55.7 | 6.7 | 3.6 | 2.5 | 1.0 | 46 | 19 | 198 |
| | Caddell (Low PI) | 54.2 | 60.7 | 8.6 | 2.0 | 2.2 | 1.0 | 42 | 13 | 178 |
| SJT | Lumpkin | 42.9 | 51.7 | 11.7 | 7.1 | 2.6 | 2.0 | 53 | 44 | 133 |
| | Turner | 47.5 | 48.6 | 15.3 | 11.9 | 1.0 | 2.3 | 70 | 66 | 166 |

The results of the tests relevant to Item 247 are summarized in Table 4.5. Based on this table, the El Paso material is classified as Grade 1. The Lubbock (High PI) base does not meet the plasticity, and Texas Triaxial Classification requirements. The Abilene (Black Lease) material does not meet the gradation, whereas the Abilene (Old Bobby Noble) material fails to meet gradation, classification and strength requirements. The three Brownwood mixes do not pass the gradation and strength requirements. The San Angelo (Lumpkin) material does not meet the plasticity and strength criteria, whereas the San Angelo (Turner) material fails to satisfy the gradation as well as the strength requirements.

## Material Considered Out-of-Specification

Table 4.5 summarizes the results from the laboratory tests on the ten materials and compares them with the requirements provided by TxDOT Specification Item 247 for a Grade 1 base. Each number in the parentheses in the table represents that the material is out-of-specification for that particular requirement. Based on Table 4.5, it can be concluded:

- El Paso material passes all requirements for a Grade 1 base.
- Abilene (Black Lease) material passes all requirements for a Grade 1 base except for gradation which is just slightly out-of-limit for No. 40 sieve.
- Materials from Abilene (Old Bobby Noble), Lubbock (High PI and Low PI), Brownwood (Medium, Good and Vulcan) and San Angelo (Lumpkin) are more or less out-of-specification.

Based on the amount of out-of-specification and in consideration of the difference in material sampling at the pit and the errors in specimen preparation and testing, we considered the material from Black Lease in Abilene District and the material from Turner in San Angelo District as Grade 1 materials.

## Remedial Measures

To ensure that the materials classified as out-of-specification can be economically improved to meet the requirements for Grade 1, the following remedial measures were adopted:

- Chemical treatment for Lubbock (High PI), Brownwood “Medium” and Brownwood (Vulcan) materials,
- Gradation modification for San Angelo (Lumpkin) material, and
- Both chemical treatment and gradation modification for Abilene (Old Bobby Noble) and Brownwood “Good” materials.

Since, the properties of two different Lubbock base materials are close to each other; only one of the Lubbock materials was treated with calcium-based additive to observe its impact on strength and stiffness of materials. In addition, El Paso limestone, as our local material, was also treated with different additives to help in developing appropriate test protocol and procedures.

**Table 4.5 - Evaluation of the Results Based on Item 247 for a Grade 1 Base**

| Requirements for Grade 1 Base | El Paso | Lubbock | | Abilene | | Brownwood | | | San Angelo | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Cemex | Caddell (High PI) | Caddell (Low PI) | Black Lease | Old Bobby Noble | Prater (Medium) | Prater (Good) | Vulcan | Lumpkin | Turner |
| Gradation Sieve Size (% cumulative retained) | | | | | | | | | | |
| 1-¾ in. (0%) | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| 7/8 in. (10%-35%) | 23 | 21 | 11 | 19 | 24 | 18 | 20 | (7) | 15 | 24 |
| 3/8 in. (30%-50%) | 40 | (51) | 45 | 46 | 50 | 41 | 39 | 31 | 38 | (51) |
| No. 4 (45%-65%) | 55 | (66) | (66) | 62 | 65 | 55 | 53 | 50 | 50 | 63 |
| No. 40 (70%-85%) | 78 | (88) | (93) | (88) | (90) | 81 | 77 | (92) | (69) | 82 |
| Liquid limit, max. 35% | 27 | 34 | 26 | 18 | 16 | 20 | 26 | 15 | 29 | 7 |
| Plasticity index, max. 10 | 8 | (20) | 8 | 6 | 7 | 5 | 9 | 4 | (11) | 5 |
| Classification, 1.0 | 1.0 | (2.5) | (2.2) | 1.0 | (3.6) | (2.9) | (2.9) | (2.4) | (2.6) | 1.0 |
| Min. compressive strength (as per Tex-117-E) | | | | | | | | | | |
| Lateral pressure at 0 psi: 45 psi | 62 | 46 | (42) | 54 | (34) | (23) | (29) | (32) | 53 | 70 |
| Lateral pressure at 15 psi: 175 psi | 230 | 198 | 178 | 255 | (130) | (117) | (120) | 180 | (133) | (166) |

**Treatment with Chemical Additives**

The decision tree for selecting the appropriate types of additive as per current TxDOT guideline (Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures, 2005) is shown in Figure 4.2. The two main factors considered are the percentage of material passing the No. 200 sieve and the Plasticity Index (PI). Since fines were less than 25% for all the materials and PI was less than 12 except for Lubbock material, lime and cement were selected as major additives. For El Paso material, fly ash was also used as an additional additive. The additive content of 1% by dry weight was adopted for each of individual treatments. The preliminary motivation of using so low additive content was trying to see if a low quality base material (marginally out-of-specification) after such treatment could be used on a low volume roadway and still get a quality foundation layer.

**Modification of Gradation**

Fines content in a base material impacts its properties and long-term performance. However, TxDOT policy does not have a requirement for fines content (materials passing No. 200 sieve). The findings from TxDOT Project 0-4348 suggust that for a quality base material the fines content should be between 5% and 10%. For this reason, gradation modification was applied to Brownwood “Good”, San Angelo (Lumpkin) and Abilene (Old Bobby Noble) materials by adjusting their fines contents from 14% to 7%, from 19% to 5%, and from 1% to 5%, respectively, along with corresponding changes for other sieve sizes. Figure 4.3 shows the gradation curves before and after gradation modification for the three materials.

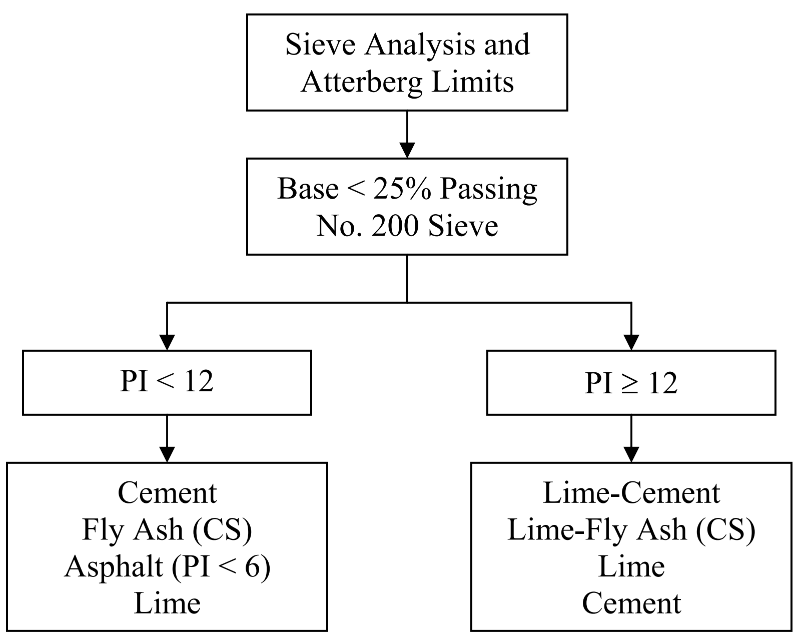



**Figure 4.2 - Decision Tree for Stabilization Selection**

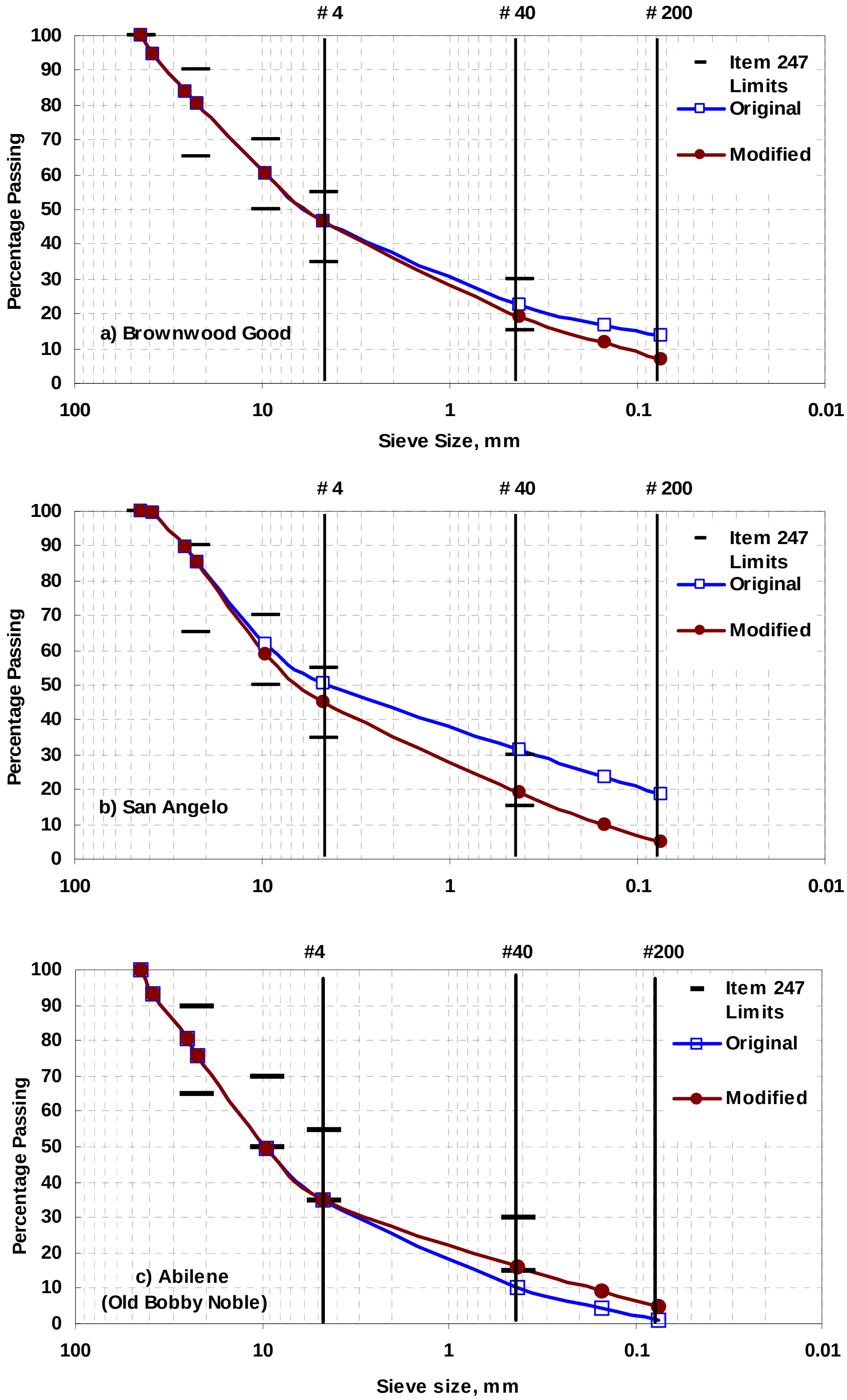



**Figure 4.3 - Gradation Curves of Materials before and after Gradation Modification**

## Laboratory Test Results of Treated or Gradation Modified Materials

### Index Properties

Tests for Atterberg Limits were carried out on the five chemically-treated local materials. Sieve analysis was applied to three gradation-modified materials (Brownwood Good and Abilene Old Bobby Noble) materials were subjected to both chemical treatment and gradation modification).

Table 4.6 summarizes the index parameters obtained for the five base materials before and after treatment. The LL's of all materials met the requirement of Item 247. The PI of Lubbock material decreased from 20 to 14 with 1% lime and to12 with 1% cement, respectively. The PI of the San Angelo material is close to 10 (without chemical treatment). The PI values of two Brownwood materials, which came from the same quarry (source) but different layers, increased after chemical treatment. The reason for this is unknown.

**Table 4.6 - Index Parameters for Materials before and after Treatment**

| Material Source | | | Atterberg Limits | | Constituent, % | | |
|---|---|---|---|---|---|---|---|
| **District** | **Quarry** | **Material Type** | **LL** | **PI** | **Gravel** | **Sand** | **Fines** |
| ABL | Old Bobby Noble | Raw | 16 | 7 | 65 | 34 | 1 |
| | | 1% Cement | 20 | 5 | | | |
| | | New Gradation | Same as for Raw | | 65 | 30 | 5 |
| BWD | Prater (Medium) | Raw | 20 | 5 | 55 | 44 | 1 |
| | | 1% Lime | 29 | 8 | | | |
| | | 1% Cement | 29 | 9 | | | |
| | Prater (Good) | Raw | 26 | 9 | 53 | 33 | 14 |
| | | 1% Cement | 24 | 11 | | | |
| | | New Gradation | Same as for Raw | | 53 | 40 | 7 |
| | Vulcan | Raw | 15 | 4 | 50 | 47 | 3 |
| | | 1% Cement | 17 | 4 | | | |
| ELP | Cemex | Raw | 27 | 8 | 55 | 40 | 5 |
| | | 1% Lime | 12 | 2 | | | |
| | | 1% Cement | 13 | 3 | | | |
| | | 1% Fly Ash | 12 | 2 | | | |
| LBB | Caddell (High PI) | Raw | 34 | 20 | 66 | 33 | 1 |
| | | 1% Lime | 23 | 14 | | | |
| | | 1% Cement | 27 | 12 | | | |
| SJT | Lumpkin | Raw | 29 | 11 | 50 | 31 | 19 |
| | | New Gradation | | | 55 | 40 | 5 |

### Moisture Density and Related Test

Moisture-Density curves of the six lightly treated or modified base materials were determined using Tex-113-E procedure. Several statements can be made from the relationship, as shown in Table 4.7. The modification of gradation and the use of additives impact the OMC and the MDD of the materials. For most of the base materials, the OMC decreases after the use of cement.

**Table 4.7 - Characteristics of Base Materials before and after Treatment**

| Material | | | OMC, % | MDD, pcf | UCS at OMC, psi | Modulus at OMC, ksi |
|---|---|---|---|---|---|---|
| **District** | **Quarry** | **Material Type** | | | | |
| **ABL** | Old Bobby Noble | Raw | 6.2 | 138 | 46 | **92** |
| | | 1% Cement | 7.0 | 134 | 210 | **394** |
| | | New Gradation | 7.6 | 135 | 27 | **36** |
| **BWD** | Prater (Medium) | Raw | 10.8 | 133 | 49 | **24** |
| | | 1% Lime | 11.2 | 125 | 38 | **50** |
| | | 1% Cement | 11.1 | 124 | 69 | **149** |
| | Prater (Good) | Raw | 13.3 | 125 | 29 | **27** |
| | | 1% Cement | 10.9 | 121 | 69 | **210** |
| | | New Gradation | 11.1 | 125 | 44 | **25** |
| | Vulcan | Raw | 6.6 | 142 | 20 | **22** |
| | | 1% Cement | 7.0 | 142 | 138 | **419** |
| **ELP** | Cemex | Raw | 6.4 | 143 | 28 | **16** |
| | | 1% Lime | 7.3 | 144 | 38 | **55** |
| | | 1% Cement | 8.5 | 137 | 78 | **113** |
| | | 1% Fly Ash | 6.8 | 139 | 48 | **17** |
| **LBB** | Caddell (High PI) | Raw | 11.6 | 124 | 19 | **9** |
| | | 1% Lime | 10.7 | 119 | 58 | **26** |
| | | 1% Cement | 12.6 | 125 | 42 | **376** |
| **SJT** | Lumpkin | Raw | 12.7 | 122 | 40 | **28** |
| | | **New Gradation** | **12.1** | **123** | **29** | **16** |

Significant increase in strength and modulus is observed when the material is treated with 1% cement. The OMC and the MDD after the use of lime vary depending on the type of material. But the modulus and the strength of most of the lime treated base materials increased considerably. As observed for Brownwood “Good” and San Angelo (Lumpkin) materials, the OMC decreased when the fine content was reduced from 14 to 7% and from 19 to 5%, respectively. But for Abilene (Old Bobby Noble) material, the OMC increased when the fine content was increased from 1 to 5%. The MDD, the strength and the modulus after gradation modification vary depending on the type of material.

**Triaxial Compression Test**

All unconfined and confined triaxial compression tests were performed as per Tex-117-E. Regardless of the gradation or additive type, each specimen was prepared at the corresponding optimum moisture content for the given gradation and additive content. The curing method, however, is depended on the type of additive used. For cement-treated materials, the specimens were cured in a moist room for seven days as per procedure Tex-120-E. For lime-treated and fly ash-treated materials, specimen curing was achieved as per procedures Tex -121-E and Tex-127-E, respectively.

For all procedures, 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 140°F. After the specimens returned to room temperature, they were wrapped in filter paper to draw water into the specimen through capillary wetting and finally enclosed in a stainless steel triaxial chamber for 10 days.

According to a study by Scullion et al. (2003), an unconfined compressive strength of about 300 psi is satisfactory for cement-treated base materials. Procedures Tex-121-E and Tex-127-E recommend that the satisfactory strength value should be greater than 150 psi for either lime-treated or fly ash-treated base materials. The low dosage of additives would not fulfill these requirements as shown in Figure 4.4. This was anticipated because for economical reasons the calcium-based additives were used to just strengthen the material to pass the Item 247 requirements.

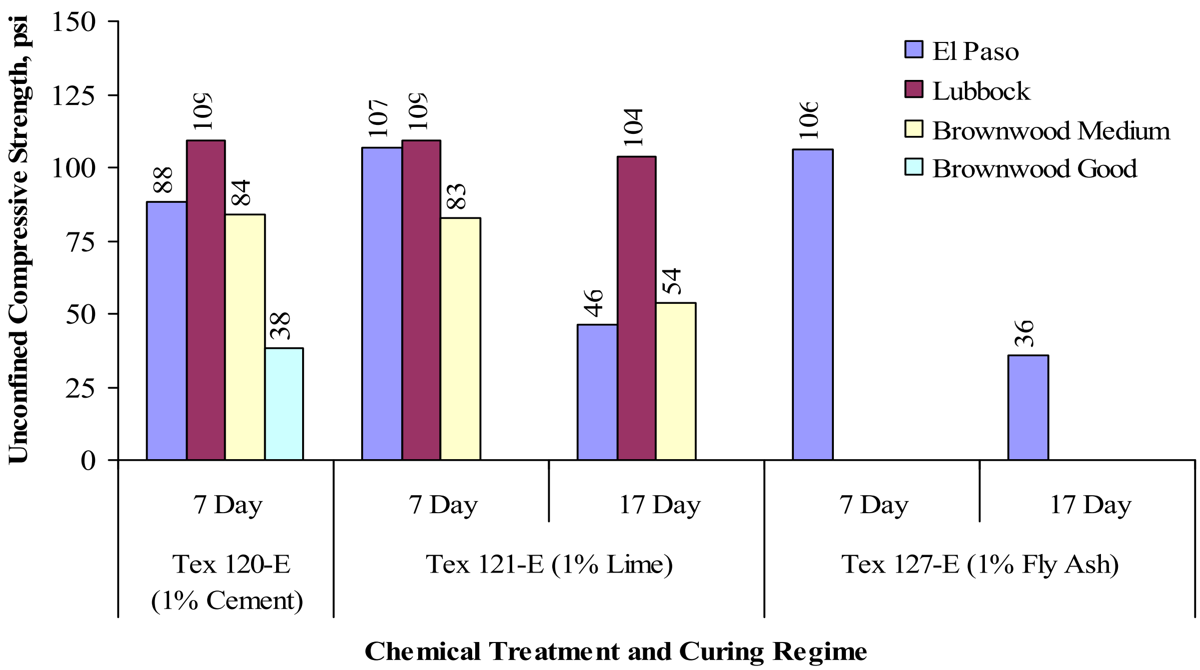



**Figure 4.4 - Unconfined Compressive Strengths of Different Materials as per Tex 120-E, Tex 121-E and Tex 127-E**

To further study the materials with chemical treatment and gradation modification, another set of specimens were prepared and cured as per Tex-117-E and proposed Tex-143-E. Results from compression tests on these specimens at different specified lateral pressures are summarized in Table 4.8 and compared with the corresponding requirements of TxDOT Item 247.

The Texas triaxial classifications of Lubbock and two Brownwood materials after chemical treatment somewhat improved. The classification of El Paso material treated with lime and cement remained unchanged and dropped with fly ash treatment. After gradation modification, the classifications of Brownwood and San Angelo materials were basically unchanged.

**Table 4.8 – Triaxial Compression for Base Materials before and after Treatment or Modification**

| MATERIAL SOURCE | | | ANGLE OF INTERNAL FRICTION, DEGREE | | COHESION, PSI | | TEXAS TRIAXIAL CLASS | | STRENGTH AT ZERO LATERAL PRESSURE, PSI | | STRENGTH AT 15 PSI LATERAL PRESSURE, PSI |
|---|---|---|---|---|---|---|---|---|---|---|---|
| District | Quarry | Material Type | Tex 117-E | Tex 143-E | Tex 117-E | Tex 143-E | Tex 117-E | Tex 143-E | Tex 117-E | Tex 143-E | Tex 117-E |
| ABL | Old Bobby Noble | Raw | 46.8 | 48.6 | 4.5 | 10.8 | 3.6 | 2.3 | 34 | 80 | 130 |
| | | 1% Cement | 52.7 | 50.0 | 21.6 | 35.3 | 1.0 | 2.2 | 143 | 210 | 284 |
| | | New Gradation | 52.4 | 50.9 | 3.0 | 8.1 | 3.4 | 2.1 | 26 | 27 | 128 |
| BWD | Prater (Medium) | Raw | 48.9 | 55.8 | 6.2 | 4.0 | 2.9 | 1.0 | 23 | 52 | 117 |
| | | 1% Lime | 49.3 | 55.8 | 11.4 | 12.0 | 2.2 | 1.0 | 55 | 79 | 148 |
| | | 1% Cement | 53.8 | 66.3 | 11.7 | 5.4 | 1.0 | 1.0 | 79 | 71 | 229 |
| | Prater (Good) | Raw | 47.6 | 50.3 | 7.2 | 10.5 | 2.9 | 2.1 | 29 | 22 | 120 |
| | | 1% Cement | 53.8 | 51.2 | 10.7 | 14.4 | 2.0 | 2.1 | 78 | 78 | 229 |
| | | New Gradation | 48.8 | 46.2 | 4.0 | 8.8 | 3.4 | 2.5 | 21 | 33 | 134 |
| | Vulcan | Raw | 54.8 | 56.8 | 7.0 | 6.9 | 2.4 | 1.0 | 32 | 46 | 180 |
| | | 1% Cement | 55.9 | 55.7 | 30.4 | 33.1 | 1.0 | 1.0 | 181 | 138 | 293 |
| ELP | Cemex | Raw | 58.2 | 59.8 | 9.7 | 7.8 | 1.0 | 1.0 | 62 | 28 | 230 |
| | | 1% Lime | 58.9 | 51.0 | 11.6 | 21.3 | 1.0 | 2.1 | 74 | 38 | 275 |
| | | 1% Cement | 57.8 | 62.6 | 21.7 | 2.2 | 1.0 | 1.0 | 146 | 78 | 339 |
| | | 1% Fly Ash | 50.2 | 62.9 | 7.0 | 1.5 | 2.9 | 1.0 | 46 | 48 | 162 |
| LBB | Caddell (High PI) | Raw | 54.9 | 55.7 | 6.7 | 3.6 | 2.5 | 1.0 | 46 | 19 | 198 |
| | | 1% Lime | 54.8 | 57.4 | 14.6 | 7.7 | 1.0 | 1.0 | 85 | 58 | 235 |
| | | 1% Cement | 64.0 | 57.8 | 9.8 | 9.4 | 1.0 | 1.0 | 59 | 42 | 316 |
| SJT | Lumpkin | Raw | 42.9 | 51.7 | 11.7 | 7.1 | 2.6 | 2.0 | 53 | 44 | 133 |
| | | New Gradation | 51.6 | 41.8 | 9.1 | 12.6 | 2.3 | 2.9 | 50 | 24 | 173 |

The two Brownwood materials that failed to meet the Item 247 specification for strength before treatment gained considerable amount of strength when treated with 1% cement for both zero and 15 psi lateral pressures. Lubbock material exhibited higher strength when treated with 1% lime rather than 1% cement. San Angelo (Lumpkin) material, after adjusting the gradation, met the required compressive strength for zero psi lateral pressure and almost met the strength requirement for 15 psi lateral pressure (173 psi vs. 175 psi).

As shown in Table 4.8, for El Paso, Lubbock and San Angelo (Lumpkin) materials, the unconditioned strengths (as per Tex-143-E) before and after treatment are less than or equal to those from moisture conditioned strengths (Tex-117-E), indicting that moisture does not have a negative impact on these materials. On the other hand, for the two Brownwood materials (Medium and Good) the unconditioned strengths are greater than those from moisture-conditioned specimens before and after treatment, indicating that there is a possibility of moisture susceptibility for the Brownwood materials.

## Performance and Additional Tests

Even though the requirements of Item 247 in Table 2.2 may ensure high quality base based on experience, it may not guarantee short-term and long-term quality since none of the parameters in the Table 2.2 is directly associated with mechanistic-based design. The performance tests carried out were primarily modulus-based and deformation-based. These results are represented and discussed below.

### Aggregate Quality Assessment

The determination of changes in gradation and aggregate toughness due to dynamic and static loads are of particular importance to characterize base materials. These changes can be measured in the laboratory using the Aggregate Impact Value (AIV) and Aggregate Crushing Value (ACV) as described in Chapter 3. A value of less than 30 is generally considered an acceptance limit. As shown in Figure 4.5, all base materials are considered reasonably good materials except the San Angelo (Turner) and the El Paso materials, as the ACV and the AIV (wet) of San Angelo (Turner) and the El Paso materials are greater than 30.

### Resilient Modulus

In almost all mechanistic-empirical design methods, the resilient modulus tests are advocated to determine the resilient properties of material. Since TxDOT currently does not have a protocol for performing the modulus test for base materials, ASHTO T-307 was followed.

Tests were carried out on “unconditioned” (at optimum moisture content, similar to Tex-143) specimens and moisture-conditioned (capillary wetting, similar to Tex-117-E) specimens. The results from resilient modulus tests on all materials before and after chemical treatment or gradation modification under the two different curing conditions are shown in Table 4.9 and Table 4.10. To compare the results from different tests, a representative modulus at a confining pressure of 5 psi and a deviatoric stress of 15 psi was estimated for each test. A representative resilient modulus of about 40 ksi at optimum moisture content is typically considered reasonable.

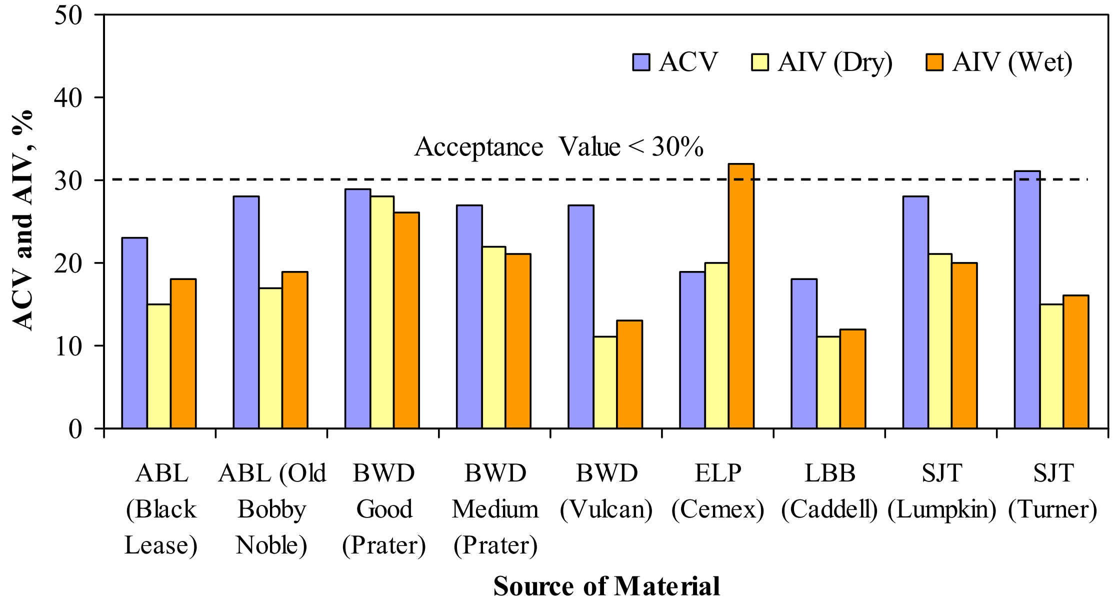



**Figure 4.5 - ACV and AIV of Different Materials**

Based on this criterion, all treated or modified bases except San Angelo should perform satisfactorily.

The variation in representative resilient modulus at optimum and capillary-saturated condition are compared in Figures 4.6 and 4.7, respectively. For the materials treated with 1% cement, the moisture-conditioned moduli are greater than those of the optimum-conditioned, perhaps due to hydration. For the materials whose gradation is adjusted, the saturated moduli are lower to those from the optimum-conditioned.

**Permanent Deformation**

The parameters of interest are the strain (or resilient strain) after 200- cycle loading, and the total permanent strain upon completion of test. The slope of the best-fit line to data passed 200 cycles, b, and the intercept at one cycle, a, are used in the mechanistic-empirical design programs. The smaller these parameters are, the lower the potential of the rutting of the base will be. These parameters for all bases under the optimum and saturated conditions are summarized in Tables 4.11 and 4.12. Based on the permanent deformation values, except El Paso, all the materials treated with additives exhibit low permanent strains. Under saturated condition, the Brownwood Good exhibits high permanent strains after the adjustment of gradation.

**Table 4.9 - Resilient Moduli of Different Materials before and after Treatment at Optimum Condition**

| Material Source | | | Nonlinear Model Parameters | | | | Representative Resilient Modulus, ksi | Seismic Modulus, ksi |
|---|---|---|---|---|---|---|---|---|
| District | Quarry | Material Type | $k_1$, ksi | $k_2$ | $k_3$ | $R^2$ | | |
| ABL | Black Lease | Raw | 91 | 0.31 | -0.11 | 0.72 | 111 | 136 |
| | Old Bobby Noble | Raw | 16 | 0.32 | -0.13 | 0.93 | 19 | 28 |
| | | 1% Cement | 127 | 0.24 | 0.00 | 0.84 | 187 | 200 |
| | | New Gradation | 12 | 0.49 | -0.04 | 0.91 | 24 | 44 |
| BWD | Prater (Medium) | Raw | 92 | 0.10 | -0.19 | 0.78 | 65 | 88 |
| | | 1% Lime | 60 | 0.34 | -0.12 | 0.89 | 75 | 89 |
| | | 1% Cement | 72 | 0.41 | -0.16 | 0.92 | 90 | 111 |
| | Prater (Good) | Raw | 57 | 0.28 | -0.12 | 0.82 | 65 | 84 |
| | | 1% Cement | 135 | 0.23 | -0.09 | 0.89 | 153 | 97 |
| | | New Gradation | 27 | 0.47 | -0.16 | 0.96 | 37 | 23 |
| | Vulcan | Raw | 18 | 0.65 | -0.08 | 0.93 | 41 | 21 |
| | | 1% Cement | 25 | 0.50 | 0.00 | 0.98 | 56 | 49 |
| ELP | Cemex | Raw | 50 | 0.39 | -0.42 | 0.88 | 30 | 38 |
| | | 1% Lime | 30 | 0.90 | -0.35 | 0.91 | 50 | 86 |
| | | 1% Cement | 55 | 0.30 | -0.12 | 0.83 | 64 | 405 |
| | | 1% Fly Ash | 18 | 0.28 | -0.07 | 0.97 | 23 | 54 |
| LBB | Caddell (High PI) | Raw | 22 | 0.45 | -0.18 | 0.93 | 28 | 37 |
| | | 1% Lime | 126 | 0.26 | -0.12 | 0.80 | 138 | 108 |
| | | 1% Cement | 65 | 0.39 | -0.27 | 0.89 | 59 | 147 |
| | Caddell (Low PI) | Raw | 9 | 0.51 | 0.00 | 0.98 | 20 | 31 |
| SJT | Lumpkin | Raw | 10 | 0.60 | -0.06 | 0.95 | 22 | 28 |
| | | New Gradation | 12 | 0.38 | -0.15 | 0.89 | 15 | 16 |
| | Turner | Raw | 38 | 0.46 | -0.08 | 0.96 | 64 | 26 |

**Table 4.10 - Resilient Moduli of Different Materials before and after Treatment at Saturated Condition**

| Material Source | | | Nonlinear Model Parameters | | | | Representative Resilient Modulus, ksi | Seismic Modulus, ksi |
|---|---|---|---|---|---|---|---|---|
| **District** | **Quarry** | **Material Type** | **$k_1$, ksi** | **$k_2$** | **$k_3$** | **$R^2$** | | |
| **ABL** | Black Lease | Raw | 61 | 0.38 | -0.07 | 0.96 | 93 | 147 |
| | Old Bobby Noble | Raw | 19 | 0.85 | -0.43 | 0.91 | 23 | 483 |
| | | 1% Cement | N/A | N/A | N/A | N/A | N/A | N/A |
| | | New Gradation | 18 | 0.57 | -0.05 | 0.99 | 39 | 26 |
| **BWD** | Prater (Medium) | Raw | 27 | 0.57 | -0.17 | 0.97 | 43 | 21 |
| | | 1% Lime | 62 | 0.25 | -0.19 | 0.90 | 55 | 273 |
| | | 1% Cement | 36 | 0.70 | -0.06 | 1.00 | 94 | 410 |
| | Prater (Good) | Raw | 19 | 0.61 | -0.11 | 0.93 | 38 | 45 |
| | | 1% Cement | 83 | 0.95 | -0.22 | 0.90 | 211 | 130 |
| | | New Gradation | 20 | 0.45 | -0.16 | 0.95 | 27 | 35 |
| | Vulcan | Raw | 40 | 0.45 | -0.20 | 0.87 | 48 | 261 |
| | | 1% Cement | 27 | 0.33 | 0.00 | 0.96 | 46 | 63 |
| **ELP** | Cemex | Raw | 41 | 0.37 | -0.17 | 0.72 | 47 | 55 |
| | | 1% Lime | 84 | 0.52 | -0.31 | 0.75 | 84 | 249 |
| | | 1% Cement | 351 | 0.41 | -0.13 | 0.95 | 478 | 908 |
| | | 1% Fly Ash | 44 | 0.73 | -0.34 | 0.92 | 57 | 40 |
| **LBB** | Caddell (High PI) | Raw | 27 | 0.50 | -0.23 | 0.95 | 32 | 20 |
| | | 1% Lime | 24 | 0.49 | -0.14 | 0.94 | 47 | 55 |
| | | 1% Cement | 111 | 0.26 | -0.12 | 0.81 | 122 | 373 |
| | Caddell (Low PI) | Raw | 12 | 0.50 | 0.00 | 0.98 | 27 | 19 |
| **SJT** | Lumpkin | Raw | 55 | 0.48 | -0.23 | 0.93 | 64 | 80 |
| | | New Gradation | 33 | 0.49 | -0.24 | 0.89 | 38 | 39 |
| | Turner | Raw | 26 | 0.50 | -0.11 | 0.91 | 43 | 147 |

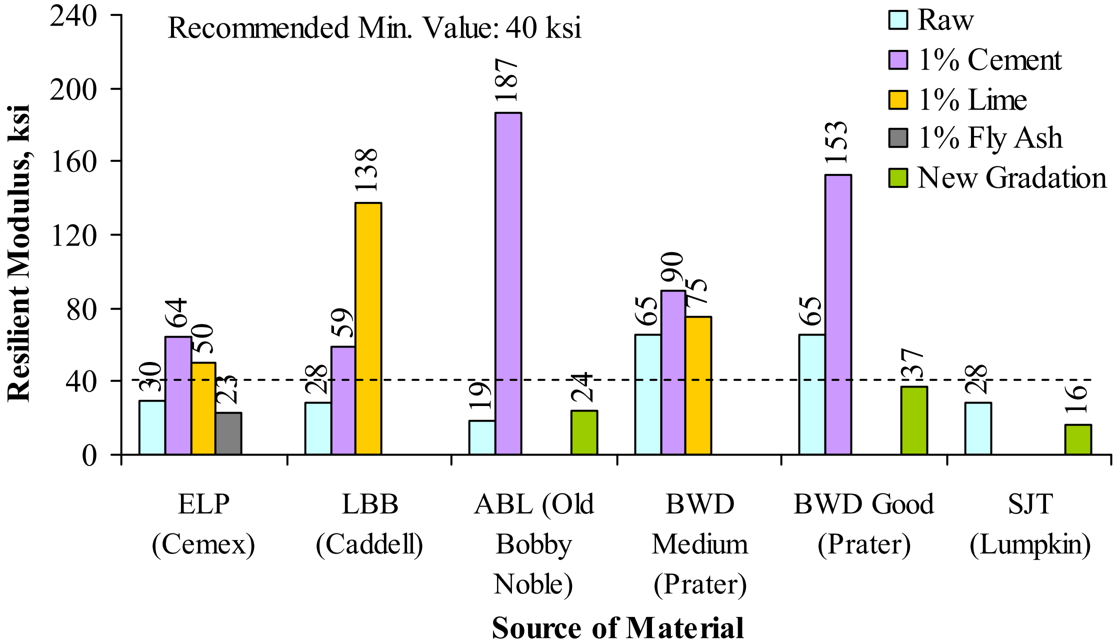



**Figure 4.6 - Representative Resilient Modulus for Different Materials at Optimum Moisture Content**

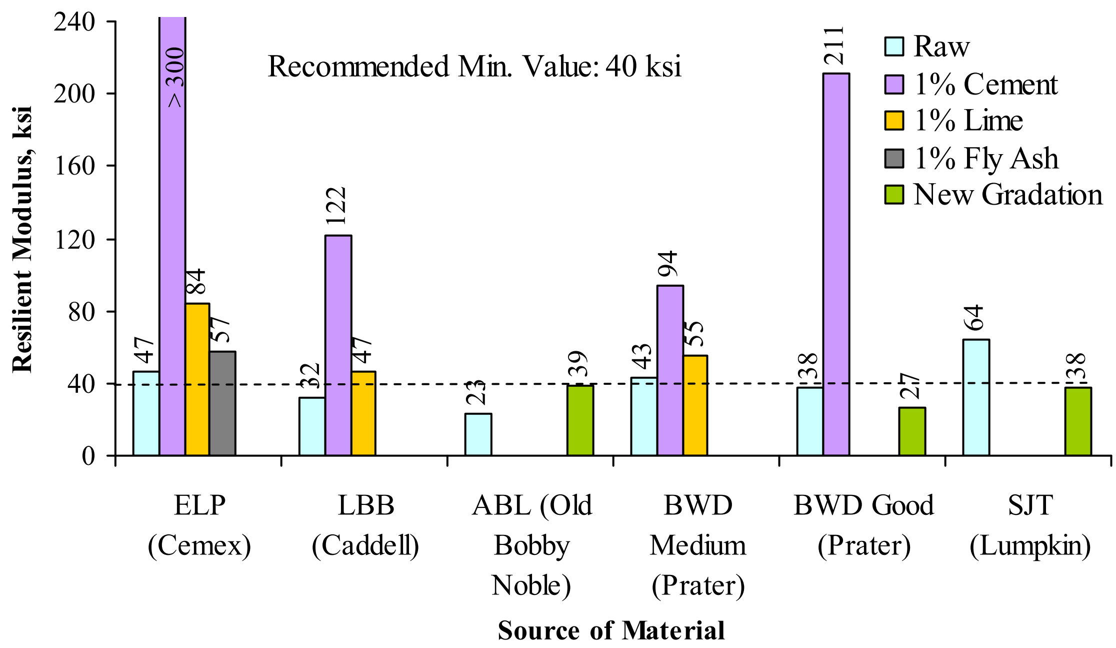



**Figure 4.7 - Representative Resilient Modulus for Different Materials at Saturated Condition**

**Table 4.11 – Permanent Deformation Parameters before and after Treatment at Optimum Condition**

| Material Source | | | Model Parameters | | | Resilient Strain (micro) | Permanent Strain (micro) |
|---|---|---|---|---|---|---|---|
| District | Quarry | Material Type | a | b | $R^2$ | | |
| ABL | Black Lease | Raw | 410 | 0.22 | 0.85 | 2540 | 4038 |
| | Old Bobby Noble | Raw | N/A | N/A | N/A | N/A | N/A |
| | | 1% Cement | 214 | 0.29 | 0.76 | 880 | 1475 |
| | | New Gradation | 104 | 0.20 | 0.82 | 283 | 401 |
| BWD | Prater (Medium) | Raw | 31 | 0.32 | 0.84 | 147 | 255 |
| | | 1% Lime | 174 | 0.13 | 0.87 | 335 | 417 |
| | | 1% Cement | 71 | 0.11 | 0.78 | 124 | 152 |
| | Prater (Good) | Raw | 646 | 0.10 | 1.00 | 1071 | 1253 |
| | | 1% Cement | 539 | 0.02 | 0.67 | 581 | 598 |
| | | New Gradation | 449 | 0.09 | 0.82 | 689 | 800 |
| | Vulcan | Raw | 32 | 0.30 | 0.80 | 137 | 229 |
| | | 1% Cement | 4183 | 0.08 | 0.84 | 6120 | 6975 |
| ELP | Cemex | Raw | 24 | 0.33 | 0.78 | 126 | 223 |
| | | 1% Lime | 148 | 0.25 | 0.83 | 522 | 810 |
| | | 1% Cement | 1196 | 0.06 | 0.99 | 1628 | 1793 |
| | | 1% Fly Ash | 185 | 0.40 | 1.00 | 1499 | 2865 |
| LBB | Caddell (High PI) | Raw | 173 | 0.46 | 1.00 | 1925 | 4031 |
| | | 1% Lime | 48 | 0.03 | 0.74 | 56 | 59 |
| | | 1% Cement | 823 | 0.04 | 0.88 | 983 | 1046 |
| | Caddell (Low PI) | Raw | 2120 | 0.17 | 0.83 | 4914 | 6592 |
| SJT | Lumpkin | Raw | 1728 | 0.10 | 0.84 | 2906 | 3475 |
| | | New Gradation | 63 | 0.33 | 0.83 | 321 | 566 |
| | Turner | Raw | 427 | 0.08 | 0.84 | 644 | 741 |

**Table 4.12 – Permanent Deformation Parameters before and after Treatment at Saturated Condition**

| Material Source | | | Model Parameters | | | Resilient Strain (micro) | Permanent Strain (micro) |
|---|---|---|---|---|---|---|---|
| District | Quarry | Material Type | a | b | $R^2$ | | |
| ABL | Black Lease | Raw | 583 | 0.06 | 0.82 | 802 | 895 |
| | Old Bobby Noble | Raw | 89 | 0.20 | 0.87 | 245 | 346 |
| | | 1% Cement | N/A | N/A | N/A | N/A | N/A |
| | | New Gradation | 4210 | 0.10 | 0.81 | 7056 | 8475 |
| BWD | Prater (Medium) | Raw | 783 | 0.06 | 0.84 | 1050 | 1164 |
| | | 1% Lime | 921 | 0.05 | 0.81 | 1184 | 1294 |
| | | 1% Cement | 66 | 0.12 | 0.71 | 121 | 149 |
| | Prater (Good) | Raw | 1142 | 0.10 | 0.99 | 1931 | 2274 |
| | | 1% Cement | 169 | 0.08 | 0.84 | 251 | 288 |
| | | New Gradation | 191 | 0.25 | 0.82 | 672 | 1042 |
| | Vulcan | Raw | 307 | 0.07 | 0.68 | 438 | 494 |
| | | 1% Cement | 1694 | 0.11 | 0.84 | 2914 | 3536 |
| ELP | Cemex | Raw | 594 | 0.12 | 0.84 | 1074 | 1316 |
| | | 1% Lime | 272 | 0.06 | 0.83 | 368 | 409 |
| | | 1% Cement | 195 | 0.05 | 0.75 | 248 | 269 |
| | | 1% Fly Ash | 711 | 0.14 | 0.84 | 1413 | 1786 |
| LBB | Caddell (High PI) | Raw | 885 | 0.12 | 1.00 | 1683 | 2055 |
| | | 1% Lime | 13 | 0.27 | 0.81 | 51 | 83 |
| | | 1% Cement | 306 | 0.07 | 0.82 | 437 | 496 |
| | Caddell (Low PI) | Raw | 2006 | 0.18 | 0.91 | 4990 | 6794 |
| SJT | Lumpkin | Raw | 371 | 0.13 | 0.84 | 709 | 887 |
| | | New Gradation | 514 | 0.11 | 0.84 | 904 | 1098 |
| | Turner | Raw | 418 | 0.07 | 0.79 | 593 | 672 |

## Moisture Susceptibility

Tube Suction Test (TST) is used for assessing the capillary rise of moisture or moisture susceptibility within materials. Figure 4.8 shows the dielectric constants of all materials selected after 10-day curing (2-day oven dry and 8-day moisture conditioning by capillary rise). Details of changes in dielectric constant, moisture content, seismic modulus and unconfined compressive strength at three critical curing times are summarized in Table 4.13.

Based on the change in moisture content, a reasonable material would lose a significant portion of its moisture during the first two days in the oven, and it would not absorb much moisture during the capillary process. As reflected in Table 4.13, Brownwood Good and El Paso bases when treated with cement absorb more water moisture conditioning than their initial optimum moisture contents. On the other hand, the San Angelo and the Lubbock bases absorb less moisture as compared to their optimum, the desirable condition.

Under the TST moisture conditioning, one of the concerns is that the specimens would gain significant stiffness in the first two days and then would lose the stiffness due to the introduction of moisture into the specimens. The increase and decrease in modulus should be small for a reasonable base. The retained modulus is then described as the modulus after ten days of saturation divided by the modulus after two days of oven-drying. These values are reported in Figure 4.9 for all base materials. The materials treated with additives demonstrate high values of retained modulus, whereas the two bases, Brownwood Good and San Angelo materials after gradation adjustment, demonstrate the lower values of retained modulus.

The retained strength as described in the TST tests were also determined. The retained strength is defined as the unconfined compressive strength after 10 days of moisture conditioning divided by the unconfined compressive strength after 24 hours of curing in room temperature. The retained strengths are demonstrated in Figure 4.10. The acceptable limit is typically 80%. Based on this level, all bases are reasonable after chemical treatment or gradation modification.

Finally, the performance-based evaluation of test results for each material is summarized in Table 4.14. Based on this table, after treatment almost all the base materials meet all the requirements except the dielectric constants which are nevertheless within the acceptance limits (10 to16) for marginal quality base materials.

## Small Scale Test

The results from the small-scale tests on Lubbock base material from Caddell Pit and Abilene base material from Old Bobby Noble quarry are discussed in this section. Four specimens for each material were prepared for small-scale testing. The base layers in two specimens did not contain additives while other two were treated with 1% lime for Lubbock base material and with 1% cement for Abilene base materials. Two of the specimens were constructed using a clay subgrade and the other two using a sandy subgrade. The major properties of the materials obtained from the standard tests are summarized in Table 4.15 through Table 4.18.

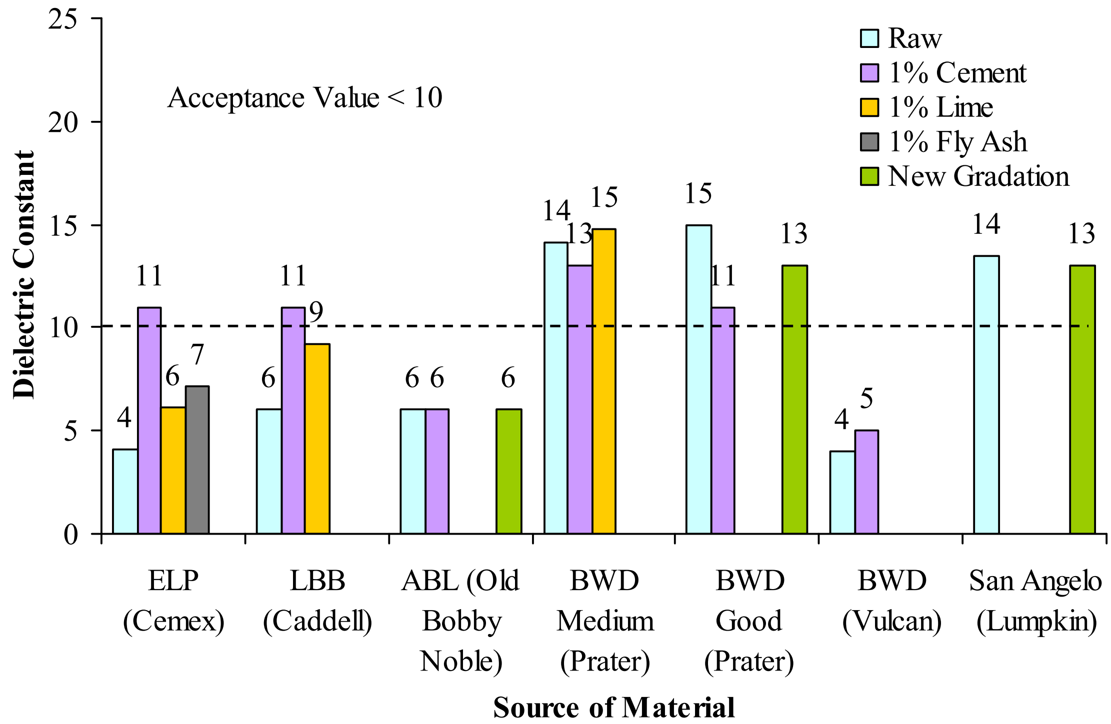



**Figure 4.8 - Dielectric Constants of Different Materials**

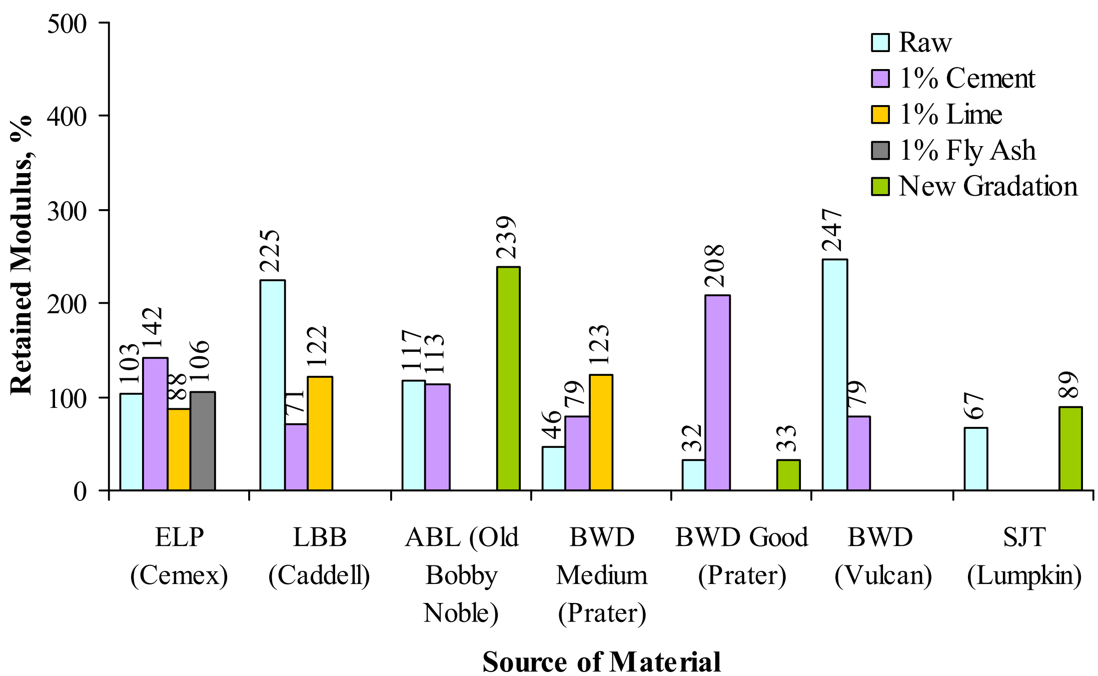



**Figure 4.9 - Retained Moduli of Different Materials**

Table 4.13 - Variations in Dielectric Constant, Moisture Content and Seismic Modulus with Time

| Base | | | Dielectric Constant | | | Change in Moisture Content, % | | | Seismic Modulus, ksi | | | Unconfined Compressive Strength, psi | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| District | Quarry | Material Type | Day 1 | Day 2 | Day 10 | Day 1 | Day 2 | Day 10 | At OMC (Tex 143) | Day 10 (Tex 117) | Retained Modulus, % | At OMC | D 10 (UCS) | Retained Strength, % |
| ABL | Black Lease | Raw | 4.8 | 4.6 | 4.3 | -5.5 | -6.1 | -3.4 | 36 | 37 | 103 | 103 | 127 | 123 |
| | Old Bobby Noble | Raw | 5.4 | 5.2 | 5.6 | -6.2 | -6.7 | -3.5 | 66 | 77 | 117 | 80 | 99 | 124 |
| | | 1% Cement | 3.8 | 3.6 | 6.2 | -4.8 | -5.7 | 1.1 | 394 | 446 | 113 | 210 | 118 | 56 |
| | | New Gradation | 6.1 | 6.1 | 6.4 | -7.0 | -7.2 | -4.1 | 8 | 49 | 613 | 27 | 55 | 204 |
| BWD | Prater (Medium) | Raw | 5.6 | 4.5 | 14.1 | -8.5 | -9.9 | 0.7 | 37 | 17 | 46 | 52 | 18 | 35 |
| | | 1% Lime | 4.7 | 3.3 | 14.8 | -8.3 | -9.5 | 1.4 | 57 | 64 | 123 | 79 | 52 | 66 |
| | | 1% Cement | 4.8 | 4.6 | 12.5 | -7.3 | -8.2 | -0.9 | 156 | 123 | 79 | 71 | 110 | 155 |
| | Prater (Good) | Raw | 5.1 | 4.3 | 15.0 | -9.8 | -11.4 | 0.2 | 22 | 7 | 32 | 22 | 18 | 82 |
| | | 1% Cement | 3.1 | 3.4 | 10.9 | -8.1 | -9.3 | 1.2 | 25 | 117 | 468 | 78 | 78 | 100 |
| | | New Gradation | 4.2 | 3.7 | 12.5 | -7.2 | -9.5 | 0.6 | 36 | 12 | 33 | 33 | 16 | 48 |
| | Vulcan | Raw | 3.9 | 3.4 | 3.5 | -5.5 | -6.1 | -3.5 | 32 | 79 | 247 | 46 | 64 | 139 |
| | | 1% Cement | 3.3 | 3.3 | 5.0 | -4.5 | -5.7 | -1.4 | 419 | 329 | 79 | 138 | 186 | 135 |
| ELP | Cemex | Raw | 3.5 | 3.3 | 4.1 | -4.7 | -5.3 | -2.9 | 33 | 34 | 103 | 28 | 108 | 386 |
| | | 1% Lime | 4.0 | 3.9 | 6.1 | -6.1 | -6.2 | -2.7 | 58 | 51 | 88 | 38 | 152 | 400 |
| | | 1% Cement | 4.6 | 3.9 | 11.4 | -1.5 | -1.0 | 4.2 | 104 | 148 | 142 | 78 | 131 | 168 |
| | | 1% Fly Ash | 4.4 | 4.1 | 7.2 | -2.8 | -5.8 | -2.8 | 17 | 18 | 106 | 48 | 79 | 165 |
| LBB | Caddell (High PI) | Raw | 4.5 | 4.2 | 6.0 | -8.3 | -9.0 | -4.6 | 4 | 9 | 225 | 19 | 26 | 137 |
| | | 1% Lime | 4.3 | 3.8 | 9.2 | -6.7 | -7.8 | -0.7 | 59 | 72 | 122 | 58 | 103 | 178 |
| | | 1% Cement | 4.3 | 4.5 | 10.6 | -7.7 | -9.0 | -0.7 | 106 | 75 | 71 | 42 | 81 | 193 |
| | Caddell (Low PI) | Raw | Could not complete the test | | | | | | | | | | | |
| SJT | Lumpkin | Raw | 5.8 | 5.0 | 13.5 | -8.4 | -9.5 | -0.2 | 12 | 8 | 67 | 44 | 31 | 70 |
| | | New Gradation | 4.2 | 4.1 | 12.7 | -9.0 | -9.8 | -1.2 | 9 | 8 | 89 | 24 | 47 | 196 |
| | Turner | Raw | 4.5 | 3.6 | 8.3 | -5.8 | -6.4 | -1.7 | 19 | 40 | 211 | 66 | 70 | 106 |

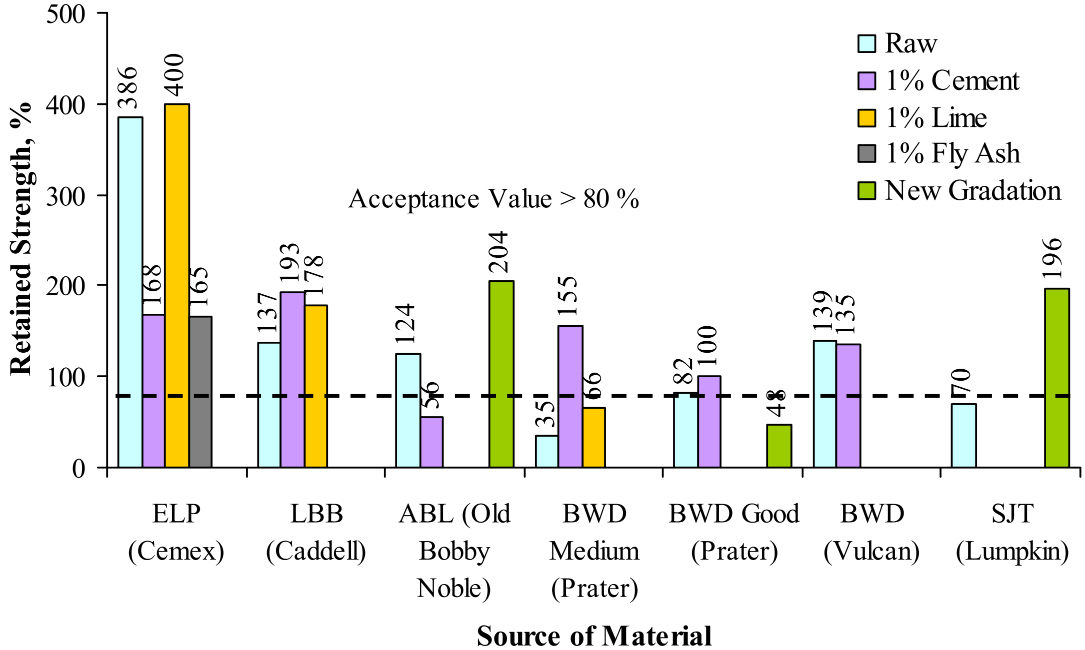



**Figure 4.10 - Retained Strength of Different Materials**

**Table 4.14 - Evaluation of the Results Based on Performance Tests**

| **Source** | | **Aggregate Stiffness** | | | **Dielectric Constant (<10)** | | **Resilient Modulus (>40 ksi)** | | | | **Retained Strength (>80%)** | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | **ACV** (<30%) | AIV (<30%) | | | | at OMC | | Saturated | | | |
| **District** | Quarry | | Dry | Wet | Before | After | Before | After | Before | After | Before | After |
| **ABL** | Old Bobby Noble | 28 | 17 | 19 | 6 | 6 | (19) | 187 | (23) | N/A | 124 | (56) |
| **BWD** | Medium | 27 | 22 | 21 | (14) | (13) | 65 | 90 | 43 | 94 | (35) | 155 |
| | Good | 29 | 28 | 26 | (15) | (11) | 65 | 153 | (27) | 211 | 82 | 100 |
| | Vulcan | 27 | 11 | 13 | 4 | 5 | 41 | 56 | 48 | 46 | 139 | 135 |
| **ELP** | Cemex | 19 | 20 | (32) | 4 | (11) | (30) | 64 | 47 | 478 | 386 | 168 |
| **LBB** | Caddell | 18 | 11 | 12 | 6 | 9 | (28) | 138 | (32) | 47 | 137 | 193 |
| **SJT** | Lumpkin | 28 | 21 | 20 | (14) | (13) | (28) | (16) | (38) | 64 | (70) | 196 |

**Table 4.15 - Moisture-Density Test Results**

| **Parameter** | **Subgrade** | | **Lubbock Base** | | **Abilene Base** | |
|---|---|---|---|---|---|---|
| | **Clay** | **Sandy** | **Raw** | **1% Lime** | **Raw** | **1% Cement** |
| **Optimum Moisture Content (%)** | 16.5 | 11.0 | 11.6 | 10.7 | 6.2 | 7.0 |
| **Maximum Dry Unit Weight (pcf)** | 112 | 120 | 124 | 119 | 138 | 134 |

**Table 4.16 – Triaxial Compression Test Results**

| Parameter | | Subgrade | | Lubbock Base | | Abilene Base | |
|---|---|---|---|---|---|---|---|
| | | Clay | Sandy | Raw | 1% Lime | Raw | 1% Cement |
| Tex 117-E | Angle of Internal Friction (degrees) | 24.3 | 30.5 | 54.9 | 54.8 | 46.8 | 52.7 |
| | Cohesion (psi) | 9.1 | 8.4 | 6.7 | 14.6 | 4.5 | 21.6 |
| | Classification | 4.0 | 3.5 | 2.5 | 1.0 | 3.6 | 1.0 |
| | Strength at 0 psi stress (psi) | 29 | na | 46 | 85 | 34 | 143 |
| | Strength at 15 psi stress (psi) | 64 | na | 198 | 235 | 130 | 284 |
| Tex 143-E | Angle of Internal Friction (degrees) | 49 | 35 | 56 | 55 | 48.6 | 50.0 |
| | Cohesion (psi) | 4.9 | 5.2 | 3.6 | 7.7 | 10.8 | 35.3 |
| | Classification | 2 | 4 | 1 | 1 | 2.3 | 2.2 |
| | Strength at 0 psi stress (psi) | 52 | | 19 | 58 | 80 | 210 |
| | Strength at 10 psi stress (psi) | 95 | 56 | 135 | 176 | 131 | 394 |

**Table 4.17 - Permanent Deformation Test Results**

| Moisture Condition | Parameters | Subgrade | | Lubbock Base | | Abilene Base | |
|---|---|---|---|---|---|---|---|
| | | Clay | Sandy | Raw | 1% Lime | Raw | 1% Cement |
| Optimum | $\varepsilon_r$, $\mu$strain | 2801 | 610 | 1925 | 56 | 283 | 80 |
| | $\alpha$ | 0.84 | 0.76 | 0.04 | 0.03 | 0.80 | 0.71 |
| | $\mu$ | 0.05 | 0.12 | 0.54 | 0.97 | 0.07 | 0.07 |
| Saturated | $\varepsilon_r$, $\mu$strain | N/A | 550 | 1683 | 51 | 245 | N/A |
| | $\alpha$ | | 0.86 | 0.06 | 0.07 | 0.80 | N/A |
| | $\mu$ | | 1.03 | 0.88 | 0.72 | 0.07 | N/A |

**Table 4.18 - Resilient Modulus Test Results**

| Parameter | | | Subgrade | | Lubbock Base | | Abilene Base | |
|---|---|---|---|---|---|---|---|---|
| | | | Clay | Sandy | Raw | 1% Lime | Raw | 1% Cement |
| At Optimum Moisture Content | Model Parameters | $k_1$, ksi | 3 | 15 | 22 | 126 | 16 | 127 |
| | | $k_2$ | 0.44 | 0.35 | 0.45 | 0.26 | 0..32 | 0.24 |
| | | $k_3$ | -0.22 | -0.20 | -0.18 | -0.12 | -0.13 | 0.0 |
| | | $R^2$ | 0.95 | 0.86 | 0.93 | 0.80 | 0.93 | 0.84 |
| | Representative Res. Mod. (ksi) | | 4 | 18 | 28 | 138 | 19 | 187 |
| After 10 days of Capillary Saturation | Model Parameters | $k_1$, ksi | Not Possible to Test | 3 | 27 | 24 | 19 | N/A |
| | | $k_2$ | | 1.00 | 0.50 | 0.49 | 0.85 | N/A |
| | | $k_3$ | | -0.03 | -0.23 | -0.14 | -0.43 | N/A |
| | | $R^2$ | | 0.81 | 0.95 | 0.94 | 0.91 | N/A |
| | Representative Res. Mod. (ksi) | | | 14 | 32 | 36 | 23 | N/A |

The results from modulus tests on the base layer of each small-scale specimen are summarized in Tables 4.19 and 4.20. These results indicate that the bases treated with 1% lime or 1% cement are significantly stiffer than in its virgin state, which is consistent with the results from the laboratory tests.

As an example, the load-deflection curves from the three moisture conditions for specimens of Lubbock base material are summarized in Figure 4.11. Assuming that 100 mils of deformation correspond to failure, the loads at failure are summarized in Table 4.21. Under the optimum moisture condition, the base stabilized with 1% lime carried substantially more load as compared to the base without stabilization. This pattern was also observed for the other two moisture conditions.

Permanent deformations from the small-scale tests are included in Table 4.22. The permanent deformation or resilient deformation (rutting) after 200 cycles of loading is substantially less for the base treated either with 1% lime or with 1% cement under the optimum condition. Under the subgrade-saturated condition, the specimens on sandy subgrade performed better. Again, the resilient deformations are less for the treated bases. When both the base and subgrade became moist, the resilient deformations are much higher than the optimum condition for all cases. However, for the lime-treated samples, the resilient deformations are less than 100 mils.

Based on this study, it can be concluded that the results from the small-scale tests are reflective of the performance of the bases in the condition similar to that in the field, and that the treatment of a local base material with small amount additive will provide a better-performing pavement.

**Table 4.19 – PSPA Test Results**

| Status | Modulus, ksi | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Lubbock | | | | Abilene | | | |
| | Raw | | 1% Lime | | Raw | | 1% Cement | |
| | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG |
| **Optimum** | 75 | 81 | 120 | 155 | 93 | 100 | 240 | 419 |
| **SG Saturated** | 56 | 67 | 104 | 108 | 89 | 56 | 175 | 286 |
| **Base Saturated** | 44 | 36 | 75 | 86 | 45 | NA | 135 | 243 |

**Table 4.20 - DCP Test Results**

| Status | Modulus, ksi | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Lubbock | | | | Abilene | | | |
| | Raw | | 1% Lime | | Raw | | 1% Cement | |
| | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG |
| **Optimum** | 11 | 15 | 22 | 23 | 16 | 22 | 30 | 38 |
| **SG Saturated** | 12 | 14 | 23 | 22 | 10 | 22 | 21 | 41 |
| **Base Saturated** | 14 | 11 | 19 | 19 | 7 | 8 | 18 | 26 |

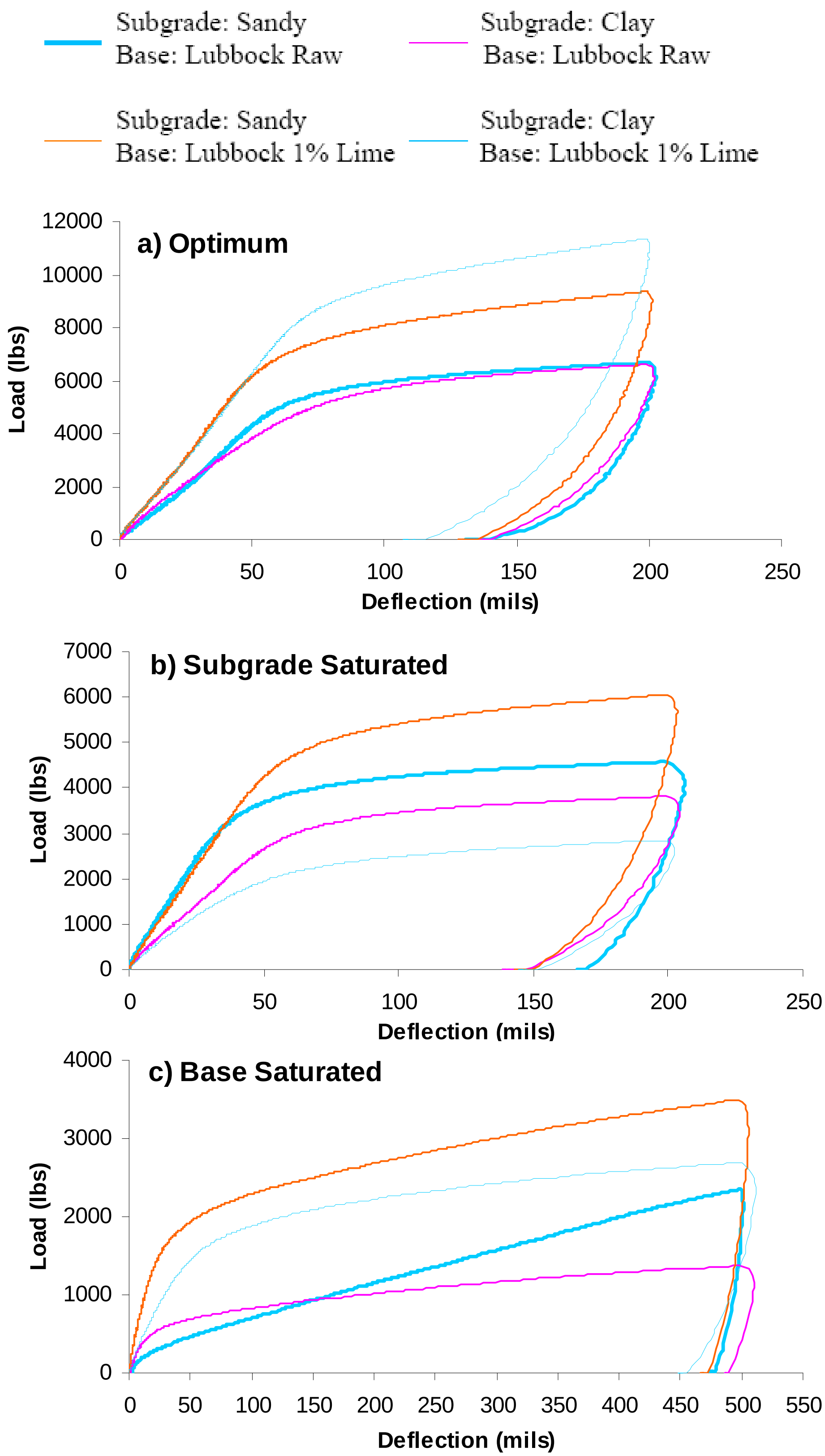



**Figure 4.11 - Load Deflection Curves from Small Scale Test**

**Table 4.21 - Loads Corresponding to 100 mils of Deflection from Small-Scale Tests**

| Moisture Condition | Load, lbs | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Lubbock Base | | | | Abilene Base | | | |
| | Raw | | 1% Lime | | Raw | | 1% Cement | |
| | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG |
| **Optimum** | 5941 | 5692 | 8079 | 9608 | 6909 | 6726 | 8491 | 6090 |
| **SG Saturated** | 4243 | 3456 | 5417 | N/A | 1133 | 1904 | 4899 | 3213 |
| **Base Saturated** | 698 | 818 | 2285 | 1874 | NA | NA | 3104 | 2100 |

**Table 4.22 – Base Permanent Deformations (mils) after 200 Loading Cycles from Small-Scale Tests**

| Moisture Condition | Lubbock Base | | | | Abilene Base | | | |
|---|---|---|---|---|---|---|---|---|
| | Raw | | 1% Lime | | Raw | | 1% Cement | |
| | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG | Sandy SG | Clay SG |
| **Base Optimum** | 42 | 41 | 18 | 14 | 7 | 4 | 3 | 15 |
| **SG Saturated** | 81 | 58 | 29 | N/A | N/A | N/A | 14 | 41 |
| **Base Saturated** | 171 | N/A | 74 | 89 | N/A | N/A | 33 | 58 |

**Field Monitoring**

Due to the limitation of actual construction using the base materials selected for this project, only the FWD data on two projects, FM 1702 and FM 2376 in Brownwood District, and the PSPA data on one road, FM 587, in Lubbock District are available for indirectly comparing the results from field and laboratory tests.

The pavement section in FM 1702 consists of two surface courses (about 1 in. thick), an 8-in. base and the subgrade. The base material used in this road is the "Medium" to "Good" from Prater pit. The pavement section in FM 2376 consists of a 2-in. of an HMA layer, a 14-in. base and the subgrade. The base material used in this road is from Vulcan pit. Both materials were used as-is (without any chemical treatment). The thicknesses of the base layers as constructed for the two rods are close to those provide by the structural analysis in Chapter 5.

The FWD moduli and laboratory tests and compared in Figure 4.12 for the Brownwood sites. For material for each pit, the results from laboratory tests include the resilient modulus of specimen prepared at the optimum moisture content, the average FFRC modulus of specimens prepared for triaxial strength test (Tex-143-E) and the unconfined compressive strengths (UCS) as per Tex-117-E and Tex-143-E. Field and laboratory moduli for the materials from Prater are consistently greater than those from Vulcan. The strength parameters (including the confined strengths at 15-psi lateral pressure as shown in Table 4.4) however, demonstrated the opposite trend for the moduli. These results indicate that modulus and strength are not always consistent for material characterization, and the inconsistency should be considered in mix design (using strength) and pavement design/evaluation (using modulus).

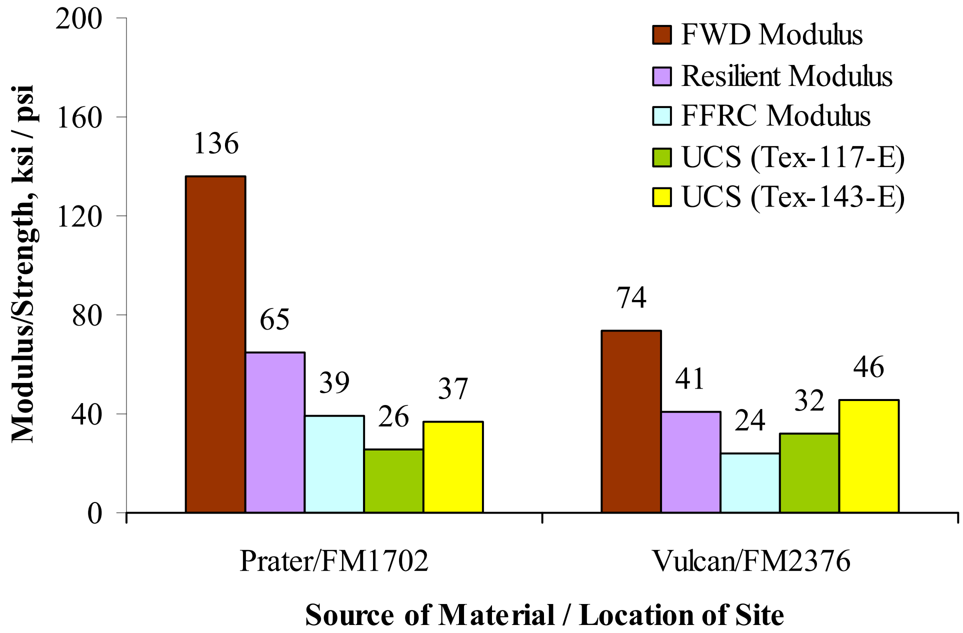



**Figure 4.12 – Comparison of Results from FWD Measurements and Laboratory Tests**

The pavement section in FM 597 consists of a 0.5 in. thick surface course (at the time when the PSPA tests were carried out), a 7-in. base and the subgrade. The base materials used in this road came from two different stockpiles of the same quarry (Caddell). One stockpile had a PI of about 20 and the other a PI of about 8. The high-PI material was pretreated with 1% lime at the quarry. Both materials were also stabilized with 7% fly ash for construction, unknown to the research team.

Figure 4.13 shows a comparison of the results from PSPA measurement and laboratory tests. The improvement by adding 1% lime to the high PI material is evident from the laboratory results (which were carried out without the addition of the fly ash). However, the field moduli are not significantly different simply because of the addition of the 7% fly ash masks the impact of the 1% lime.

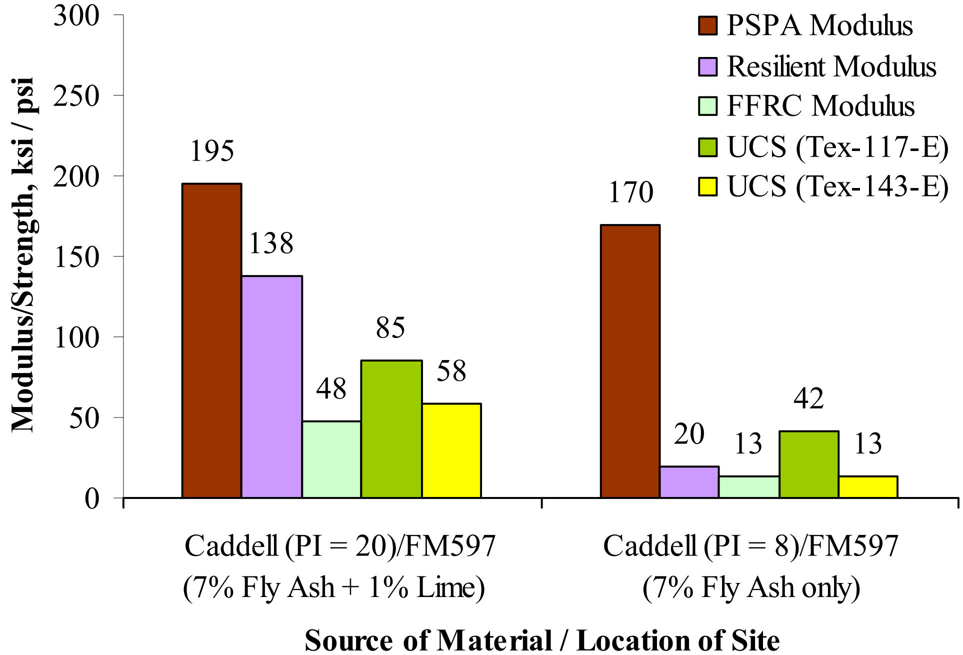



**Figure 4.13 – Comparison of Results from PSPA Measurements and Laboratory Tests**

# Chapter 5

# Structural Analysis and Cost Evaluation

## Introduction

The purpose of a pavement is to carry traffic safely, conveniently and economically over its design life. The pavement must have adequate thickness to ensure that the stresses and strains due to traffic loads at all levels in the pavement and subgrade are within acceptable limits. Aside from the structural adequacy, the economical feasibility of a given project is also very important. In the context of pavement design, structural adequacy and economic feasibility should be considered.

For structural analysis, a sensitivity study was conducted to identify the parameters that impact the pavement performance the most. Since the major structural distress in low-volume roads is rutting, this type of distress is emphasized in this study. Also the equivalent thicknesses of two sets of base layers, one with high-quality material and the other one with local low-quality material, were determined to evaluate the costs between the two alternatives. Finally, the feasibility of using the lower-quality local materials as a subbase was explored.

## Structural Analysis

Figure 5.1 shows the critical stresses and strains that are used to quantify the fatigue cracking of the surface layer and the rutting of subgrade. In most classical structural design programs (such as FPS19 or Texas Triaxial), the design thicknesses of the layers are directly or indirectly estimated based on the criteria that the stresses at the interfaces between the surface layer and base and between the base and subgrade are low enough so that the cracking of the surface layer and rutting of the subgrade will not be an issue. For a given traffic condition, the thicker the layers overlying the base, the thicker the base layer and the stiffer the subgrade are, the lower the classical critical stresses and strains will be. With these design algorithms, the rutting of subgrade can be controlled by replacing a good quality base with a thicker low-quality base.

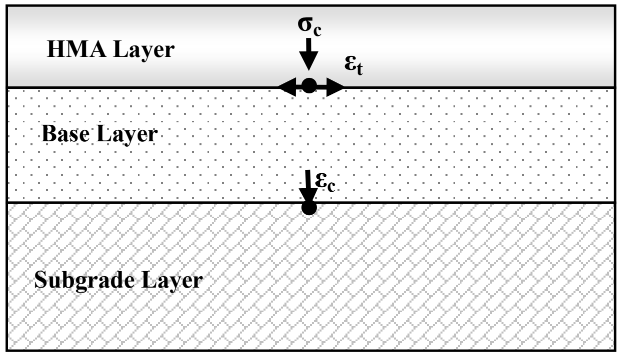



**Figure 5.1 - Critical Stresses and Strains in a Three-Layer Flexible Pavement System**

An important aspect of the pavement performance relevant to this study that the classical design programs (such as FPS19 and Texas Triaxial) neglect is the rutting of the base layer. For a given thickness of HMA, a lower quality base may rut, even though the subgrade may not. This is very critical for the low-volume roads were the HMA is quite thin, or only the surface-treatment is applied. Two software packages that can model the rutting of individual pavement layers are available. Zhou and Scullion (2005) at TTI developed a convenient pre- and post-processor for the classical VESYS program originally developed by Kenis (1977) called VESYS5W. Tirado et al. (2006) developed an advanced version of VESYS5W called TxIntPave which addresses some of the well-known limitations of VESYS5W and can be used to estimates the rutting of each individual layer of in a flexible pavement with the number of truck passes.

Figure 5.2 depicts a typical graph obtained from TxIntPave showing the rutting contribution to each pavement layer with traffic volume.

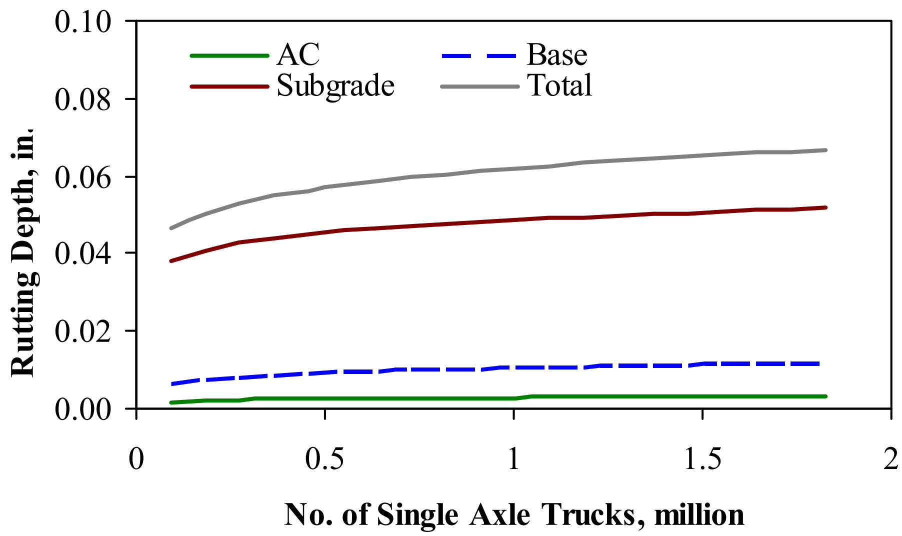



**Figure 5.2 - Typical Graph Obtained from VESYS (TxIntPave) Program**

The parameters of interest in TxIntPave for base layer are modulus parameters ($k_1$, $k_2$ and $k_3$) that can be obtained from resilient modulus test, and permanent deformation parameters $\alpha$ and $\mu$ (see Equations 3.3 and 3.5). Other parameters influencing the rutting of pavement structure are the thicknesses of surface layer (seal coat or HMA) and base layer and the modulus of the subgrade.

## Sensitivity Analysis

An extensive sensitivity analysis was carried out to understand which parameters contribute the most to the rutting of the base layer. The pavement related parameters used in the sensitivity study are summarized in Tables 5.1 and 5.2. For each parameter a range and a baseline value are assigned. The baseline values are considered as "typical" for Texas. The range covers the possible values that should be expected for corresponding materials. The traffic related parameters used in this analysis are shown in Table 5.3 where the Average Daily Traffic (ADT) is specified as 250 for low volume loads.

Two categories of bases are considered: low quality and high quality (meets requirements of TxDOT Item 247 for Grade 1). Similarly, two types of subgrade are considered: weak (most likely to represent the East Texas) and strong (most likely to represent the West Texas).

**Table 5.1 - TxIntPave Input Parameters for Base Layer**

| Parameter | Resilient Modulus Parameters | | | Permanent Deformation Parameters | |
|---|---|---|---|---|---|
| | $k_1$, ksi | $k_2$ | $k_3$ | $\alpha$ | $\mu$ |
| Range | 10 to 50 (Low Quality)<br>50 to150 (High Quality) | 0.1 to 0.5 | -0.5 to -0.1 | 0.5 to 0.9 | 0.01 to 0.50 |
| Baseline Value | 30 (Low Quality)<br>100 (High Quality) | 0.3 | -0.3 | 0.7 | 0.05 |

**Table 5.2 – Typical Properties of Reference Pavement Layers**

| Layer | Modulus, ksi | | Poisson's Ratio | Thickness, in. | |
|---|---|---|---|---|---|
| | Range | Baseline Value | | Range | Baseline Value |
| AC | 300 | | 0.33 | 1 to 7 | 1 |
| Base | 30 (low quality), 100 (high quality) | | 0.35 | 6 to 18 | 12 |
| Subgrade | 4 to 20 (weak)<br>10 to 50 (strong) | 12 (weak)<br>30 (strong) | 0.35 | N/A | |

**Table 5.3 - Traffic Related Parameters**

| | | | |
|---|---|---|---|
| ADT (with 0% growth rate) | 250 | Tire Configuration | Dual |
| Vehicle Type | Single Axle | Tire Pressure | 70 psi |
| Axle Type | Single 18 k | Tire Spacing | 13.5 in. |
| Analysis Period | 20 years | Tire Radius | 4.5 in. |

To conduct the sensitivity analysis, the rutting in each pavement layer with load repetition was estimated for the base line values. Each parameter in Tables 5.1 and 5.2 was individually perturbed within its corresponding range during the analysis with TxIntPave. The results from the perturbed pavement were then normalized to the corresponding those from the baseline values.

**ACP Layer**

Since this research is focusing on low-volume roads, the baseline value for ACP thickness is set to 1 in., assuming that the layer is a seal coat. To study the effect of ACP thickness on the rutting of pavement structure, thickness varying from 1 in. to 7 in. is considered. The modulus and the Poisson's ratio for all cases are kept constant at 300 ksi and 0.33, respectively.

The impact of ACP thickness on the rutting of ACP layer is shown in Figure 5.3. The y-axis marked as "Normalized Rutting," is simply the rut depth for a given ACP thickness divided by the rut depth measured for the baseline pavement with an ACP thickness of 1 in. For example, when the thickness of the ACP layer on top of the low quality base is increased from 1 in. to 7 in., the rutting of the ACP layer increases by about 50 times. However, for the same case but with a high quality base, the ACP rutting increases by less than 20 times. This significant increase in rutting should not be interpreted that the pavement will not perform reasonably well in rutting (since the rutting of the pavement with 1 in. ACP is very small); it simply states that the rutting of the ACP is sensitive to the quality of the base and the thickness of the ACP.

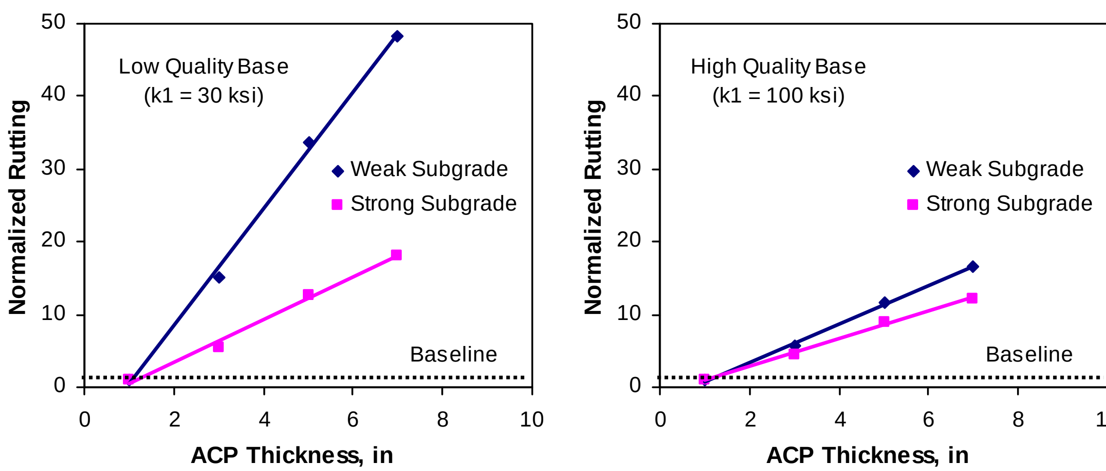
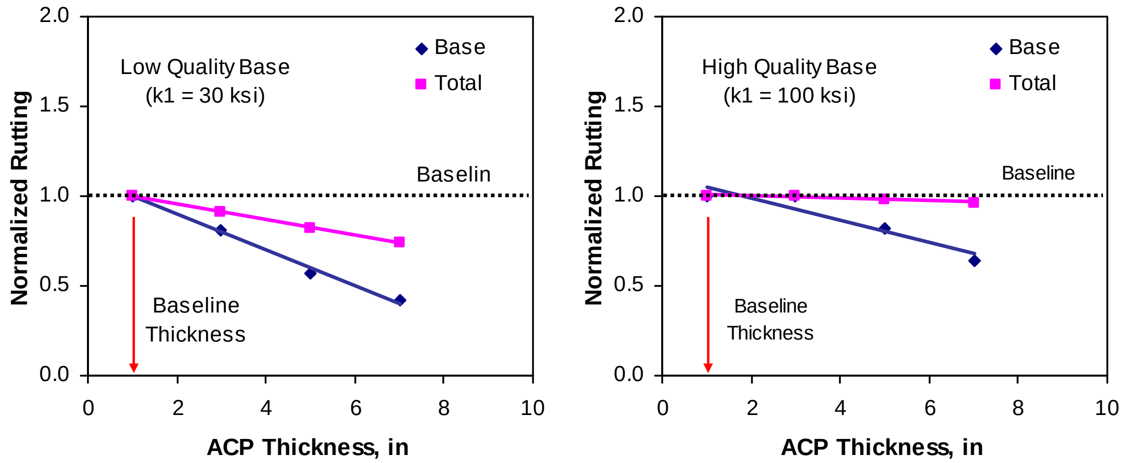
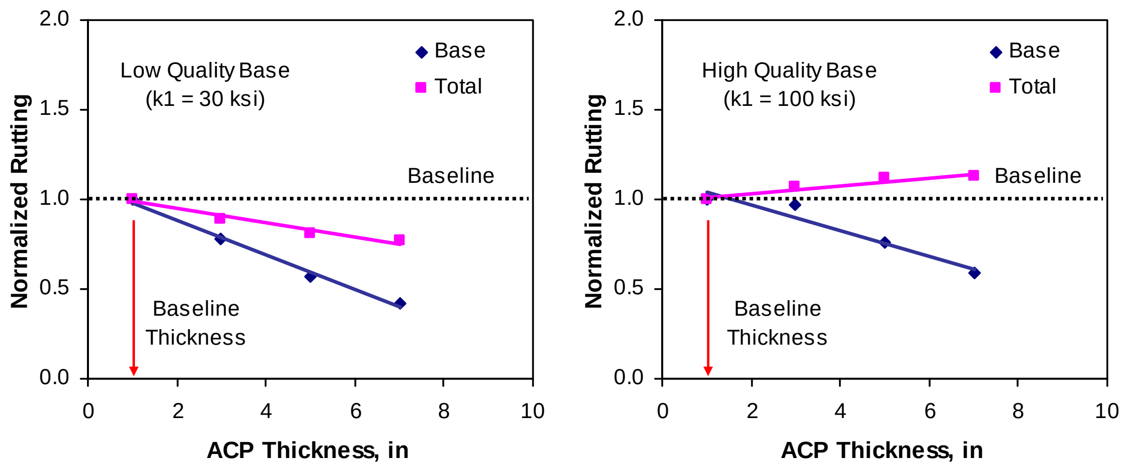



**Figure 5.3 - Variations in Rutting in ACP Layer with ACP Thickness**

From Figure 5.3, as the thickness of the ACP layer increases, the rutting in that layer increases. The ACP rutting is significantly more pronounced for the low quality bases and weak subgrades. On the other hand, the rutting in base layer decreases as ACP thickness increases, and is less affected by the quality of the base and subgrade (see Figure 5.4). When the pavement is built with a weak base, the increase in the ACP layer would result in a decrease in total (ACP plus base and subgrade) rutting. However for a strong base layer, the total rutting is less dependent of the ACP layer thickness. This means that if a strong base is used, the use of a thick ACP may not be necessary for controlling pavement rutting.




a) Pavement on Weak Subgrade (Modulus = 12 ksi)




b) Pavement on Strong Subgrade (Modulus = 30 ksi)

**Figure 5.4 -Variations in Rutting with ACP Thickness**

## Base Layer

The impact of $k_1$ on the rutting of base layer and the total rutting of pavement structure is shown in Figure 5.5. For a high quality base, $k_1$ has less influence on the rutting of base layer and the total rutting of pavement structure; but for a low quality base, $k_1$ has a significant effect on both of them. In addition, for a pavement with a high quality base, the rutting in the base and the total rutting of the pavement are less affected by the quality of the subgrade.

The influences of nonlinear parameters $k_2$ and $k_3$ are shown in Figures 5.6 and 5.7, respectively. As $k_2$ increases, the rutting of base layer and the total rutting of pavement structure decrease. Parameter $k_3$ also shows the same trend; i.e., as the value $k_3$ increases (its absolute value becomes smaller), less rutting is anticipated. In resilient modulus tests, $k_2$ increases and the absolute value of $k_3$ decreases as the percent fines decreases. This indicates that for two bases with similar stiffness, the one with lower fine contents should result in a lower rutting.

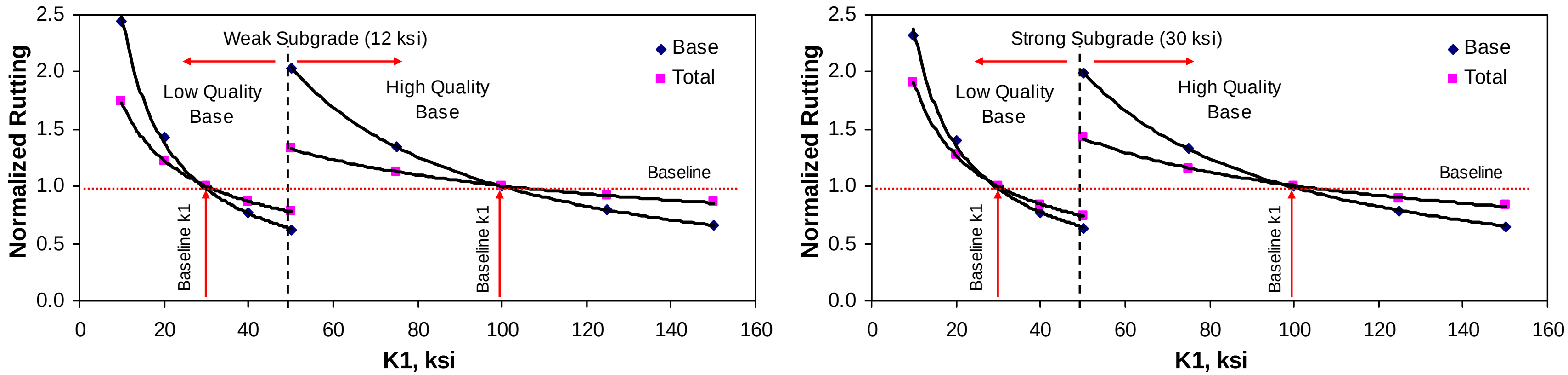
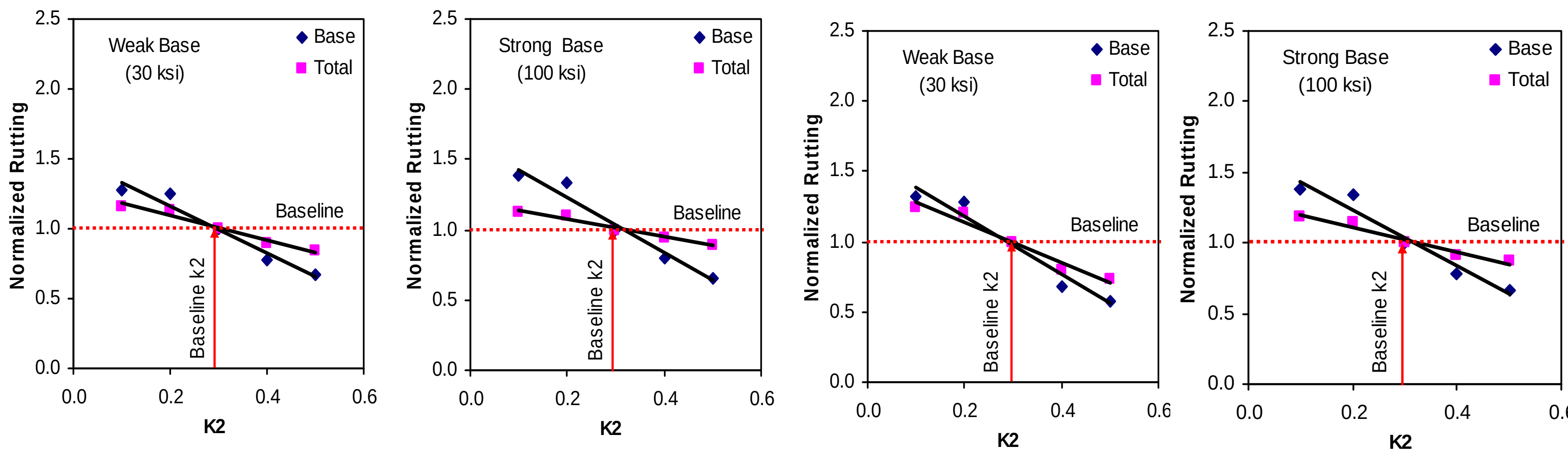



**Figure 5.5 - Variations in Rutting with Parameter $k_1$ of Base**




a) Pavement on Weak Subgrade (12 ksi)

b) Pavement on Strong Subgrade (30 ksi)

**Figure 5.6 - Variations in Rutting with Parameter $k_2$ of Base**

The impacts of $\mu$ and $\alpha$ on the rutting of base layer and the total rutting of pavement structure are shown in Figures 5.8 and 5.9, respectively. With the increase in $\alpha$, the rutting of both, the base layer and the overall pavement structure decreases. Significant increase in rutting is observed when $\alpha$ is lower than 0.7. As $\mu$ increases, the rut depth in the base layer and the overall pavement structure increases as well. Significant increase in rutting is observed when $\mu$ is greater than 0.1.

The impact of base thickness on the rutting of pavement structure is shown in Figure 5.10. For a weak base, increase in base thickness results in more rutting in the base layer as well as the total rutting of pavement structure. On the other hand, for a strong base, increase in base thickness decreases the total rutting of pavement structure.

**Utilization of Subbase**

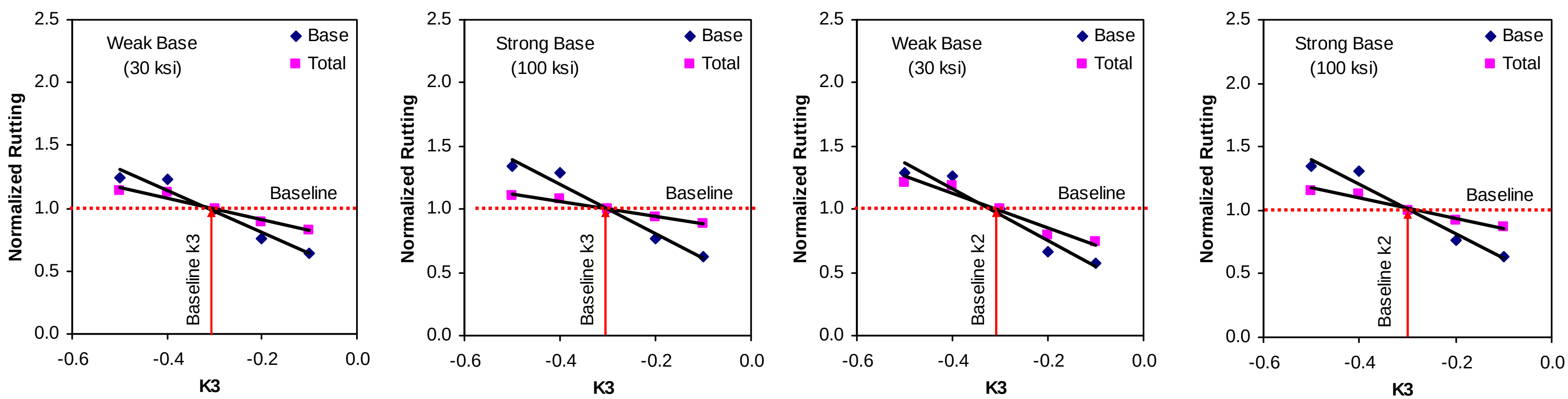
One possible way to utilize the lower quality local materials is to use them as a subbase layer. For simplicity, it is assumed that the base course consists of a 6-in. thick high quality (HQ) layer over a 6-in. thick low quality (LQ) layer and that the values of $k_1$ are 100 ksi for the high quality layer and 30 ksi for the low quality layer. The influences of $\alpha$ and $\mu$ on the rutting of the combined base and subbase course are shown in Figures 5.11 and 5.12, respectively. For comparison, the results from analyses on the base course of the same thickness but consisting of low quality material only and high quality material only are also included these two figures.

The results shown in Figures 5.11 and 5.12 indicate that the use of a low quality material as the subbase does not result in a significant rutting increase in the entire base course as compared with the rutting for the base course built with a high quality material only. This statement is almost independent of the subgrade modulus. Practically speaking, as long as the top 8 in. to 10 in. of the base is high quality and rut resistant, the reminder of the base thickness is only necessary to control the rutting of subgrade and can be potentially of the lower quality. Even though these results are not verified in this study, a number of cases in TxDOT database of successful pavements points to the same conclusions.

**Subgrade**

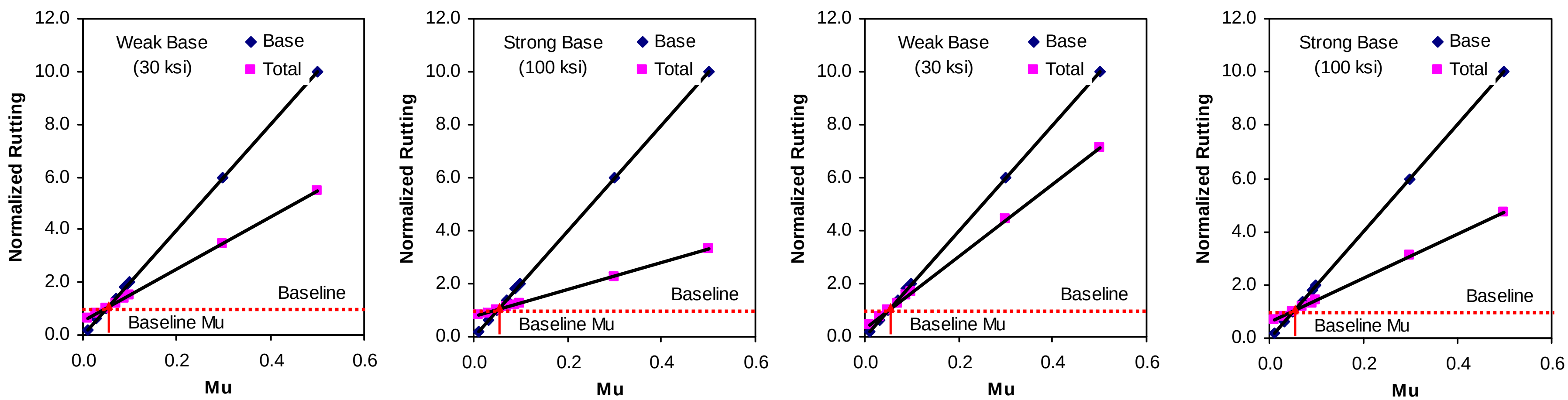
The impact of subgrade modulus on the rutting of base and overall pavement structure is shown in Figure 5.13. As the modulus of the subgrade increases, the total rutting of the pavement structure decreases. The quality of subgrade has little influence on the rutting of the base layer. But it impacts the total rutting of the pavement structure, mostly due to increase in the subgrade rutting.

In general, the conclusions from this sensitivity study can be summarized in Table 5.4:



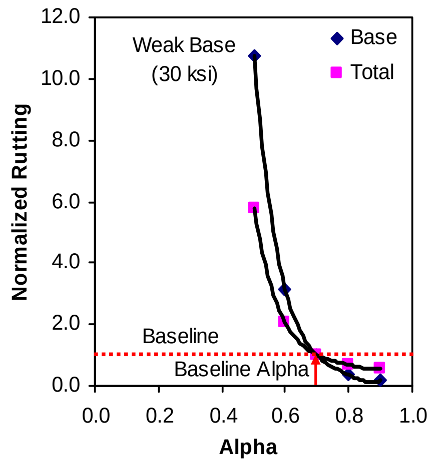

a) Pavement on Weak Subgrade (12 ksi)    b) Pavement on Strong Subgrade (30 ksi)

**Figure 5.7 - Variations in Rutting with Parameter k3 of Base**

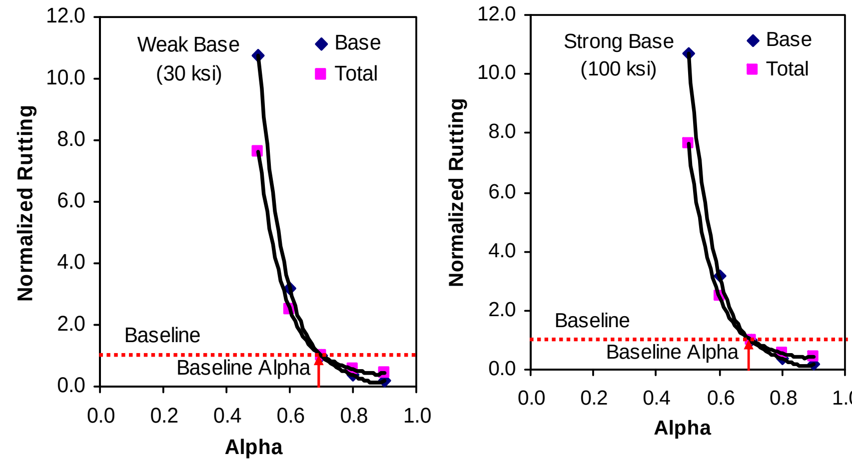



a) Pavement on Weak Subgrade (12 ksi)    b) Pavement on Strong Subgrade (30 ksi)

**Figure 5.8 - Variations in Rutting with Parameter μ of Base**

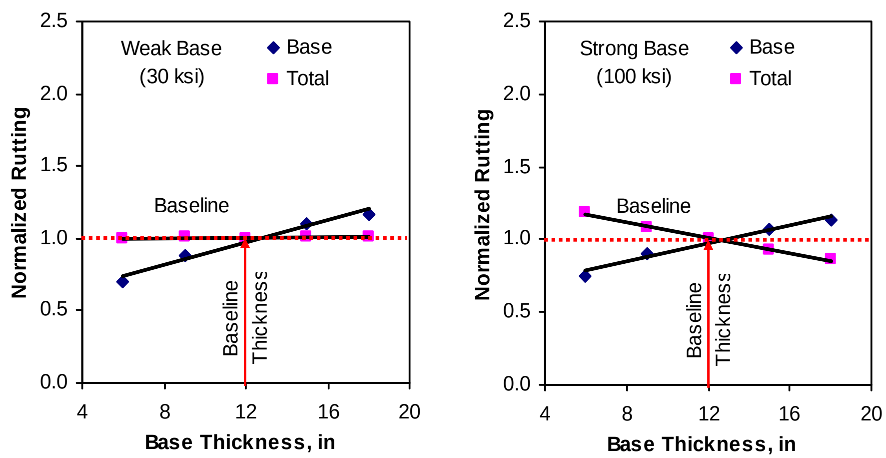



a) Pavement on Weak Subgrade (12 ksi)

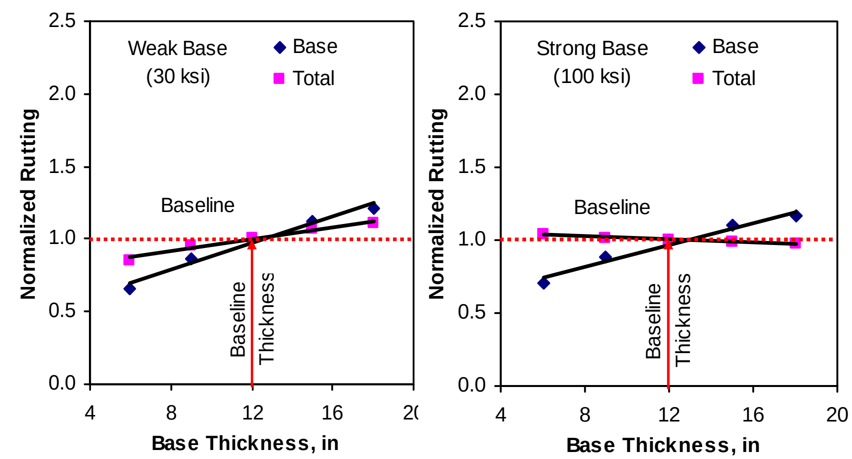



b) Pavement on Strong Subgrade (30 ksi)

**Figure 5.9 - Variations in Rutting with Parameter Alpha of Base**




a) Pavement on Weak Subgrade (12 ksi)




b) Pavement on Strong Subgrade (30 ksi)

**Figure 5.10 - Variations in Rutting with Thickness of Base**

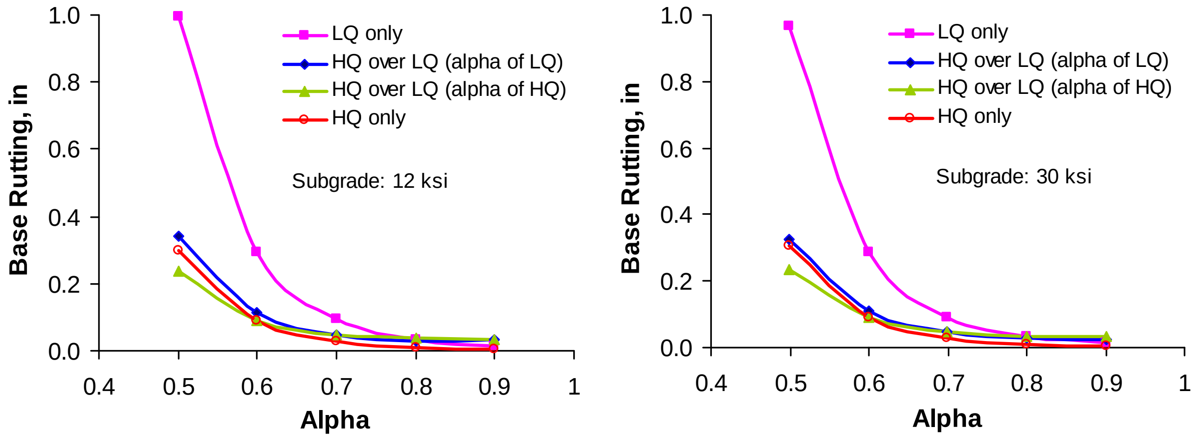



**Figure 5.11 - Comparison of Variations in Rutting with Parameter Alpha for Base Courses Consisting of One Layer and Two Layers**

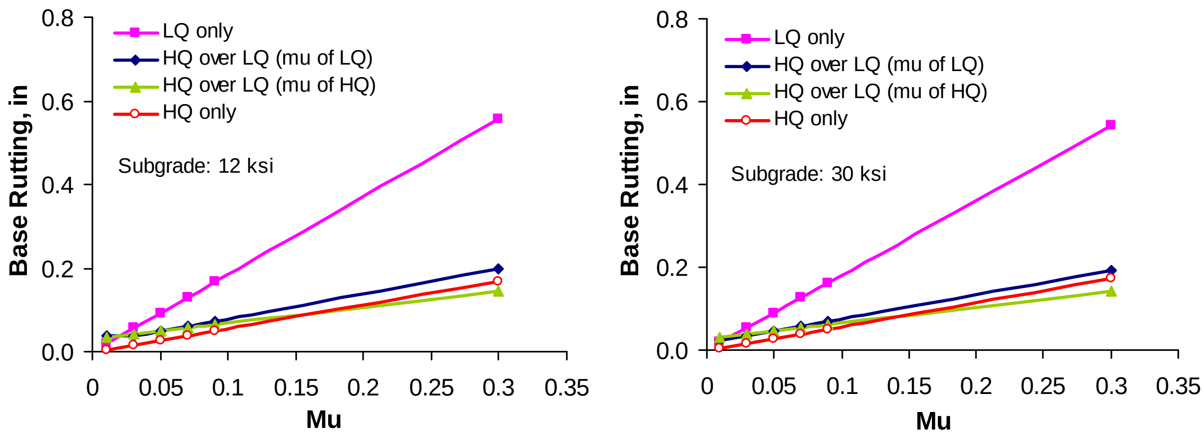
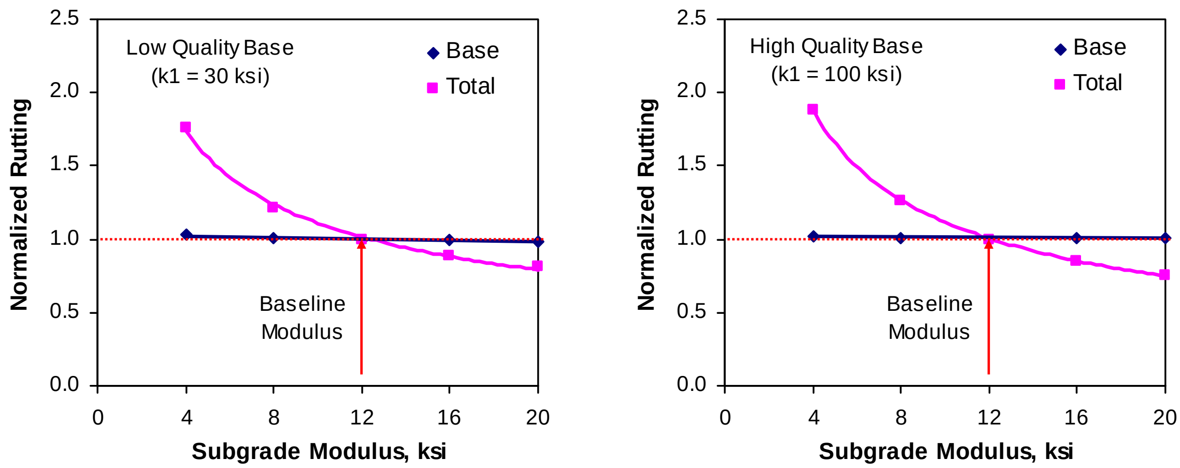
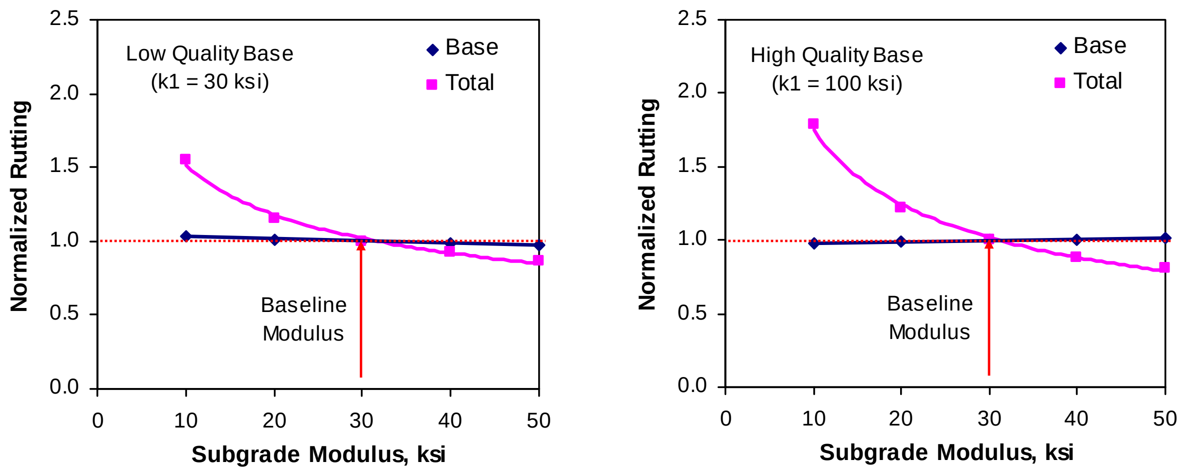



**Figure 5.12 - Comparison of Variations in Rutting with Parameter Mu for Base Courses Consisting of One Layer and Two Layers**




a) Pavement on Weak Subgrade




b) Pavement on Strong Subgrade

**Figure 5.13 - Variations in Rutting with Subgrade Modulus**

**Table 5.4 - Parameters Sensitive to the Rutting of Base and the Total Rutting of Pavement**

| Layer | Parameters | Base Rutting | Total Rutting |
|---|---|---|---|
| AC* | Thickness | Not Sensitive | Not Sensitive |
| Base | k1 | Sensitive | Sensitive |
| | k2 | Not Sensitive | Not Sensitive |
| | k3 | Not Sensitive | Not Sensitive |
| | Alpha | Very Sensitive | Very Sensitive |
| | Mu | Very Sensitive | Very Sensitive |
| | Thickness | Not Sensitive | Not Sensitive |
| Subgrade | Modulus | Not Sensitive | Sensitive |

* Rutting in ACP layer is very sensitive to ACP thickness

## Determination of Equivalent Layer Thickness

The main principle of structural equivalency is that the pavements constructed with high-quality material and low-quality material should experience the same amount of surface layer fatigue cracking, subgrade rutting as well as experiencing acceptable rutting in the base and pavement structure. The first two modes of failure can be checked using the FPS19. While the rutting of the base and pavement structure can only be checked using VESYS or TxIntPave.

The first step in the equivalency analysis is to determine the thickness of base layer when pavement is constructed with high-quality/hauled-in material. FPS19 program can be used for this purpose. The thickness of the base and possibly HMA can be adjusted then to achieve the equivalency in remaining life as discussed below.

### Fatigue Cracking of HMA

In principle for two alternative pavement sections to experience the same amount of fatigue cracking, the tensile strains at the bottoms of their respective HMA layers should be the same (see Figure 5.1). For a low-volume road with surface treatment, the fatigue cracking of the surface layer is not of a concern. However, if an HMA layer greater than 2 in. to 3 in. is considered for the high quality base, the thickness of the HMA has to be increased for the low quality base to achieve equivalency in the relevant tensile strains. A simple software package developed under Project 0-5223 can be used for this purpose.

### Rutting of Subgrade

In principle for two alternative pavement sections to experience the same amount of subgrade rutting, the compressive strains at the top of their respective subgrade layers should be the same (see Figure 5.1). For a low-volume road with surface treatment, this can be achieved by increasing the thickness of the low quality base relative to the high quality base. A more expensive alternative is to increase the thickness of the HMA layer. The same simple software package discussed for fatigue cracking can also be used for subgrade rutting equivalency.

### Rutting of Base

As indicated before, the thickening of the base layer with a low quality material may increase the rutting of the base in particular and the pavement system in general. As such, checking for this problem is crucial. Unfortunately, a simple equivalency relationship between the strains as discussed for the other two modes of failure is not available. Preferably, either VESYS5W or TxIntPave should be used. In the absence of these software packages, an approximate method is recommended to estimate the rutting of the base and the pavement system for roads with very thin HMA or surface treatment.

The equation for calculating the accumulated permanent strain of each individual layer, $\varepsilon_p$, after N applications of load can be written as

$$\varepsilon_p = \frac{\mu}{1-\alpha} \cdot \varepsilon_r \cdot N^{1-\alpha} \tag{5.1}$$

where $\varepsilon_r$ is called the elastic or resilient strain and $\alpha$ and $\mu$ are permanent deformation parameters obtained from laboratory tests as discussed in Chapter 3. The resilient strain, for a layered elastic system can be approximated as

$$\varepsilon_r = \frac{\sigma}{M_r} \tag{5.2}$$

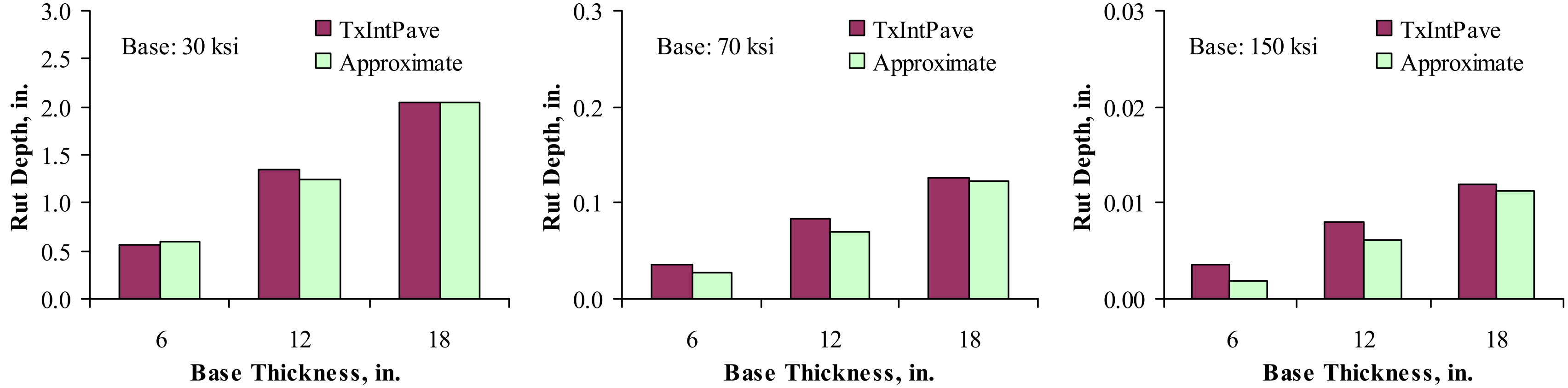
where $\sigma$ is the representative vertical compressive stress in the layer and $M_r$ is the representative resilient modulus calculated as discussed in Chapter 3. An extensive parametric study conducted by Gautum (2008) indicates that the most appropriate depth in the base to calculate these two parameters is 7 in. (or the actual thicknesses for bases that are thinner than 7 in.) for base and 12.5 in for subgrade. After the resilient strain for each layer is determined, the rutting in each layer, which is the product of permanent strain (from Equation 5.1) and the actual layer thickness, can be obtained. For estimating the rutting of the subgrade, Gautum (2008) proposes a subgrade thickness of 25 in. The total rutting in a pavement is then determined by summing the rutting of the base and subgrade.

As an example, the base rut depths obtained from TxIntPave and the approximate method are compared in Figure 5.14. The results are fairly closed for base thicknesses of greater than 6 in. (up to 18 in.) and for the base moduli ranging from 30 ksi to 70 ksi. For strong base (modulus of 150 ksi), the rutting is under estimated when base thickness is 6 in. or less.

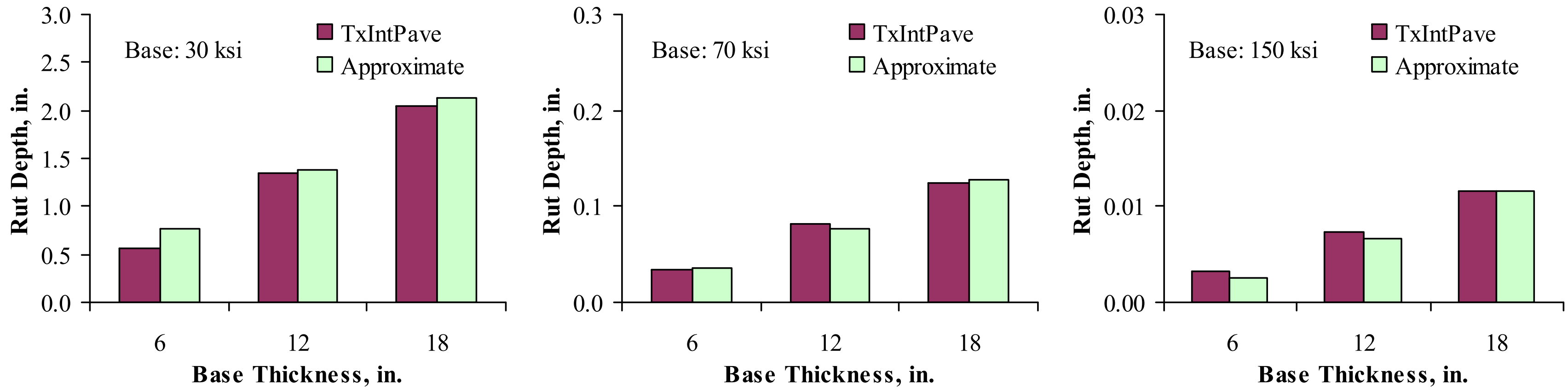
Based on this discussion, a flow chart for determining equivalent base thickness is shown in Figure 5.15 with the following criteria:

- If the rutting of high-quality material is greater than 0.5 inch, the thickness of base layer for low-quality material is adjusted until the rutting of low-quality material becomes the same or close to the rutting of high-quality material.

- If the rutting of high-quality material is less than 0.5 inch, the thickness of base layer for low-quality is adjusted until the rutting of low-quality material becomes close to 0.5 in.




a) Base with Variable Moduli on a Weak Subgrade (Modulus = 10 ksi)




b) Base with Variable Moduli on a Strong Subgrade (Modulus = 20 ksi)

**Figure 5.14 - Variations of Rut Depth in Base Layer with Base Thickness**

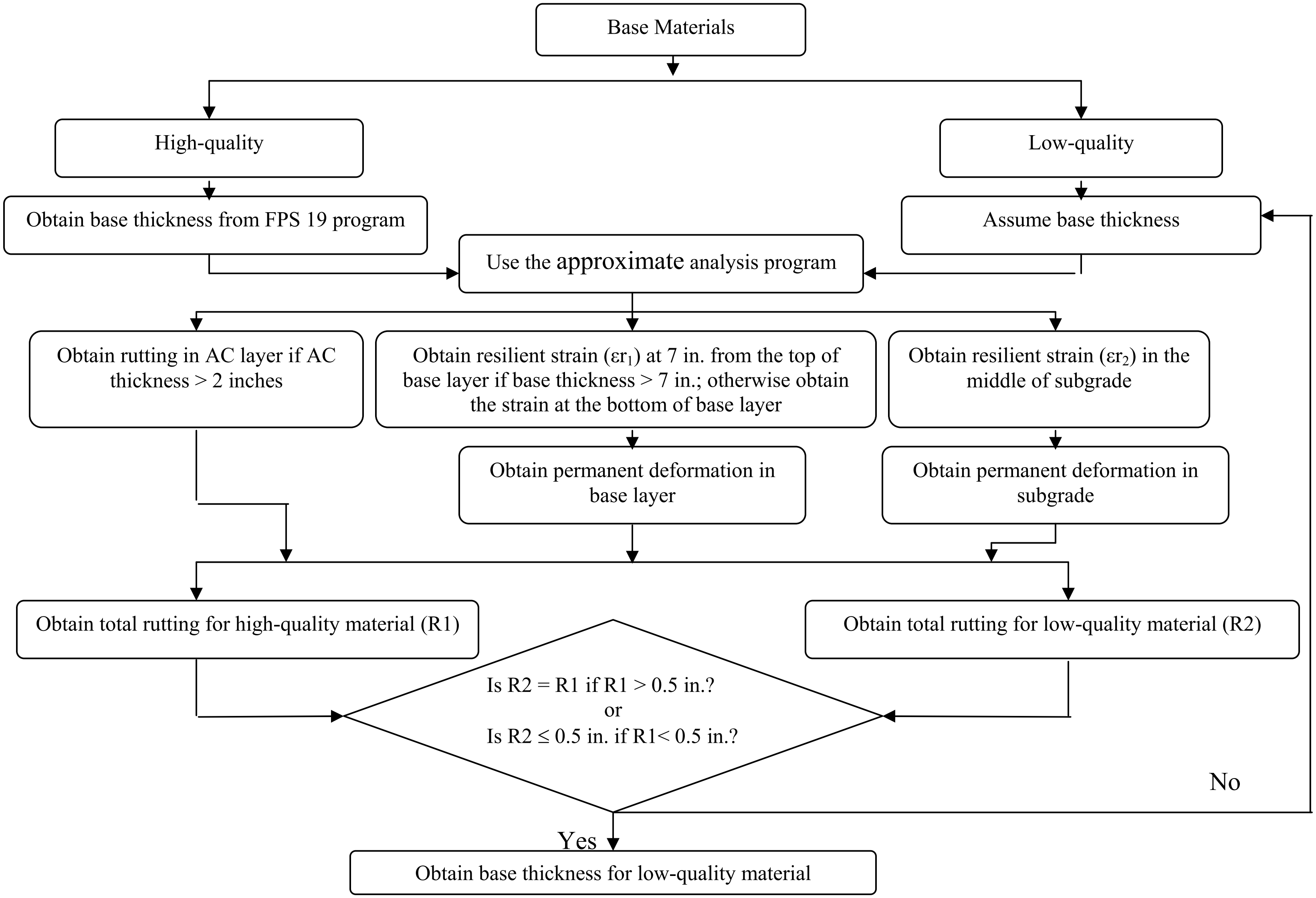



**Figure 5.15 - Flow Chart for Determining Equivalent Base Thickness**

## Result from Structural Analysis

The first step of structural analysis is to determine the base thickness from FPS 19 program for high-quality (or treated) base material. The minimum layer thickness required for each material used in this project is subjected to a low-volume traffic load (ADT = 250) over a design life of 20 years. Two different types of subgrade with the moduli of 20 ksi and 10 ksi were considered to document their impact on the base thickness. These analyses are based on the assumption that the pavement structure is only covered by a surface-treatment (no HMA layer). The default values of Poisson's ratio were used for all layers.

Table 5.9a shows the base thickness obtained from FPS19 for the high quality/treated materials. As per the Texas Triaxial design check embedded in the FPS19, the minimum base thickness for a subgrade with a modulus 20 ksi is 5.5 in., and for a subgrade modulus of 10 ksi, 14.5 in. These minimum thicknesses were enforced in Table 5.9a as well. The thickness required for high quality base is increased as the modulus of the subgrade modulus decreased. For example, the thickness required for the El Paso base increased from 12.5 in. to 14.5 in. as the modulus of the subgrade decreased from 20 ksi to 10 ksi. For the Vulcan (Brownwood) material, the quality of subgrade does not seem to impact much on the base thickness. The total rutting obtained from TxIntPave for the sections designed with the FPS19. The rut depths are rather small for all cases.

Table 5.9b contains the similar information as Table 5.9a but with the corresponding low quality base thicknesses. For practical reasons the thickness of the base was limited to 18 in. In this case the total rut depths of the pavement are significantly higher than those in Table 5.10. Most of the additional rutting can be contributed to the rutting of the base.

Figure 5.16 demonstrates typical trends of variations in rut depth with base thickness for the high-quality and low-quality base materials similar to those from Turner and Lumpkin pits, respectively, in San Angelo District. A relatively strong subgrade of 20 ksi was used for the analysis. With the increase in the high-quality base thickness, the subgrade rut depth decreases substantially while the base rut depth increases marginally. As a result, an increase in the base thickness will result in a decrease in total rutting of the pavement structure. In the case of the low-quality base, the rutting of the subgrade is reduced with the increase in base thickness. However, the base itself ruts quite substantially, resulting in a significant increase in the rut depth with the increase in base thickness. As such, increasing the thickness of the low-quality base layer to provide additional support to the subgrade is not prudent.

To test this finding, a third set of rut depth analysis with TxIntPave was carried out by using the thicknesses reported for the high-quality base but with the properties of the low-quality base. Table 5.9c contains the rut depths from this exercise. In general the rutting of the base layer decreased relative to those reported in Table 5.9b, indicating that using thinner base layers than predicted by FPS19 will reduce the base rutting. However, the total rutting increased relative to those reported in Table 5.9a, indicating that the total rutting of the sections built with the low quality base is substantially higher than the high quality base for the same cases. However, the total rut depths in Table 5.9c, for the most part, are still acceptable for low volume roads.

## Table 5.5 - Rutting Contribution for Different Base Thicknesses Required by FPS19 Design

### a) For High-Quality or Treated Base Materials

| Source of Material | Subgrade Modulus 20 ksi | | | Subgrade Modulus 10 ksi | | |
|---|---|---|---|---|---|---|
| | Base Thickness, in. | Base Rutting, in. | Total Rutting, in. | Base Thickness, in. | Base Rutting, in. | Total Rutting, in. |
| ELP (Cemex)* | 12.5 | 0.01 | 0.04 | 14.5 | 0.01 | 0.06 |
| LBB (Caddell)* | 7.5 | 0.00 | 0.05 | 14.5 | 0.00 | 0.05 |
| ABL (Old Bobby Noble)* | 6.5 | 0.02 | 0.07 | 14.5 | 0.03 | 0.07 |
| BWD Medium (Prater)* | 9.0 | 0.01 | 0.04 | 14.5 | 0.01 | 0.06 |
| BWD Good (Prater)* | 7.0 | 0.00 | 0.05 | 14.5 | 0.00 | 0.04 |
| BWD (Vulcan)* | 14.0 | 0.01 | 0.04 | 15.0 | 0.01 | 0.07 |
| SJT (Turner) | 12.5 | 0.01 | 0.04 | 14.5 | 0.01 | 0.06 |

***: Treated with additive**

### b) For Low-Quality (Raw) Base Materials

| Source of Material | Subgrade Modulus 20 ksi | | | Subgrade Modulus 10 ksi | | |
|---|---|---|---|---|---|---|
| | Base Thickness, in. | Base Rutting, in. | Total Rutting, in. | Base Thickness, in. | Base Rutting, in. | Total Rutting, in. |
| ELP (Cemex) | 18.0 (19.5) | 0.27 | 0.31 | 18.0 (19.5) | 0.27 | 0.32 |
| LBB (Caddell) | 18.0 (20.5) | 0.91 | 0.94 | 18.0 (20.5) | 0.89 | 0.94 |
| ABL (Old Bobby Noble) | 18.0 (23.5) | 0.13 | 0.16 | 18.0 (24.5) | 0.12 | 0.18 |
| BWD Medium (Prater) | 12.0 | 0.13 | 0.16 | 14.5 | 0.10 | 0.16 |
| BWD Good (Prater) | 12.0 | 0.11 | 0.14 | 14.5 | 0.09 | 0.14 |
| BWD (Vulcan) | 16.5 | 0.14 | 0.17 | 17.5 | 0.14 | 0.19 |
| SJT (Lumpkin) | 18.0 (22.0) | 0.04 | 0.08 | 18.0 (22.5) | 0.04 | 0.10 |

**Note: Thicknesses provided in the parentheses are the actual thicknesses obtained from FPS 19**

### c) For Low-Quality Materials with Layer Thicknesses from Corresponding High Quality Materials

| Source of Material | Subgrade Modulus 20 ksi | | | Subgrade Modulus 10 ksi | | |
|---|---|---|---|---|---|---|
| | Base Thickness, in. | Base Rutting, in. | Total Rutting, in. | Base Thickness, in. | Base Rutting, in. | Total Rutting, in. |
| ELP (Cemex) | 12.5 | 0.19 | 0.23 | 14.5 | 0.21 | 0.28 |
| LBB (Caddell) | 7.5 | 0.39 | 0.46 | 14.5 | 0.71 | 0.78 |
| ABL (Old Bobby Noble) | 6.5 | 0.05 | 0.12 | 14.5 | 0.10 | 0.17 |
| BWD Medium (Prater) | 9.0 | 0.07 | 0.12 | 14.5 | 0.10 | 0.16 |
| BWD Good (Prater) | 7.0 | 0.06 | 0.12 | 14.5 | 0.09 | 0.14 |
| BWD (Vulcan) | 14.0 | 0.11 | 0.15 | 15.0 | 0.11 | 0.17 |
| SJT (Lumpkin) | 12.5 | 0.03 | 0.08 | 14.5 | 0.03 | 0.11 |

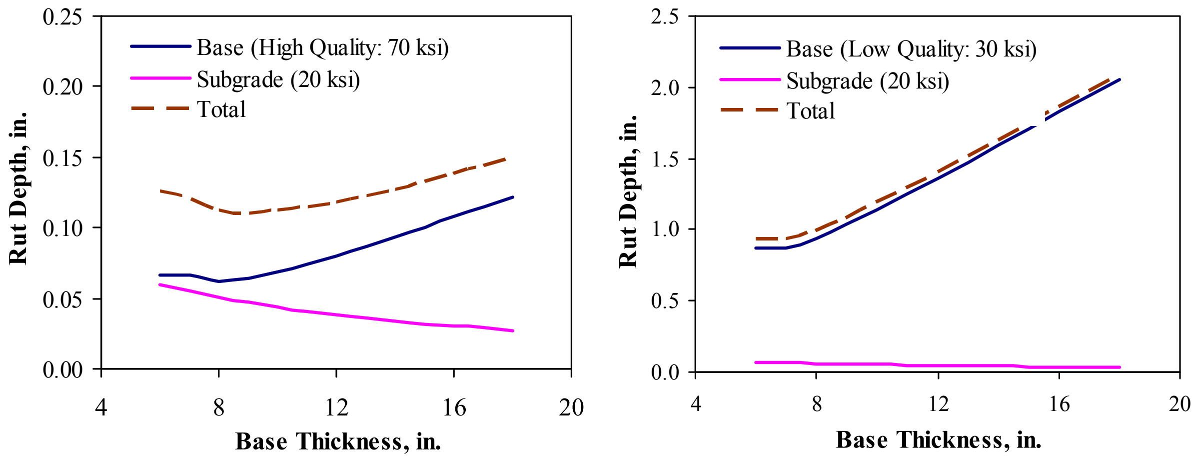



**Figure 5.16 - Typical Trend of Rut Depth for Bases of High and Low Quality Materials**

## Cost Analysis

To better understand the savings between the costs of construction using local materials and hauled-in materials, especially when treatment or modification of gradation is involved, several associated costs such as material cost, construction cost and transportation cost should be considered. Figure 5.17 shows a simple program developed in excel to evaluate the cost of construction using local base material for roadway base and subbase as discussed next. Appendix A contains detailed explanation about this program.

**Methodology of Cost Analysis**

The first step in cost analysis is to locate the sources of the high-quality and the low-quality/local base materials. The main consideration in the analysis is that the travel distance to haul in the high-quality base material is farther than the travel distance for the low-quality/local base material. Based on this criterion, it is assumed that the transportation cost for the high-quality base material is more than the transportation cost for the low-quality/local base material.

The second step of the analysis consists of estimating the volume of the material that is needed to construct the base knowing the length, width and the thickness of the base layer. The most important parameters in the cost analysis are the cost of material, the cost of construction and the cost of transportation. It is assumed that the material cost of the low-quality/ local base material is lower than the cost of the high-quality/hauled in base material. Since, as discussed in Chapter 4, the low-quality/local base material generally requires treatment to improve its properties, it is assumed that the construction cost of pavements with the low-quality/local base material is higher than the construction cost with the high-quality/hauled-in base material.

Finally, with all the parameters mentioned above, the total costs of the high-quality/hauled-in materials and the low-quality/local materials are determined. Based on the total cost, cost saving using the high-quality or low-quality/local base material is analyzed. A graph that provides the breakeven point on the extra distant that the high quality base should be hauled is also provided (see Figure 5.17).

| **1) Project Information** | | *Example* |
|---|---|---|
| **Sample ID:** | | **I 10 Sample 1** |
| **Sample Date:** | | **7/25/2008** |
| **Controlling CSJ:** | | **000-00-000** |
| **County:** | | **El Paso** |
| **District:** | | **El Paso** |
| **Sampled by:** | | **Biraj Gautam** |
| **Sample Location (Quarry):** | | **Cemex** |
| **Distance For Hauling High Quality Base, mile** | **70** | **50** |
| **Distance For Hauling Local Base, mile** | **30** | **50** |

| **2. Pavement Section** | |
|---|---|
| **Length of the pavement section, mile** | **10** |
| **Number of lanes** | **2** |
| **Width of the lane, ft.** | **12** |
| **Width of the shoulder, ft.** | **4** |

| **3. Base Layer Information** | **High Quality Base** | **Local Base** |
|---|---|---|
| **Thickness of base layer, in** | **8** | **12** |
| **Density of base, lb/ft3** | **135** | **135** |

| **4. Cost Information** | **High Quality Base** | **Local Base** |
|---|---|---|
| **Material cost per SY for 12 in. base, $** | **7** | **6** |
| **Construction cost per SY, $** | **7** | **9** |
| **Transportation cost per ton per mile, $** | **0.25** | |

**5. Results**

| | **High Quality Base** | **Local Base** |
|---|---|---|
| **Total Cost** | **$ 3,534,119** | **$ 3,685,267** |

**Local base is 4% more expensive. Therefore, High Quality Base is more economical**

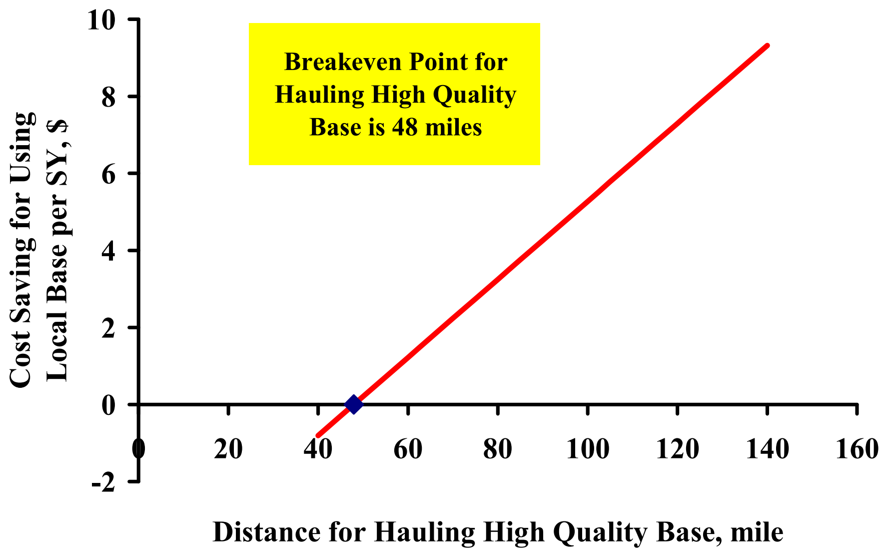



**Figure 5.17 – Worksheet of Cost Analysis for Roadway Base Construction with Low and High Quality Materials**

# Chapter 6

# Guidelines and Protocols

## Introduction

The main purpose of this chapter is to provide guidelines and strategies for using local materials in the base and/or subbase construction of low-volume roads.

To establish test procedures and guidelines, nine base materials from five TxDOT districts were collected and tested. The major laboratory tests on these materials include:

- Triaxial Compression
- Tube Suction related such as retained strength, retained modulus, moisture change and dielectric constant
- Resilient Modulus and Permanent Deformation
- Small-scale performance simulation

Based on the results (see Chapter 4), the chemical treatment or gradation modification or both were applied to six materials to ensure that the materials classified as out-of-specification can be economically improved to meet or be close to the requirements for Grade 1 of Item 247.

The decision for selecting the appropriate types of additive was based on the current TxDOT guidelines (Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures, 2005) and the gradation modification was based on the findings from TxDOT Project 0-4348.

## Guidelines and Test Protocols

The requirements for Grade 1 base materials by TxDOT Item 247 are the basis of evaluating and using local pit materials for roadway base and subbase. The flow chart of activities is shown in Figure 6.1 and detailed step by step process in the following paragraphs. Two strategies are proposed for improving the out-of-specification local bases: (1) Improving Gradation and (2) Chemical Treatment with Calcium-based Additives (limited to 2% for economical reasons).

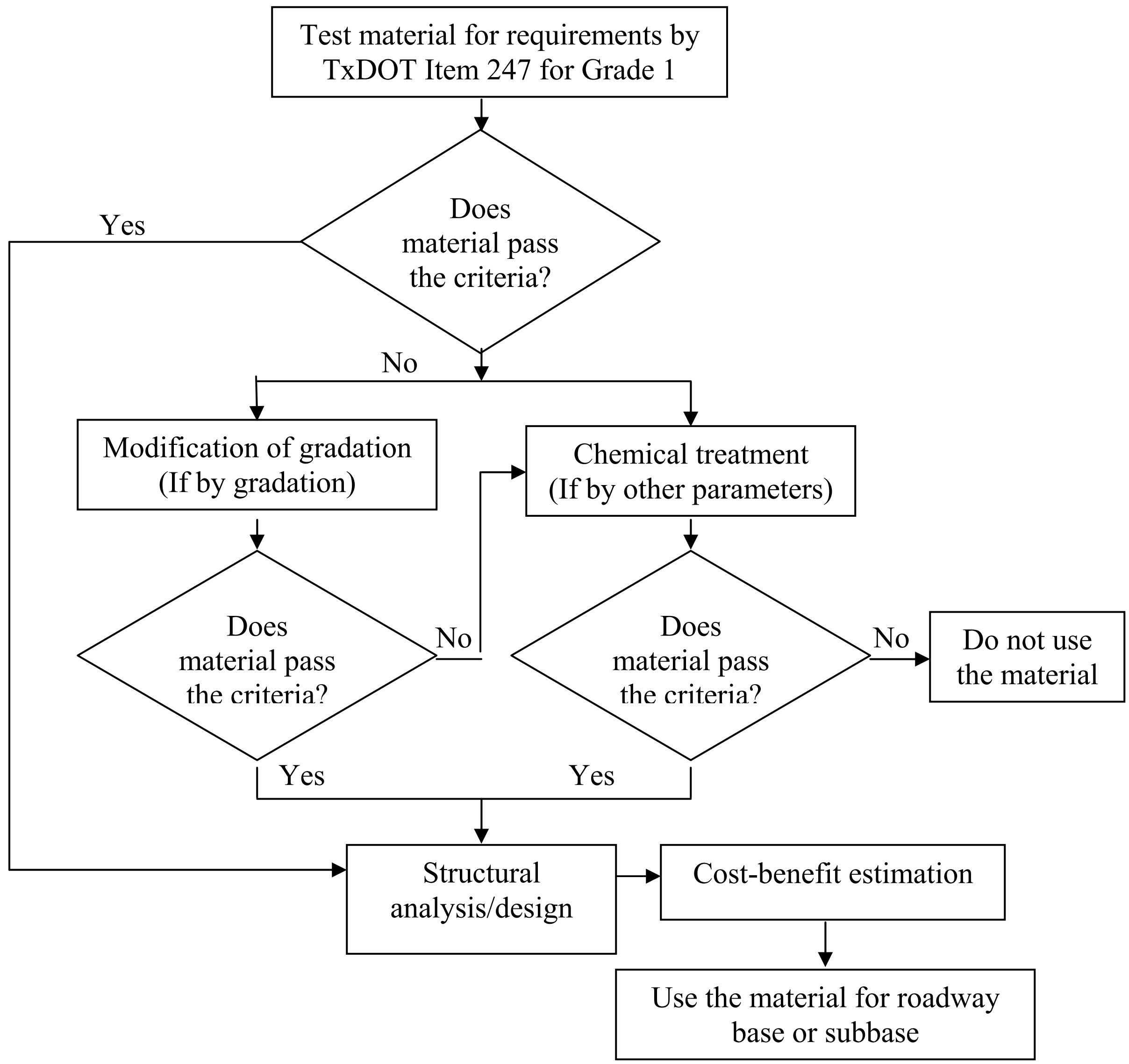



**Figure 6.1 - Flow Chart of Test and Evaluation Protocols**

**1. Sieve Analysis**

The sieve analysis is carried out as per Tex-110-E except that a No. 200 sieve should be added to the sieve stack. The gradation curve is compared to the Item 247 Grade 1 requirements.

*If the gradation is slightly or partially out of Item 247 limits, particularly, for No. 40 sieve, the modification of gradation may be an option. Also if more than 15% of the material is finer than No. 200, the modification of gradation may be considered.*

## 2. Atterberg Limits

Liquid Limit (LL) and Plastic Limit (PL) of material sample are tested as per Tex 104-E and Tex-105-E. Plasticity Index (PI) of the material is calculated as per Tex-106-E.

*The LL should be less than 35 and the PI should be less than 10. If LL or PI or both are out of these limits, chemical treatment is recommended.*

## 3. Moisture-Density

Moisture-density (MD) test is carried out as per Tex-113-E to obtain the optimum moisture content and the maximum dry density.

***Optional Step****: To estimate the variations in strength with moisture, the specimens prepared for developing the MD curves can be cured for 24 hours and subjected to unconfined compressive strength (UCS) testing as per Tex-117-E. If the UCS at optimum is significantly less than the limits set for strength at 0 psi lateral pressure, treatment is recommended.*

## 4. Strength

Testing for compressive strength of a specimen should conform to procedure Tex-117-E. Strength testing as per proposed Tex-143 is optional.

*If one of the strengths at 0 and 15 psi lateral pressures as per Tex-117-E does not meet the requirement of Item 247, chemical treatment is recommended.*

## 5. Moisture Susceptibility

The retained strength defined as the ratio of the strength obtained from zero lateral pressure after moisture conditioning as per Tex-117-E and strength at zero lateral pressure after 24 hrs of curing at room temperature (similar to Tex-143) should be the primary parameter for assessing the moisture susceptibility.

*The retained unconfined compressive strength should be greater than 80%. If the retained strength is less than 80%, chemical treatment is recommended.*

Tube Suction Test (TST) as per Tex-144-E is recommended for secondary assessing the moisture susceptibility of *untreated* material through dielectric constant measurement.

*If dielectric constant is greater than 16, chemical treatment is recommended, depending upon the strength values as per Tex 117-E.*

## 6. Resilient Modulus and Permanent Deformation

***In order to ensure the performance of the local base, the resilient modulus and permanent deformation tests should be mandatory***. Depending on the availability of the equipment, these tests should be carried out in-house or should be performed by a commercial laboratory. These values are required for structural analysis. The additional cost associated with this task is justified to ensure that the local base will not experience excessive permanent deformation. The resilient modulus test should be performed as per AASHTO T-307.

*A representative[1] resilient modulus greater than 40 ksi is recommended.*

The modulus test can be performed with a free-free resonant column (FFRC) device as per Tex-149-E as a preliminary estimate.

*A seismic modulus of at least 80 ksi is proposed at this time.*

The permanent deformation should be conducted as per NCHRP 1-29.

*The primary reason for conducting the permanent deformation tests is to obtain parameters needed for assessing the rutting of the base as discussed before. As such it is difficult to set acceptable limits. Usually, permanent deformation in excess of 2% may be considered excessive without structural analysis as discussed in Step 8.*

**7a. Chemical Treatment**

Determine the type and amount of additives as discussed below. Repeat Steps 3 through 6.

**a) Type of Additive** - The decision tree (see Figure 6.2) for selecting the appropriate types of additive as per current TxDOT guidelines (Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures, 2005) should be followed. The two main factors considered are the percentage of material passing the No. 200 sieve and the Plasticity Index (PI). If the PI is greater than 10 by a large margin, the use of lime is recommended.

**b) Amount of Additive** – For economical reasons, the percentage of additive should not exceed 2% by dry weight of the material being tested. Since the amount of additives used is small, the strength parameters of the treated materials should be obtained as per Tex-117-E (instead of Tex-120-E or Tex-121-E or Tex-127-E) with specified limits reflected in Item 247.

*If the treated material does not satisfy the strength requirements of Item 247, the use of greater amount of additives can be considered, if deemed economical. In that case, procedures Tex-120-E, Tex- 121-E or Tex-127-E may be conducted depending on the type of additives used.*

---

[1] Representative modulus is estimated at a confining pressure and a deviatoric stress representative of the middle of the base layer due to an 18-kip equivalent single axle load. The typical values of the confining pressure and deviatoric stress of 15 psi and 15 psi, are recommended for a typical base.

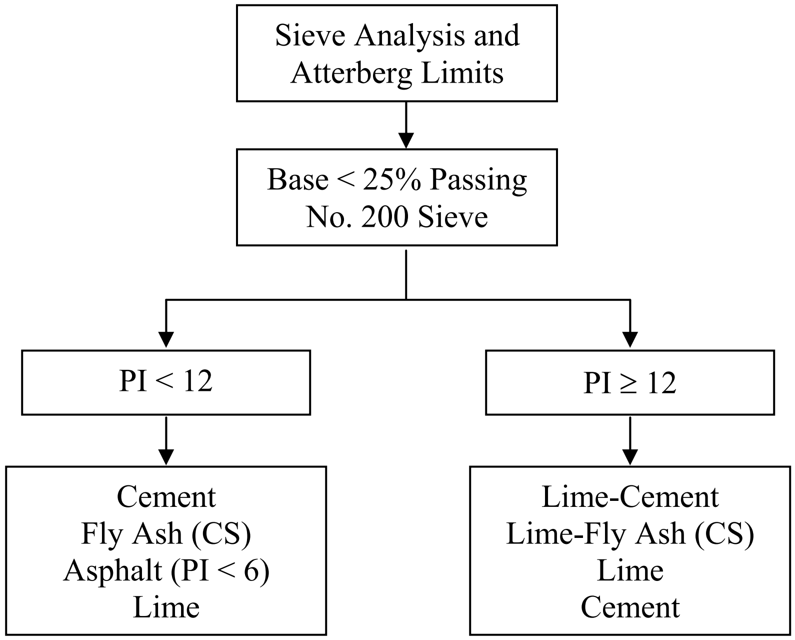



**Figure 6.2 – Decision Tree for Selecting Type of Additive**

**7b. Modification of Gradation**

Based on the result of sieve analysis in Step 1, the gradation of the material should be changed so that it would conform to the Grade 1 requirements of Item 247. If the percent passing No. 200 is substantially more than 12%, consider reducing the fine content of the mix.

*The viability of the new gradation should be evaluated following Steps 3 to 6 above.*

**8. Structure Evaluation**

The thickness requirements for the base course with the local materials should be carefully evaluated to ensure that the base layer is stable in terms of rutting. As documented in this report, the current TxDOT design procedures (i.e. FPS19 or Texas Triaxial method) not only do not address this mode of failure, they provide thicknesses that may further aggravate it. Two other software packages that can address this mode of failure are available. Either VESYS (available from TTI) or TxIntPave (available from UTEP) should be used for this purpose. In the absence of these programs, an approximate method is proposed in Chapter 5 can be used as a preliminary check.

*These programs require inputs that can only be obtained utilizing the Permanent Deformation tests discussed in Section 6. The use of presumptive values for the required parameters is strongly discouraged.*

## 9. Cost Analysis

The cost of additional processing and construction steps of the local bases (either through change in gradation or chemical treatment) should be compared with the additional cost of the transportation of the high-quality bases. Due to the extremely volatile costs of construction materials and fuel, it would be difficult to provide rigid guidelines. A worksheet, specifically developed for this purpose (see Appendix A for its user manual), can be used to determine the cost effectiveness of the use of local bases.

## 10. Use of Local Materials as Subbase

For bases thinner than 12 in., the use of the local materials without appropriate modification is not prudent. If the base is thicker than 12 in., the structural and economical feasibility of using the local material as is as a subbase should be explored. Structural analysis as part of this research indicates that most of the base rutting occurs in the top 7 in. of the base, and that a subbase layer with local materials between the base and subgrade do not significantly impact the performance of the pavement.

# Chapter 7

# Summary and Conclusions

The use of high-quality materials is generally required to satisfy conventional specifications for unbound granular materials in pavements. The source of these materials can be a long distance from the construction site, resulting in high transportation costs. The use of local sources of marginal materials or the use of low-quality materials is not allowed if they do not comply with existing specifications. Since the reserves of high-quality materials are diminishing in many areas, it is necessary to develop strategies for utilizing local materials.

In this research project, we attempted to first characterize marginal materials from several local pits to determine the reasons for these materials being considered low-quality or out-of-specification. TxDOT Item 247 for a Grade 1 base was considered as the target. The materials that failed to meet the requirements of this target were then treated with calcium-based additives or their gradations were modified to "improve" their quality. The decision for selecting the appropriate types of additive was based on the current TxDOT guidelines (Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures, 2005) and the gradation modification was based on the findings from TxDOT Project 0-4348.

Since Item 247 does not contain performance-based criteria, the strength, modulus and permanent deformation characteristics of these materials were studied to estimate the performance of these materials. These performance parameters were used in several structural analysis software packages to judge when and how to utilize the local materials instead of higher quality, hauled-in, materials. Based on the results of this evaluation, preliminary guidelines and test protocols were developed.

Based on the knowledge gained so far, the following observations were obtained for low-quality and treated base materials:

- For all the cement-treated base materials (with 1% cement) and most of the lime-treated base materials, the strength values at 0 psi lateral pressure and 15 psi lateral pressure met the Item 247 minimum strength requirements of 45 psi and 180 psi, respectively.
- For most materials, the change in gradation did not significantly improve the quality of low-quality local material.

- Although most of the raw, gradation-modified and treated materials passed the retained modulus of 80% and the retained strength of 80%, the retained modulus and the retained modulus largely depended upon the type of material.
- The dielectric constants for the materials with increased fines content showed higher values than the ones obtained from raw materials; whereas for the materials with reduced fines content, the dielectric constants showed lower values than those obtained from raw materials. For most of the materials treated with 1% cement or 1% lime, the dielectric constants showed higher values as compared with the ones obtained from raw materials.
- Most of the raw materials did not meet the resilient modulus of 40 ksi. The resilient modulus of 40 ksi for cement-treated and lime-treated materials was readily achieved. For most materials, the resilient modulus of cement-treated base was found out to be higher than the ones obtained from lime-treated base materials.
- The equivalent thicknesses of base layer were determined by using the resilient modulus and permanent deformation parameters obtained from laboratory test. The permanent deformation test parameters $\alpha$ and $\mu$ of the base layer controls the amount of rutting that a section experiences. It is of utmost importance to obtain these parameters before replacing a high-quality base with lower quality local bases.
- The current pavement design algorithms (e.g. FPS19 or Texas Triaxial Design check) should be used with utmost care to replace a high-quality base with a lower quality base. These algorithms tend to provide base thicknesses that are over conservative. More advanced analysis with either VESYS or TxIntPave is required to ensure the stability of the low quality base.
- The utilization of the lower quality local materials as a subbase layer seems feasible and advantageous. The top 6 in. to 8 in. of the base layer placed on the low-volume road with thin surfacing or surface treatment seems to contribute to the rutting. As such for bases thicker than 12 in. the use of the low quality local base as subbase, especially for strong subgrades, is recommended.

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# Appendix A

## Cost Evaluation Tool Manual

## Introduction

The Cost Evaluation Tool was developed in Microsoft Excel in order to understand the savings between the costs of construction using local materials and hauled-in/high-quality materials, especially when treatment or modification of gradation is involved. In order to estimate the cost of construction using high-quality/hauled-in materials and treated or untreated local materials, several associated costs such as material cost, construction cost and transportation cost are considered. A detailed discussion about the use of the program is presented in this tool manual.

## Program Description

The Cost Evaluation Tool is composed of two Excel worksheets: 1) 'Input and Output' sheet and 2) 'Calculation' sheet. In the 'Input and Output' sheet, necessary inputs such as the dimensions of the pavement section, the base layer information and the cost information are provided. With the information provided as input, the program calculates the cost of construction using high quality base and local base materials. Finally, results in terms of cost comparison between the use of two different base materials, high quality and local base, are shown in the same excel sheet.

In the Calculation sheet, necessary calculations are shown in order to evaluate the cost of construction using high-quality and local base materials. Figure A.1 shows a cost evaluation program developed in Excel.

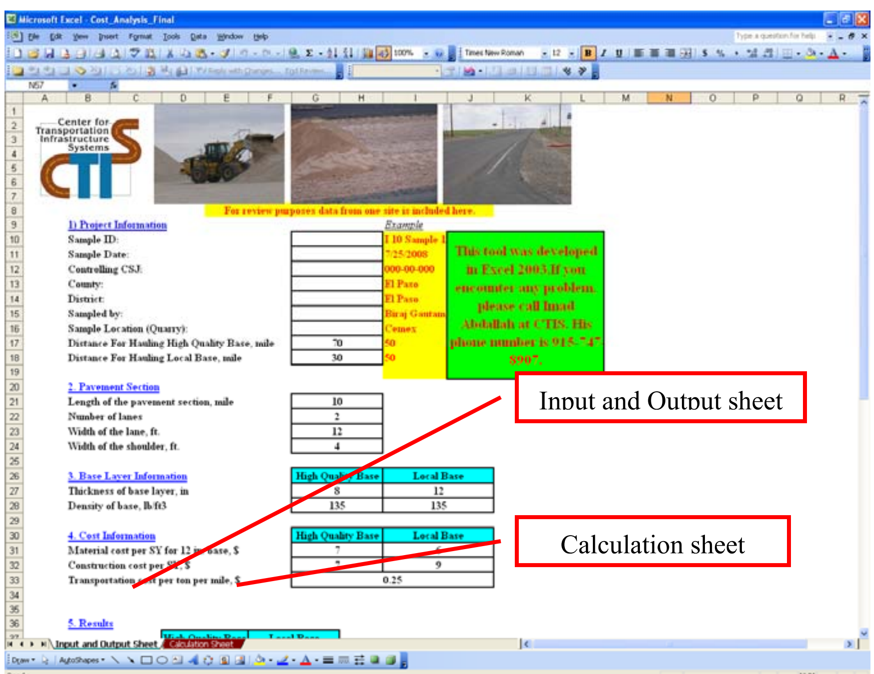


**Figure A.1- Outlook of Cost Evaluation Program**

## Section 1: Project Information

Section 1 (Project Information) is mainly for the documentation of the site. Figure A.2 shows an example of the Project Information Section. The project information, such as Sample ID, Sample Date, Controlling CSJ, County, District, Sampled by and Sample Location may be provided. Also, the average distances to haul in high-quality base and local base materials should be entered. It is generally assumed that the distance for high-quality base materials is more than that for local base materials.

| **1) Project Information** | |
|---|---|
| **Sample ID:** | |
| **Sample Date:** | |
| **Controlling CSJ:** | |
| **County:** | |
| **District:** | |
| **Sampled by:** | |
| **Sample Location (Quarry):** | |
| **Distance For Hauling High Quality Base, mile** | **70** |
| **Distance For Hauling Local Base, mile** | **30** |

**Figure A.2 - Project Information**

## Section 2: Pavement Section

In this section, the dimensions of the proposed pavement sections are input. Figure A.3 shows an example of Section 2 with a typical example. The length of pavement section, the number of lanes, the width of the lane, and the width of the shoulders should be entered.

| **2. Pavement Section** | |
|---|---|
| **Length of the pavement section, mile** | **10** |
| **Number of lanes** | **2** |
| **Width of the lane, ft.** | **12** |
| **Width of the shoulder, ft.** | **4** |

**Figure A.3- Pavement Section**

## Section 3: Base Layer Information

In this section, the information about base layer is provided. Figure A.4 shows an example of Section 3 with a typical example. The base thickness of the proposed section constructed with the high-quality base and local base materials along with their densities are entered.

| 3. Base Layer Information | High Quality Base | Local Base |
|---|---|---|
| Thickness of base layer, in | 8 | 12 |
| Density of base, lb/ft3 | 135 | 135 |

**Figure A.4 - Base Layer Information**

## Section 4: Cost Information

In this section, the information about several associated costs such as material costs, construction costs and transportation costs is input. This information is used to estimate the total cost of construction using high-quality base materials and local base materials. Figure A.4 shows an example of Section 4 with a typical example. The material cost per SY for 12 in. thick base layer for both types of material, high-quality and local base, are entered. It is generally assumed that the material cost for high-quality base is more than that for local base.

The second portion of this section requires the construction cost per SY. The construction cost includes the equipment cost, the labor cost and the cost of chemical additives. It is assumed that the local base materials are generally of low quality and the hauled-in base materials are generally of high quality. The low-quality/local base material generally requires treatment to comply with the Item 247 requirements, whereas the high-quality/hauled-in base material, for most of the cases, does not require chemical treatment. Considering this fact, it is assumed that the construction cost for low-quality/local base material is more than the construction cost for high-quality/hauled-in base material.

The last portion of this section requires transportation cost per ton per mile. Although, the rate of transportation cost for both types of base materials, high quality and local base, is the same, it is generally assumed that the total transportation cost for high-quality base is more than the total transportation cost for local base materials, as the hauling distance for high-quality base material is greater than the hauling distance for local base material.

| 4. Cost Information | High Quality Base | Local Base |
|---|---|---|
| Material cost per SY for 12 in. base, $ | 7 | 6 |
| Construction cost per SY, $ | 7 | 9 |
| Transportation cost per ton per mile, $ | 0.25 | |

**Figure A.5 - Cost Information**

## Section 5: Results

In this section, results in terms of total cost saving using high-quality base and local base materials are shown. This result is based on the inputs provided from Section 1 through Section 4. Figure A.6 shows an example of Section 5 with a typical example.

Also, to obtain allowable distance that can be traveled to haul in high-quality base material to realize potential cost savings, a graph between the cost saving using local base material in y axis and the difference in distance between the high-quality base material and the local base material in x axis is plotted. The distance where the cost saving is zero gives the additional distance that can be traveled to haul in high-quality base material.

**5. Results**

| | High Quality Base | Local Base |
|---|---|---|
| **Total Cost** | $ 3,534,119 | $ 3,685,267 |

**Local base is 4% more expensive. Therefore, High Quality Base is more economical**

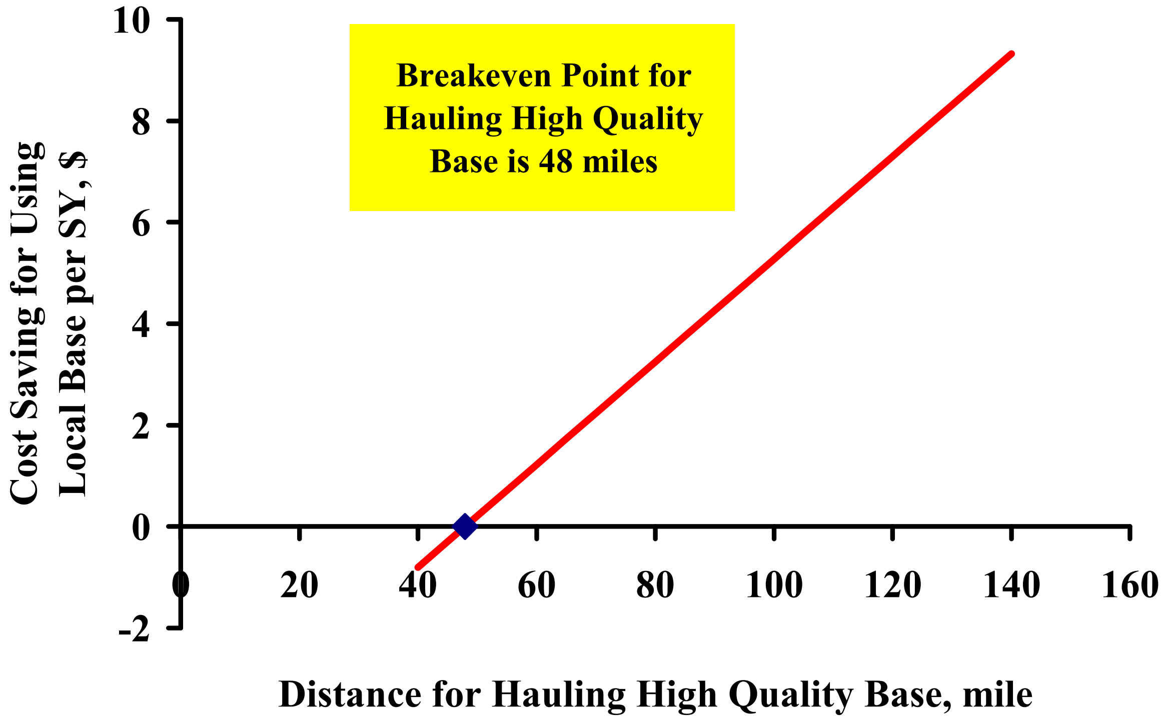


**Figure A.6 - Results of Cost Analysis**

## Section 6: Sample Calculation

## <u>Input:</u>

*<u>Section 1: Project Information.</u>*

Distance for Hauling High Quality Base, mile = 70
Distance for Hauling Local Base, mile = 30

*Section 2: Pavement Section.*

Length of the pavement section, mile = 10
Number of lanes = 2
Width of the lane, ft = 12
Width of the shoulder, ft = 4

*Section 3: Base Layer Information.*

| Base Layer Information | High Quality Base | Local Base |
|---|---|---|
| Thickness of base layer, in. | **8** | **12** |
| Density of base, pcf | **135** | **135** |

*Section 4: Cost Information.*

| Cost Information | High Quality Base | Local Base |
|---|---|---|
| Material cost per SY for **12** in. base, $ | **7** | **6** |
| Construction cost per SY, $ | 7 | 9 |
| Transportation cost per ton per mile, $ | **0.25** | |

## Calculation:

| Base Layer Information | High Quality Base | Local Base |
|---|---|---|
| Material cost per SY,$ | = (**8**/**12**) * **7** = 4.7 | = (**12**/**12**) * **6** = 6 |
| Transportation cost per mile per SY, $ | = **0.25**/(2000/(**135***27*((**8**/12)/3))) = 0.101 | = **0.25**/(2000/(**135***27*((**12**/12)/3))) = 0.152 |
| Construction cost per SY, $ | 7 | 9 |