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16. Abstract

Seismic methods provide moduli of different layers, which in many instances, may have distinct advantages over other methods used in the state of practice. Especially, seismic moduli are fundamentally-correct material properties, which can often be measured equally easily in the laboratory and in the field. In this report a comprehensive literature search that covers different aspects of pavement design with seismic moduli is presented. Based on this study, the use of seismic moduli in a mechanistic pavement design methodology is reasonable and feasible. To implement a fully-mechanistic design procedure, or to develop performance-based specifications, seismic moduli may be a better alternative. This statement is substantiated by the fact that the state of stress and strain are much better understood and defined under seismic tests. Adjusting seismic moduli for the state of the stress that the pavement is experiencing under the actual wheel load may be simpler than to understand fully the stress regimes developed during the FWD tests.

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Design Modulus Values Using Seismic Data Collection

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Feasibility Study for Determining Design Modulus Values Using Seismic Data Collection

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Acknowledgments

Executive Summary

In many current procedures for structural design of pavements an accurate determination of layer moduli is required. TxDOT has acquired the state-of-the-art equipment to perform laboratory and field modulus tests. With the onset of the movement toward the AASHTO 2002 Mechanistic Pavement Design, investigating the feasibility of supplanting the existing methods with more mechanistic approaches would be desirable. In any mechanistic pavement design procedure, accurate determination of moduli under load conditions similar to wheel loads is necessary.

Seismic methods provide moduli of different layers, which in many instances, may have distinct advantages over other methods used in the state of practice. Especially, seismic moduli are fundamentally-correct material properties, which can often be measured equally easily in the laboratory and in the field. Understanding this potential, TxDOT has invested in the development and practical use of seismic methods. As a result, the Seismic Pavement Analyzer (SPA) and the Portable Seismic Pavement Analyzer (PSPA) are available to TxDOT to measure seismic moduli practically and economically. Unfortunately, a formal design methodology that uses seismic moduli is not available. One reason for this matter is that the seismic moduli are measured at strain and stress levels that are smaller than those imposed by traffic.

In this report a comprehensive literature search that covers different aspects of pavement design with seismic moduli is presented. Based on the experience of the researchers and practitioners in nondestructive testing, pavement design, geotechnical engineering, seismology and earthquake engineering, a comprehensive conceptual design methodology has also been suggested. The comprehensive methodology has been analyzed and simplified so that it can be practical enough for use by TxDOT. The research issues and unanswered questions are comprehensively detailed. Several case studies are included to show the significance of the concept and the feasibility of it.

Based on this study, the use of seismic moduli in a mechanistic pavement design methodology is reasonable and feasible. To implement a fully-mechanistic design procedure, or to develop performance-based specifications, seismic moduli may be a better alternative. This statement is substantiated by the fact that the state of stress and strain are much better understood and defined under seismic tests. Adjusting seismic moduli for the state of the stress that the pavement is experiencing under the actual wheel load may be simpler than to understand fully the stress regimes

developed during the FWD tests. Many years of research in geotechnical earthquake engineering that deals with a similar problem has proven this.

Several issues have to be resolved before seismic moduli can be used for design. The most important issue to be addressed is to define a balance between the sophistication in the field tests, with the number and the nature of laboratory tests, with the design algorithm.

Specifically, the following items have to be considered:

1. The simplest computer algorithm that can provide the capability of determining nonlinear properties of different layers should be identified.

2. The most appropriate model to characterize the base and subgrade should be established.

3. The possibility and the negative consequences of cataloging or estimating some of the parameters used in the material models should be explored.

4. The validity of the entire system, consisting of the model used for estimating stresses and strains within a pavement section, and the nonlinear models used to determine moduli should be determined.

5. The proposed system should be fine-tuned until an acceptable compromise between the accuracy of the results, reasonableness of the laboratory and field tests, and ease of use of computer models are struck.

Implementation Statement

Since this was a feasibility study, the results may not be immediately implemented. However, with the initiation of "AASHTO 2002" program, which aims towards a mechanistic pavement design implementable by all highway agencies, this project may have significant impact. To develop a mechanistic pavement design which can contain performance-based specifications, the same engineering properties that are used to design a pavement should be used to determine the suitability of a material for construction and should be specified as criteria for accepting the material placed at the site. The only practical and available method at this time is based on seismic testing. Furthermore, it seems that with proper laboratory testing technique, and proper simulation one can develop models that are more realistic.

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1. Introduction

In many current procedures for structural design of pavements an accurate determination of layer moduli is required. TxDOT has acquired the state-of-the-art equipment to perform laboratory and field modulus tests. With the onset of the movement toward the AASHTO 2002 Mechanistic Pavement Design, investigating the feasibility of supplanting the existing methods with more mechanistic approaches would be desirable. In any mechanistic pavement design procedure accurate determination of moduli under load conditions similar to wheel loads is necessary.

Seismic methods provide moduli of different layers, which in many instances, may have distinct advantages over other methods used in the state of practice. Especially, seismic moduli are fundamentally-correct material properties, which can often be measured equally easily in the laboratory and in the field. Understanding this potential, TxDOT has invested in the development and practical use of seismic methods. As a result, the Seismic Pavement Analyzer (SPA) and the Portable Seismic Pavement Analyzer (PSPA) are available to TxDOT to measure seismic moduli practically and economically. Unfortunately, a formal design methodology that uses seismic moduli is not available. One reason for this matter is that the seismic moduli are measured at strain and stress levels that are smaller than those imposed by traffic.

In this report the feasibility of using seismic moduli is conceptually addressed. The proposed framework for designing pavements based on moduli from seismic methods is based on combining field and laboratory test results. The report contains a section on mechanistic design procedures being currently used. The definitions and fundamental differences between different laboratory and field moduli are described. The uses of seismic moduli in other disciplines are also discussed. A conceptual design methodology using seismic moduli is presented. The theoretical, experimental and conceptual shortcomings of the methodology, that can be addressed in future research projects, are also included. Finally, several case studies are included to demonstrate the process.

2. An Overview of Design Methodologies

Brown (1996), in his state-of-the-art review of the use of geomechanics in pavement engineering, suggests that an ideal mechanistic pavement design process contains the following four steps:

1. determining pavement-related physical constants, such as types of existing materials,
2. testing the candidate pavement with an NDT device to estimate its in situ moduli,
3. determining the laboratory properties of each layer, and
4. using an appropriate algorithm to estimate the remaining life of the pavement.

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Practically speaking, the algorithm used in pavement design dictates the nature of laboratory and field tests required. Therefore, defining the determination of moduli in terms of algorithms that are being used for mechanistic design is important.

Practitioners and researchers (Von Quintus et al., 1998) are in the consensus that the state of practice in pavement design at this time is based on empirical methods, and that the implementation of a mechanistic methodology (although challenging) is necessary. The practitioners and researchers (Von Quintus et al., 1998; and Smiley et al., 1998) are also in a consensus that two major modes of failure are fatigue cracking and permanent deformation. A major effort in developing and marketing a universal pavement design and rehabilitation is progressing under "AASHTO 2002 Mechanistic Pavement Design." Scholz et al. (1998) comprehensively enumerate the challenges faced by researchers heading that effort.

Ayres and Witczak (1998) summarize the models available for predicting fatigue cracking and permanent deformation. For the fatigue cracking mode, most models developed can be categorized under the general form of

$$N_F = K_1 \ (\varepsilon_t)^{K_2} \ (E_{AC})^{K_3} \tag{1}$$

where N_F is the number of repeated ESALs (remaining life) which cause the fatigue cracking damage to the pavement, ε_t is the tensile strain at the bottom of the asphaltic layer, and E_{AC} is the "dynamic" modulus of the asphalt layer. Parameters K_1 through K_3 are empirically-derived parameters. Different researchers have reported different values for these parameters (Huang, 1994; Ayres and Witczak, 1998). The modulus of the AC layer is typically measured in the laboratory or the field, but the tensile strain has to be determined from an algorithm.

Similarly, the number of ESALs that cause the rutting failure, N_R, in most models are reported to depend on the compressive strain at the top of the subgrade, ε_{vs}. A typical relationship is in the form of

$$N_R = K_1 \ (\varepsilon_{vs})^{K_2} \tag{2}$$

Again, many researchers have suggested ways of estimating parameters K_1 and K_2. The values of these parameters vary significantly among different recommendations (Huang, 1994; Ayres and Witczak, 1998). Although reconciling the recommendations from different studies is an important step, it is beyond the scope of this study and is not pursued any further. The main conclusion to be drawn from this section is that the most important parameters in any mechanistic pavement design are strains at different interfaces. Therefore, estimating these critical strains accurately is very important.

The model used to predict the critical strains should be robust and realistic. The model can be described by layer theory or one of many finite element programs. Any of these models can be linear or nonlinear, elastic or viscoelastic, and dynamic or static. Since the strains are directly related to

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the stiffness of different pavement layers, moduli of different layers should be well predicted. The material properties should be measured in a way that is compatible with the model used. If a balance between the material properties and analytical model is not struck, the results may be unreliable.

3. Determination of Significant Design Parameters

If the two models described above for cracking and rutting are reasonable, the parameters that affect them should be determined. Nazarian et al. (1997b) performed a stochastic sensitivity analysis to assess the influence of layer thickness, layer moduli and Poisson's ratio on the predicted remaining life of a pavement system using Equations 1 and 2.

The methodology used to carry out the study can be summarized in the following steps:

1. The pavement parameters (moduli, thicknesses and Poisson's ratios) were taken as random variables one at a time for the following four different pavement sections:

    - thin AC (75 mm), thin base (150 mm),
    - thin AC (75 mm), thick base (300 mm),
    - thick AC (125 mm), thin base (150 mm),
    - thick AC (125 mm), thick base (300 mm).

Each of the pavement parameters was varied while the remaining parameters were assumed constant and assigned typical values as shown in Table 1. This process was repeated for each of the pavement parameters depicted in Table 1.

2. Assigning a coefficient variation of 10 percent, 100 sample values for each pavement parameter were generated using Monte Carlo simulation techniques (Ang and Tang 1984a, 1984b).

3. For each simulated sample set, the remaining lives due to fatigue cracking and rutting were calculated, and compared with the remaining lives from the original design. The variation from the baseline design was calculated from:

$$Variation\ (percent) = \frac{RL_{perturbed} - RL_{baseline}}{RL_{baseline}}\ 100\%$$

(3)

where $RL_{perturbed}$ and $RL_{baseline}$, obtained from Equations 1 and 2, correspond to the remaining lives from the base line pavement profile and the perturbed pavement profile, respectively.

4. Based on the variation of the remaining lives from the input moduli the factors that significantly affect the remaining life were identified. A set of arbitrary limits was used to define the significance of a given parameter in the remaining life. These levels are defined in Table 2.

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Table 1 - Properties of Typical Pavement Sections Selected for This Study

Parameter	Layer	Value
Thickness	AC (t_1)	75 mm or 125 mm
	Base (t_2)	150 mm or 300 mm
Modulus	AC (E_1)	3.5 GPa
	Base (E_2)	350 MPa
	Subgrade (E_3)	70 MPa
Poisson's Ratio	AC (v_1)	0.35
	Base (v_2)	0.40
	Subgrade (v_3)	0.45

Table 2 - Level of Significance Assigned to each Parameter Based on a 10% Perturbation of Original Input Parameter

Level of Significance	Criteria concerning Coefficient of Variation	Symbol	Significance to Pavement Design
Insignificant	< 5 percent	I	Can be probably estimated with small error in final remaining life
Moderately Significant	5-15 percent	M	Must be measured to limit errors in design
Significant	15-25 percent	S	Must be measured reasonably accurately for satisfactory design
Very Significant	> 25 percent	V	Must be measured very accurately or design may not be considered appropriate

Typical comparisons of the remaining lives with respect to the baseline design due to fatigue and rutting for a pavement with thin AC and thin base layers are shown in Figure 1. The baseline remaining lives refer to the remaining lives calculated from the parameters shown in Table 1 before any perturbation of the parameters. To decide if a parameter is sensitive to a certain model, the standard deviation of the remaining life associated with that parameter is compared with the 10 percent perturbation allowed. Depending on whether the standard deviation is larger or smaller than 10 percent input, one can judge if the parameter is sensitive or not sensitive to a certain design.

From Figure 1a, a 10 percent variation in Poisson's ratios of the AC (v_1) and base (v_2) results in a very small variation in the remaining life due to rutting. Therefore, these two parameters are

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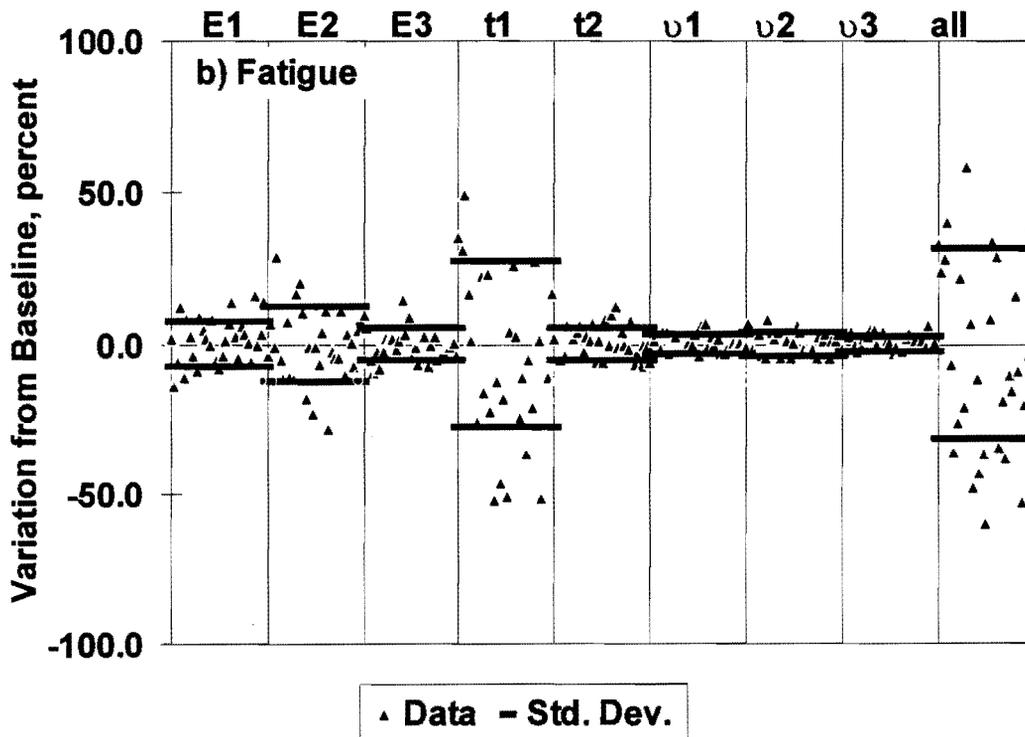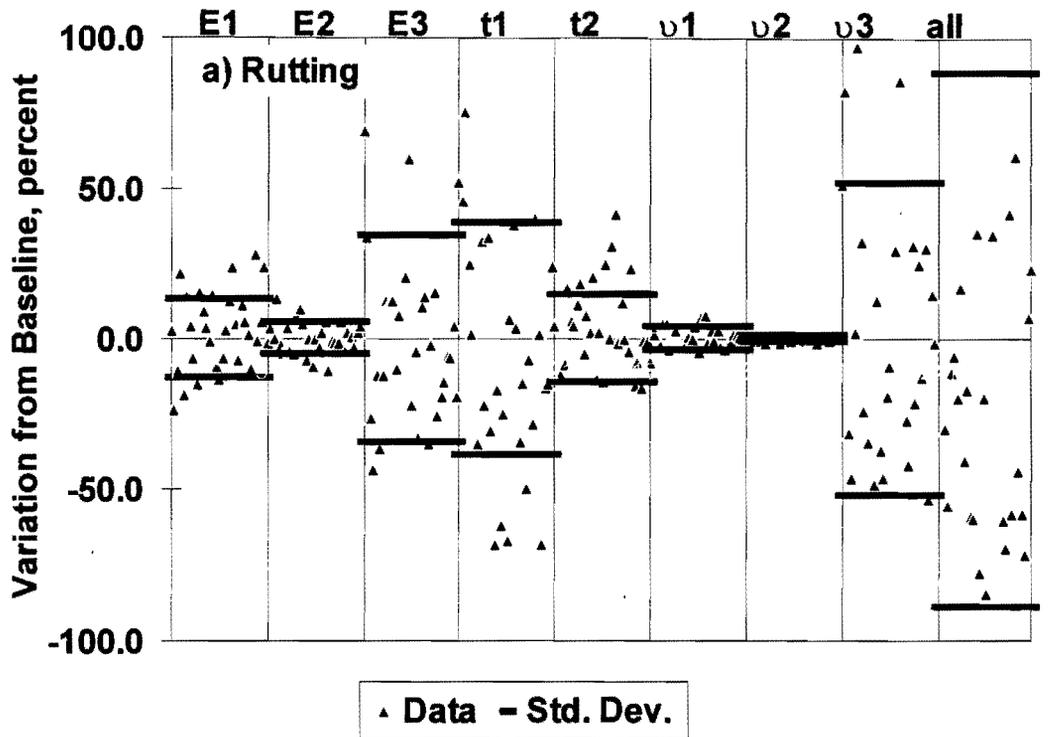
Figure 1 - Sensitivity of Different Pavement Parameters on Remaining Life of a Typical Pavement with "Thin AC and Thick Base"

categorized as insignificant. On the other hand, varying the modulus of subgrade (E_3) or thickness of the AC (t_1) or Poisson's ratio of subgrade (v_3) by 10 percent would change the remaining life due to rutting by more than 25 percent. Therefore, these three parameters are considered as very significant. The results are summarized in Table 3 for different pavement structures.

Based on this study, the parameters itemized in Table 4 significantly affect the remaining life of a flexible pavement. Therefore, not only the modulus of each layer should be accurately measured, the thickness and Poisson's ratio of some layers should also be known. In this project, we will concentrate on modulus values.

4. Determination of Modulus

The stiffness profile can either be determined with field testing or laboratory testing. For a more sophisticated analysis, the behavior of a material in terms of variation in stiffness with stress level, strain amplitude, and the strain rate should be determined. This behavior is typically established by conducting laboratory tests such as the resilient modulus test.

Laboratory tests are essential to study the parameters that affect the properties of materials. However, moduli from laboratory tests are normally less than the in situ results by anywhere from ten to several hundred percent. This discrepancy can be due to sampling disturbance, differences in the state-of-stress between the specimen and in-place pavement material, nonrepresentative specimens, long-term time effects, and inherent errors in the field and laboratory test procedures (Anderson and Woods, 1975).

Field tests are more practical and more desirable because they are rapid to perform, and because they test a large volume of material in its natural state-of-stress. Field tests typically fall into two categories — *material characterization* and *design simulation*. In *material characterization* one attempts, in a way that is the most theoretically-correct, to determine the engineering properties of a material (such as modulus or strength). The material properties measured in this way, are fundamental material properties that are not related to a specific modeling scenario. To use these material properties in a certain design methodology, they should be combined with an appropriate analytical or numerical model to obtain the design output.

In the *design simulation*, one tries to her/his best ability to simulate the design condition experimentally, and then back-figure some material parameter that is relevant to that condition. The seismic methods can be considered as methods that provide material characterization; whereas the deflection-based methods are geared more toward the design simulation. Both approaches have advantages and disadvantages that are summarized in Table 5.

4.1. Laboratory Measurements of Moduli

Currently, the common way to develop the stress-strain relationship is through laboratory tests. The laboratory method of choice in pavement engineering is the resilient modulus (M_R) test. However, other methods, such as cyclic triaxial tests or torsional shear resonant column tests, can be readily applied to the base material.

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Table 3 - Significance of Pavement Parameters in Remaining Life of Pavement

Parameter	Layer	Relative Significance							
		Rutting				Fatigue			
		Thin AC		Thick AC		Thin AC		Thick AC	
		Thin Base	Thick Base	Thin Base	Thick Base	Thin Base	Thick Base	Thin Base	Thick Base
Thickness	AC (t_1)	V	V	V	V	M	I	M	V
	Base (t_2)	V	S	V	V	M	S	V	I
Modulus	AC (E_1)	M	M	M	M	I	I	M	M
	Base (E_2)	M	M	I	M	M	S	S	S
	Subgrade (E_3)	S	V	M	V	M	M	M	M
Poisson's Ratio	AC (v_1)	I	I	I	I	I	I	I	I
	Base (v_2)	I	M	I	M	M	M	I	M
	Subgrade (v_3)	V	V	V	V	I	I	I	I

Table 4 - Parameters that Affect Remaining Life of Flexible Pavements

Failure Criteria	Important Pavement Parameters[*]
Fatigue Cracking	• Modulus of Base • Thickness of Base • Thickness of AC • Modulus of Subgrade • Poisson's Ratio of Base • Modulus of AC
Rutting	• Thickness of AC • Poisson's Ratio of Subgrade • Thickness of Base • Modulus of Subgrade • Modulus of AC • Modulus of Base

[*] Parameters are sorted in their order of significance

Table 5 - Advantages and Disadvantages of Methods Used to Obtain Moduli

Test Method	Major Advantage(s)	Major Weakness(es)
Laboratory Methods	Valuable for developing constitutive model for a material (i.e., variation in modulus with the state of stress and strain)	Very difficult to prepare specimens with the same characteristics of in situ materials Time consuming and expensive to perform
Deflection-Based Field Methods	Covers a representative volume of material Imposes loads that approximate wheel loads	Accurate determination of moduli of pavement layers may be difficult due to problems with backcalculation The state-of-stress within pavement strongly depends on moduli of different layers, and hence is unknown.
Field Seismic Methods	Covers a representative volume of material Measures a fundamentally- correct parameter (i.e., linear elastic modulus)	State-of-stress during seismic tests differ from the state-of-stress under actual loads Non-uniqueness in the results due to inversion.

A typical setup for resilient modulus test is shown in Figure 2. The testing procedure for the resilient modulus consists of subjecting a specimen to a sequence of confining pressure and cyclic deviatoric stress levels. The load pulse consists of a haversine load with a duration of 0.1 sec loading followed by a rest period of 0.9 sec. The loads applied to the specimen are monitored by a load cell, and the resilient axial deformations are measured with LVDT's or noncontact sensors. The resilient modulus at a given deviatoric stress and confining pressure is the ratio of the applied stress to measured resilient strain. Several protocols have been proposed by SHRP, AASHTO and others (see Nazarian et al.,1996 for details). The diameter of the specimen tested is typically between 75 mm (for subgrade materials) to 150 mm (for base materials).

The cyclic triaxial test is quite similar to the resilient modulus test. The major difference is that the applied loads are cyclic in nature (as opposed to a half-sine wave with rest period). Although the loading pattern is more robust, this test essentially suffers from the same limitations as the resilient modulus test. The two main limitations as reported by Woods (1978) are that the strain measurements below 0.01% are difficult, and that void ratio redistribution occurs within the specimen during cyclic testing.

The resonant column test (see Figure 3) for determining the moduli of soil specimens has been in use for the last 30 years. The test procedure has been standardized under ASTM D4015-93. Harmonic excitation is applied to the top of the specimen over a range of frequencies, so that the

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resonant frequency of the specimen can be measured. Using the equation of motion within a fixed-free column, the modulus and damping ratio of the material can be determined.

The resonant column test device can be slightly modified so that torsional shear tests can be done on the same specimen. A cyclic torsional force at a given frequency is applied to the top of the specimen. Instead of determining the resonant frequency, the stress-strain hysteresis loop is
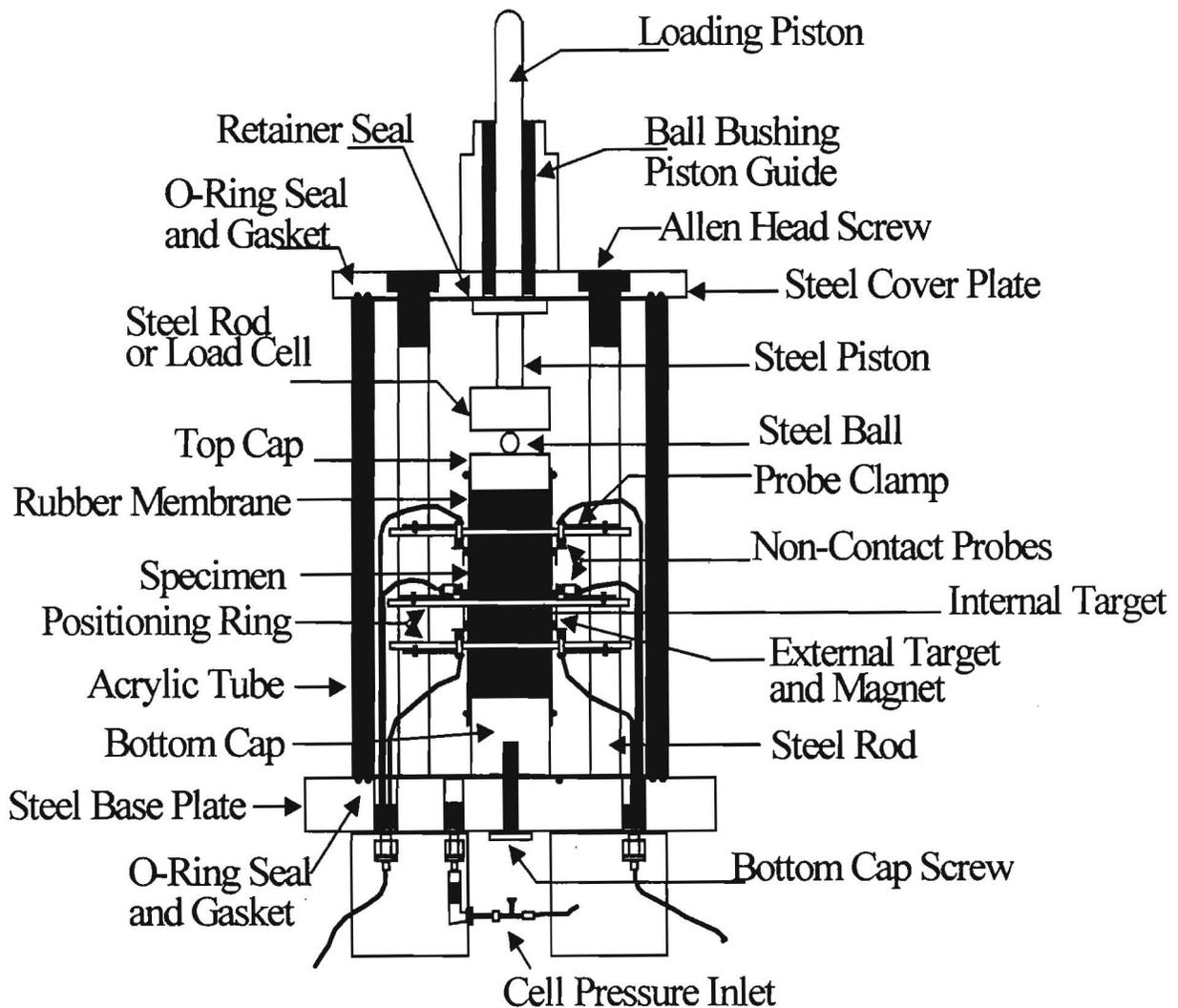
Figure 2 - A Schematic of Resilient Modulus Test Set Up
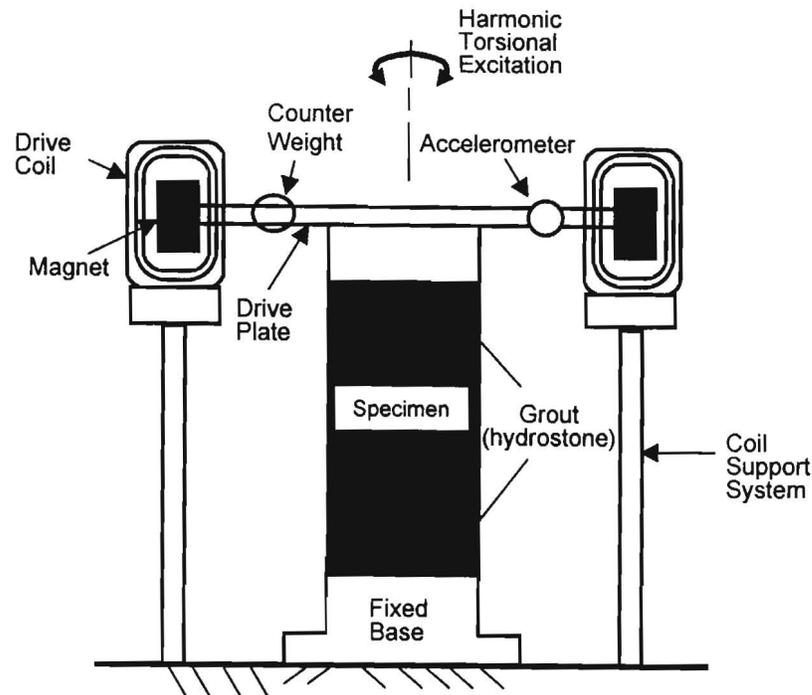
Figure 3 - A Schematic of Resonant Column

determined. The stiffness and material damping are determined from that loop. The advantage of the torsional shear tests (over resonant column tests) is that the material properties at different frequencies can be determined.

These four methods are the major comprehensive test methods that can be used to characterize dynamic properties of materials. These test procedures are considered time consuming to perform. However, simplified test methods also exist that can be used to determine the modulus of different materials rapidly. Simplified laboratory tests can be used with the more sophisticated ones during the design process. By combining the results from simplified and more comprehensive tests, one can either ensure compatibility or develop correlations that can be readily used in the field. Several of these tests are described next.

Bender elements are thin sheets of piezo-ceramic material inserted in a specimen. When subjected to appropriate electric current, a bender element bends and couples seismic energy to the specimen. A bender element can also be used to detect the coupled energy since it converts movement to voltage. A detail description of the device can be found in Baig (1992). In the last 20 years, the technology has been developed and used in the geotechnical earthquake engineering area for similar purposes. For convenience, the end caps of a regular static triaxial test setup can be retrofitted with a set of transmitting and receiving bender elements as shown in Figure 4. The data reduction, which comprises of determining the arrival time of the seismic energy, is simple and can be carried out during the test.

The limitation of the device is that the deviatoric stress cannot be varied and as such moduli at small strain levels are measured. Therefore, this method can be used parallel to resilient modulus tests when several specimens of a similar material have to be tested. Based upon tests on more than thirty

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different specimens, the bender elements can feasibly provide resilient moduli of subgrade materials at a fraction of time and at much lower investment in initial equipment costs.

A schematic of a free-free resonant column test setup is shown in Figure 5. The specimen is suspended from two wires. An accelerometer is securely placed on one end of the specimen, and the other end is impacted with a hammer instrumented with a load cell. The signals from the accelerometer and load cell are used to determine the resonant frequency, f. Once the frequency, mass density, ρ, and the length of the specimen, L, are known, Young's modulus can be found from

$$E = \rho (2 f L)^2. \tag{4}$$

Alternatively, the accelerometer can be placed in the radial direction, and the specimen can be impacted in the radial direction to determine the shear modulus. Again, the shear and Young's moduli can be combined to calculate Poisson's ratio.

For soft specimens that cannot be suspended by wires because of their structural integrity, a half-cylindrical piece of PVC pipe can be used to support specimens during testing. These tests have been performed with success on subgrade, granular base, stabilized base, and AC or PCC cores. The main limitation is that the length-to-diameter ratio should not be less than two. Overall the method is quite repeatable, and is nondestructive. In less than three minutes, the sensors can be placed, tests can be done and interpreted. The initial equipment cost is about $4,000.
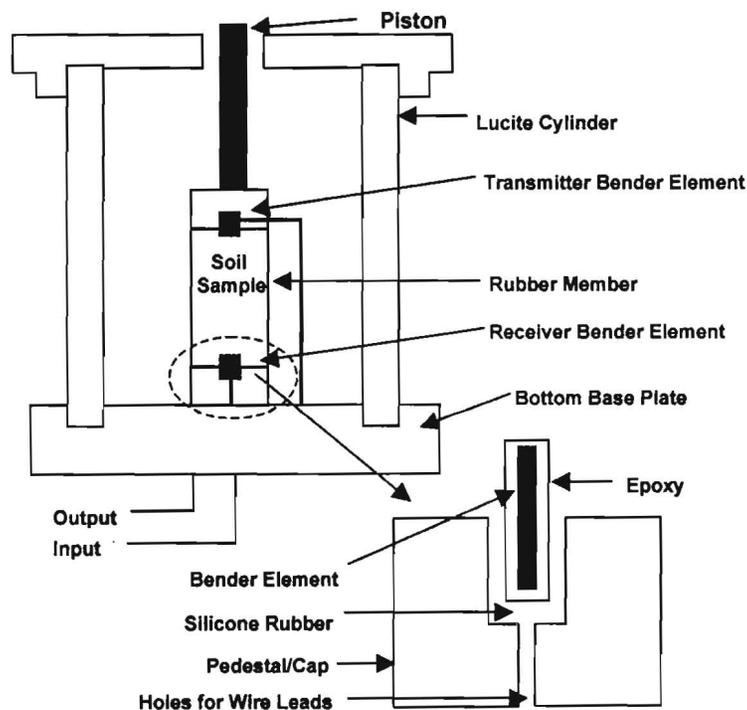
Figure 4 - Piezo-Ceramic Bender Elements
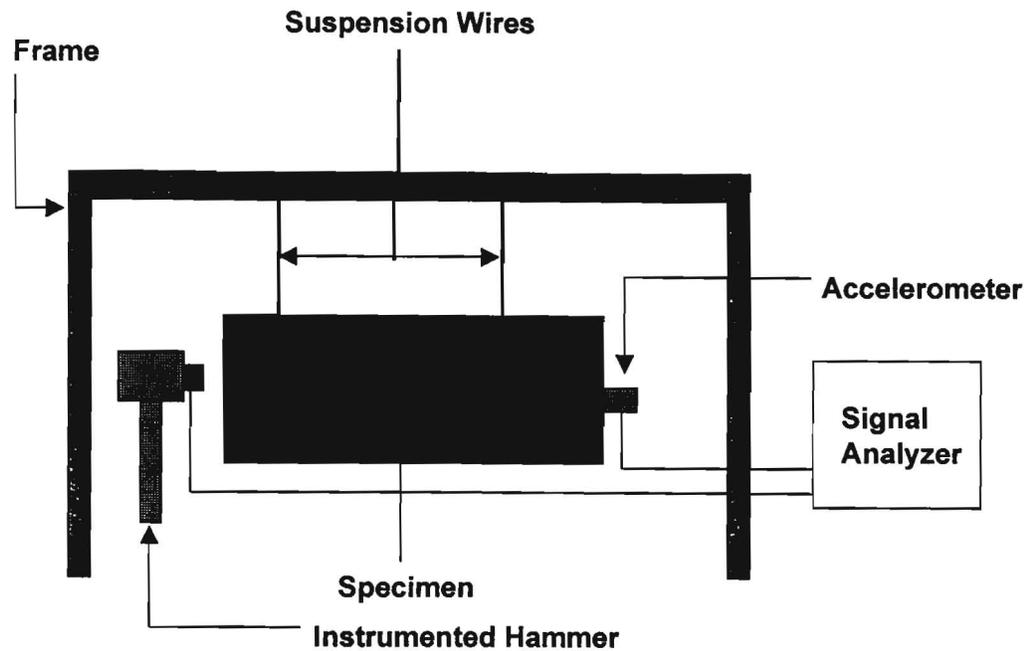
Figure 5 - Free-Free Resonant Column

The schematic of an ultrasonic testing setup is shown in Figure 6. A transmitting transducer is securely placed on the top face of the specimen. This transducer is connected to the built-in high-voltage electrical pulse generator of the device. The electric pulse transformed to mechanical vibration was coupled to the specimen. A receiving transducer is securely placed on the bottom face of the specimen, opposite the transmitting transducer. The receiving transducer, which sensed the propagating waves, was connected to an internal clock of the device. The clock can measure the travel-time of the compression waves automatically.

The equipment can be purchased from a vendor, and the supporting equipment needed to perform the test in day-to-day projects can be easily fabricated. The device is particularly useful for testing AC brikettes and stabilized layers. A typical measurement would take less than one minute, and the device costs about $ 5,000.

4.2. Field Measurements of Moduli

Several field-testing methodologies are available for determining the modulus profile of a pavement section. The main methods used are either deflection-based or seismic-based. Deflection-based devices, particularly the Falling Weight Deflectometer (FWD), are the most common field evaluation devices in Texas. The principles of the operation and the data reduction methodology for backcalculating moduli are well known and are not repeated here. Although the device is an excellent tool for the day to day pavement evaluation, some researchers and practitioners have shown

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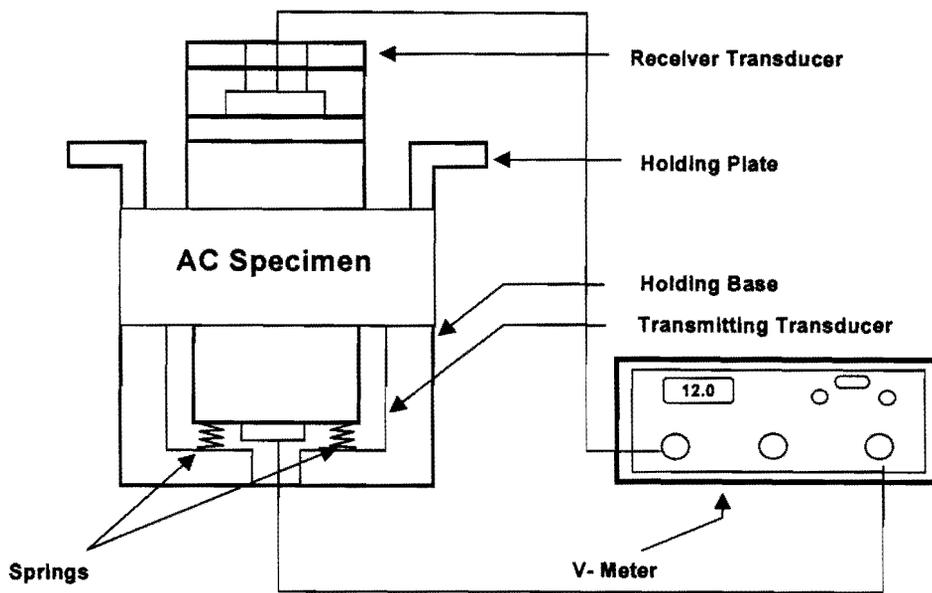
Figure 6 - Ultrasonic Laboratory Device

concern with respect to the moduli obtained with the device in certain conditions. The existence of shallow bedrock (or for projects that involve extensive cut and fill), the backcalculation methodology may not yield repeatable results.

Five different seismic tests can be categorized under the broad category of seismic and dynamic tests. These tests are:

1.   Spectral Analysis of Surface Waves (SASW),
2.   Impulse Response (IR),
3.   Ultrasonic Body Wave (UBW),
4.   Ultrasonic Surface Wave (USW), and
5.   Impact Echo (IE).

TxDOT has been a pioneer in developing the seismic methods as a tool for pavement evaluation. The Spectral-Analysis-of-Surface-Waves (SASW) method has been mainly developed with the financial assistance of TxDOT. The method can yield more accurate and comprehensive data with respect to the stiffness properties of pavement layers. However, moduli are obtained at small-strains.

Another concern has been the long testing time associated with the method. This shortcoming has been resolved by the development of the Seismic Pavement Analyzer (SPA). The details of the device are fully covered in Nazarian et al. (1995). Also under Project 1966, we have developed a hand-portable version of this device called a PSPA (Baker et al., 1995).

The SASW method is a seismic method that can nondestructively determine modulus profiles of pavement sections. A computer algorithm uses the time records to determine a representative dispersion curve (variation of wave velocity with wavelength) automatically (Nazarian and Desai, 1993). The last step is to determine the elastic modulus of different layers, given the dispersion

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curve. An automated inversion process (Yuan and Nazarian, 1993) determines the stiffness profile of the pavement section. The method provides the modulus and thickness of different layers. With some modifications to the SPA and PSPA, the method can be applied after the completion of construction of each pavement layer. The fewer the number of layers, the more easily the moduli and thicknesses can be determined.

The main parameter obtained on flexible pavements with the impulse-response (IR) method is overall stiffness of the pavement, which can be used to delineate between good and poor support. This test is equivalent to performing FWD tests with only one sensor. The impulse response tests can be performed automatically with the SPA, with the newly developed "Humboldt Stiffness Gauge," or simply with an instrumented hammer and geophone connected to a signal analyzer.

The ultrasonic-body-wave (UBW) method can directly measure Young's modulus of the top layer. In that method, the arrival of different types of waves can be determined from the signals. The limitations of this method are summarized in Nazarian et al. (1997a). Overall this method should be used with care.

The ultrasonic-surface-wave (USW) method is an offshoot of the SASW method. The major distinction between these two methods is that in the ultrasonic-surface-wave method the shear modulus of the top layer can be easily and directly determined without a complex inversion algorithm. The results from the UBW and USW methods can be combined to determine Poisson's ratio of the top layer readily.

The impact-echo (IE) method can be used to determine the thickness of pavement layers if the layer is thicker than 10 cm.

Other methods, mostly borrowed from geotechnical earthquake engineering, are also available that can provide the variation in modulus with depth. One such method is the crosshole seismic test. The method has been used in pavements in several research projects (Lee et al., 1998).

A schematic of the testing procedure normally carried out is shown in Figure 7. One or two boreholes are drilled to the maximum depth at which testing is to be conducted to accommodate receivers. Another borehole is then drilled to function as the source borehole. The source generates compression and shears waves that are detected by receivers in the other holes as they pass by. Although the method is accurate, the testing time and the site preparation procedure make the method impractical for day-to-day use.

The downhole seismic tests, as shown in Figure 8, are similar to the crosshole tests just described. The main differences are that the site preparation is much simpler because only one receiver hole is needed, and the source is placed on the ground or pavement surface. The source hole is eliminated. To perform a test at a site, seismic sensors are placed at a desired depth. A source is placed on the pavement surface. The data are collected and reduced by determining the arrival of different waves. The sensors are then placed at a different depth, and tests are repeated.

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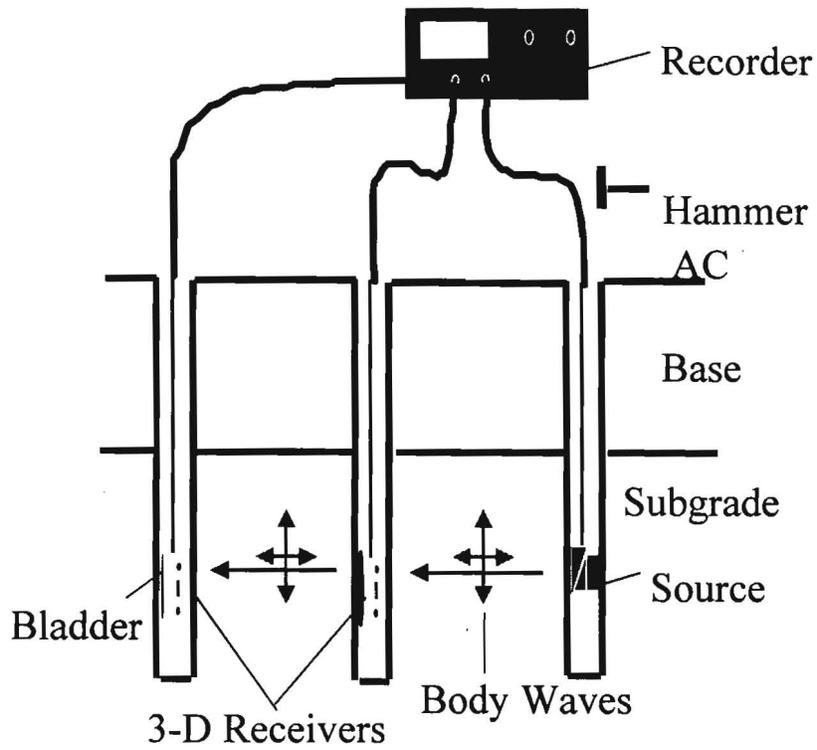
Figure 7 - Schematic of Crosshole Seismic Tests
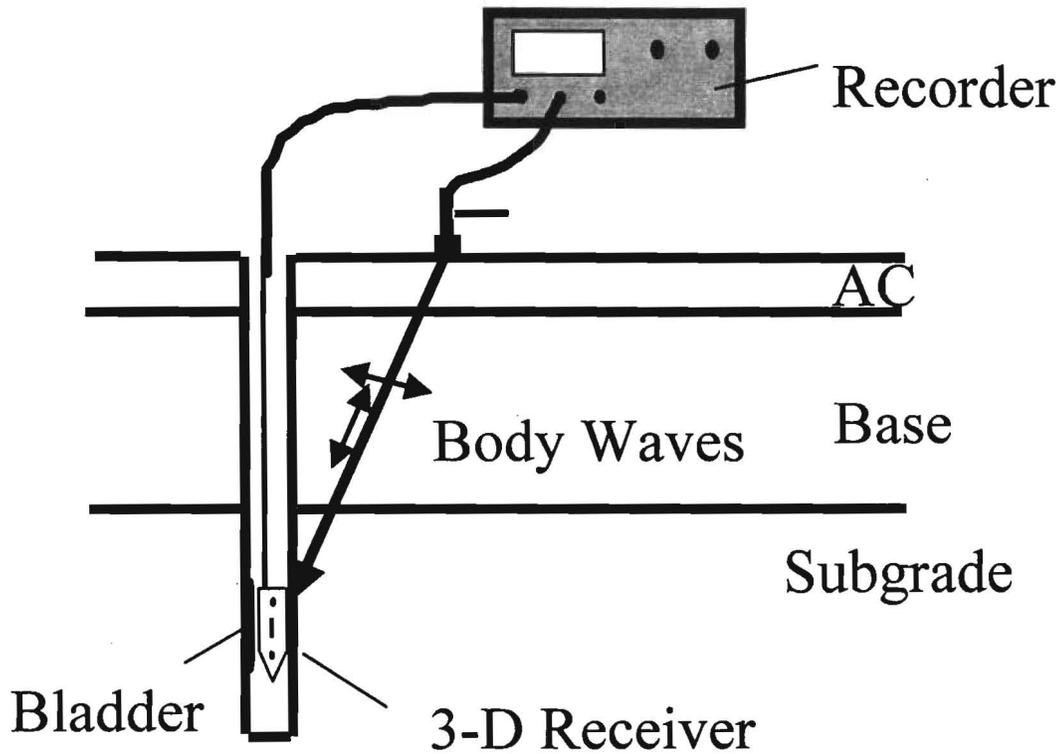
Figure 8 - Schematic of Downhole Seismic Tests

Because of significant differences in the stiffness of different pavement layers, many experts prefer this method to crosshole tests because the possibility of generating refracted waves is very small for the downhole tests (National Science Foundation, 1994). Refracted waves complicate the interpretation of the results.

Nazarian et al. (1998) have recently shown the feasibility of retrofitting a device similar to a Dynamic Cone Penetrometer (DCP), so that the downhole seismic tests can be performed with the DCP tests. Shinn et al. (1988) have developed a similar device but for deep geotechnical strata.

Other methods are either under development or have not been widely used in pavement engineering. These methods and devices are not described here for the sake of brevity. The readers are referred to Ebelhar et al. (1994), or Von Quintus et al. (1994) for papers related to these topics.

5. Modeling Behavior of Pavement Materials

The behavior of most soils and pavement materials under a load can be represented by a stress-strain curve similar to the one shown in Figure 9. Three significant parameters related to this curve are:

1. the initial tangent modulus, or maximum modulus (E_{max}) — the slope of the tangent to the curve passing through the origin,

2. the strength of the material (s_{max}) — the horizontal line asymptotic to the curve, and

3. the secant modulus $(E_1, E_2$ or $E_3)$ — the slope of a line connecting the origin to any point of the curve.

The initial tangent modulus is directly affected by the stress state, and the density of the material. The secant modulus is strongly affected by strain experienced by the material. Since pavements, specially thin ones, may experience higher strains than those applicable to initial tangent modulus of a material, means of determining the secant modulus should be developed. To estimate the secant modulus reasonably, the stress-strain curve for each layer must be fully defined. From Figure 9, if a relationship between the initial tangent modulus and secant modulus can be developed, one can easily define the stress-strain curve. Several parameters are used to define different types of moduli from different test methods. Table 6 contains these terminologies.

The basic materials used in pavements are either granular bases and subgrades, or bituminous materials, or materials made of Portland cement. Stabilized and treated materials generally fall in one the above three categories depending on the degree of stabilization. Each of these materials is discussed below.

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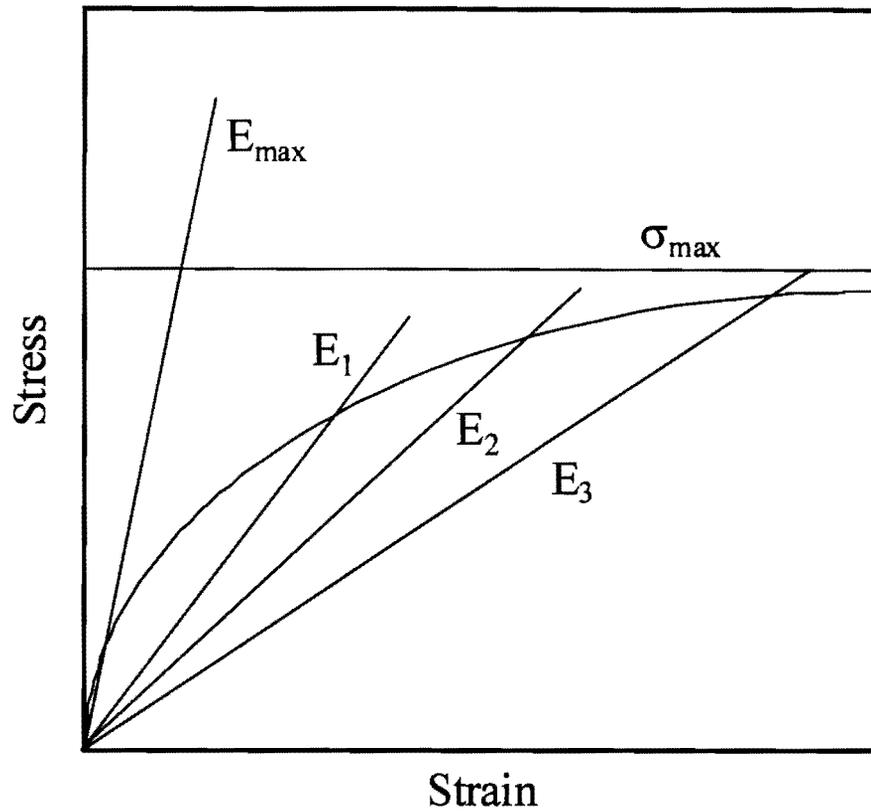
Figure 9 - Typical Stress-Strain Curve for a Pavement Material

Table 6 - Definition of Different Terms Used to Define Stiffness of Materials

Term	Definition
Resilient Modulus	The modulus of a pavement material determined in the laboratory from a variety of resilient modulus test protocols. This modulus normally corresponds to a secant modulus shown in Figure 1. Due to limitations with existing equipment in most cases determining the initial tangent modulus with the resilient modulus test is difficult.
FWD Modulus	The modulus of a layer determined from the backcalculation of deflection basins measured in the field. This modulus normally corresponds to a secant modulus for materials close to the loading pad (i.e., AC layer, base and shallow subgrade) and an initial tangent modulus for materials far from the impact point (i.e., deeper subgrade materials).
Seismic Modulus	The modulus of a layer either directly measured or backcalculated using a small seismic source. This modulus always corresponds to the initial tangent modulus since the impact is small.

5.1. Base and Subgrade Materials

The base and subgrade materials, depending on their gradation and plasticity can be divided into two groups, fine grained (cohesive) or coarse grained (cohesionless or granular). The constitutive properties of both materials are defined based on the state of stress applied to them. Barksdale et al. (1994) list half-a-dozen models that can be used for this purpose. They, however, recommended a model that is in the form of

$$M_R = k_1 \, \sigma_c^{k_2} \, \sigma_d^{k_3} \tag{6}$$

where σ_d and σ_c are the deviatoric stress and confining pressure, respectively. Parameters k_1 through k_3 are coefficients statistically determined from the results of the laboratory test. The advantage of the model preseneted in Equation 6 is that it is universally applicable to fine-grained and coarse-grained base and subgrade materials. The accuracy and reasonableness of these models are extremely important because they are the keys to combine laboratory and field results successfully.

The axial strain, ε_{ax}, is directly proportional to the deviatoric stress, through

$$\varepsilon_{ax} = \sigma_d \, / \, M_R \tag{7}$$

By substituting Equation 7 in Equation 6, one obtains the following relationship:

$$M_R = (K_1 \, \sigma_c^{K_2}) \, \varepsilon_{ax}^{K_3}. \tag{8}$$

The parameters are represented in uppercase in Equation 8 to emphasis that they are related but are different from parameters k_1 through k_3 in Equation 6.

In Equations 6 or 8, the term $k_1 \, \sigma_c^{k_2}$ or $K_1 \, \sigma_c^{K_2}$ corresponds to the initial tangent modulus, E_{max}, which is related to the confining pressure. The other term in both equations suggests that the modulus changes as the axial strain and deviatoric stress changes. Since k_3 (or K_3) are usually negative, the modulus decreases with an increase in deviatoric stress (or strain). This discussion will permit us to define parameters that affect initial tangent modulus (E_{max}) plus parameters that affect the change of modulus with strain.

Hardin and Drnevich (1972), based on many laboratory tests accumulated a list of parameters that affect modulus of both fine-grained and coarse-grained soils. These parameters, along with their significance are summarized in Table 7. The state of stress, void ratio (i.e., density) and the strain amplitude are the main parameters that affect moduli of a material. For fine-grained soils, the degree of saturation is also very important.

The excellent agreement between the parameters identified by Hardin and Drnevich and Equation 8 is important. To exhibit the resemblance, parameter K_1 is related to the void ratio and degree of saturation (when applicable) of the material, the state of stress and strain enter into the equation as confining pressure, and axial strain.

Table 7 - Parameters Affecting Modulus of Granular Bases and Subgrades

Parameter	Importance*		Parameter Affected in Equation 6		
	Coarse-Grained Materials	Fine-Grained Materials	K_1	K_2	K_3
Strain Amplitude	V	V			✔
Effective Mean Principal Stress (Confining Pressure)	V	V		✔	✔
Void Ratio	V	V	✔		
Degree of Saturation	R	V	✔	✔	
Overconsolidation Ratio	R	V	✔		
Effective Stress Envelop	R	L	✔		
Octahedral Shear Stress	L	L	✔		
Frequency of Loading	L	L	✔		
Long-Term Time Effects (Thixotropy)	R	R	✔		
Grain Characteristics	R	L	✔		✔
Soil Structures	R	R	✔		✔
Volume Change Due to Shear Strain	V	R	✔		

* V means important, L means less important, R means relatively unimportant.

The impacts of these parameters on the three K parameters in Equation 8 are also added to Table 7. Most parameters suggested by Hardin and Drnevich affect K_1. Unfortunately, most of these parameters cannot be reproduced in the laboratory specimen, and that is the reason for not being able to determine moduli in the field that are similar to those obtained from the laboratory testing. However, K_2 and K_3 are affected by few parameters, and can be readily measured in the laboratory. Therefore, determining K_2 and K_3 in the laboratory is relatively easy; whereas measuring K1 in the laboratory is practically difficult.

An alternative way of presenting the stress-strain curve shown in Figure 9 is through Equations 6 or 8. These graphs are included in Figure 10. From Figure 10a, the initial tangent modulus, E_{max}, corresponds to the modulus at very small deviatoric stress, and is related to the confining pressure ($E_{max} = k_1 \sigma_c^{k_2}$). The secant modulus at any other deviatoric stress (M_R here) can be found from the slope of the line. Figure 10b, which corresponds to Equation 8, provides the same information as Figure 10a, but in a more convenient way. In that figure, the secant modulus is directly related to strain. Again, the initial tangent modulus is related to the confining pressure, and two of the parameters from Equation 8. The variation in secant modulus with strain on a log-log scale is a straight line.
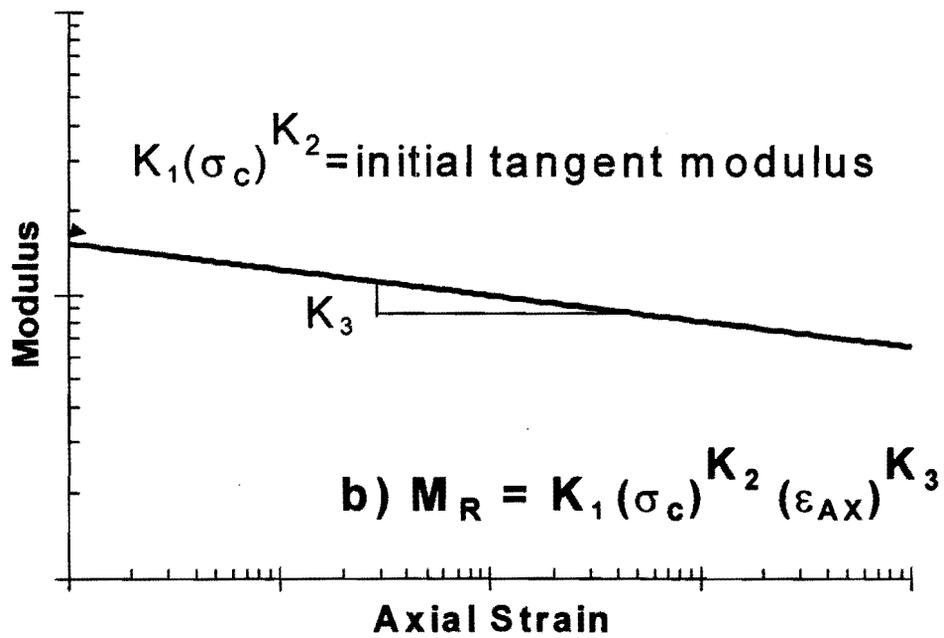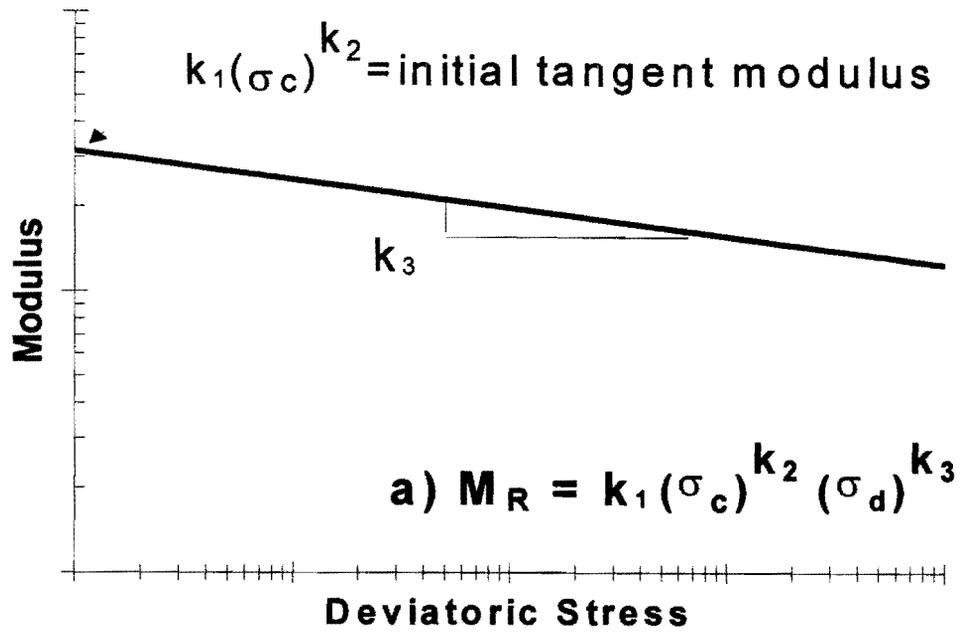
Figure 10 - Representation of Stress-Strain Curve According to Equations 6 and 8.

Table 8 - Typical Material Parameters Observed for Bases in Texas

Parameter	k_1 (KPa)	k_2	k_3	Confining Pressure(KPa)
Lower Bound	25,000	0.1	0	1
Upper Bound	100,000	0.5	-0.5	100
Default	50,000	0.3	-0.33	10

Plotting the stress-strain curve for a typical material in Texas would be beneficial. Based on our experience, typical values of K_1, K_2 and K_3 (see Equation 6) for the base material in Texas are summarized in Table 8. We have also included the default values assumed for the future representations. The confining pressure varies from 1 KPa to 100 KPa to cover the range of confining pressures typically encountered in a pavement before and during loading.

Typical stress-strain curve for a base material in Texas is shown in Figure 11[1] for three different confining pressures. As the confining pressure (denoted CP on the figure) increases, the stress-strain curve moves upward, i.e., the modulus increases. However, as the axial strain increases, the secant modulus decreases.

To quantify these statements, the variation in modulus with axial strain using Equation 8 is shown in Figure 12. The values used to generate Figure 12 are listed in Table 8. The minimum strain level shown is 10^{-4} percent that is considered as the threshold of the initial tangent modulus, E_{max}. The maximum of 1 percent strain is way above strain levels experienced by any functional pavement. Again, as the confining pressure increases or the strain level decreases the modulus increases.

A convenient method to demonstrate the modulus versus strain curves shown in Figure 12 is to "normalize" them. To normalize the results, the modulus at a given strain and confining pressure is divided by the initial tangent modulus (here at a strain level of 10^{-4} percent) measured at the same confining pressure. Figure 13 reflects such a normalized curve where the results from the three confining pressures shown in Figure 12 collapse into one unique curve. The significance of this curve is that if one measures the initial tangent modulus of a given material, one can readily determine the modulus at any other strain level. This matter has significant practical and theoretical advantages that will be discussed later.

Based on extensive work in the area of geotechnical earthquake engineering (National Science Foundation, 1994) it is not unreasonable to assume that the values of k_2 and k_3 are not very much affected by specimen disturbance, and as such can be determined from laboratory tests. However, the value of k_1 is extremely sensitive to sample disturbance, and should only be measured on an extremely high quality specimen (which is not practical to retrieve, especially for bases) or through field tests.

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[1] default values from Table 8 are used in figures unless otherwise explicitly mentioned.
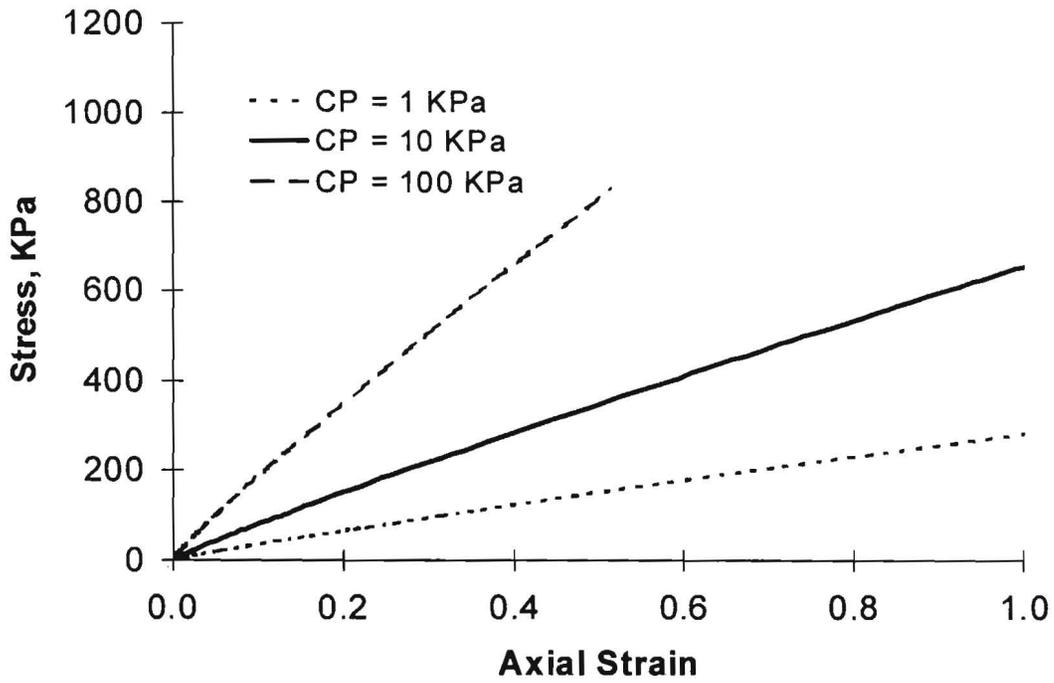
Figure 11 - Stress-Strain Curve for a Typical Base
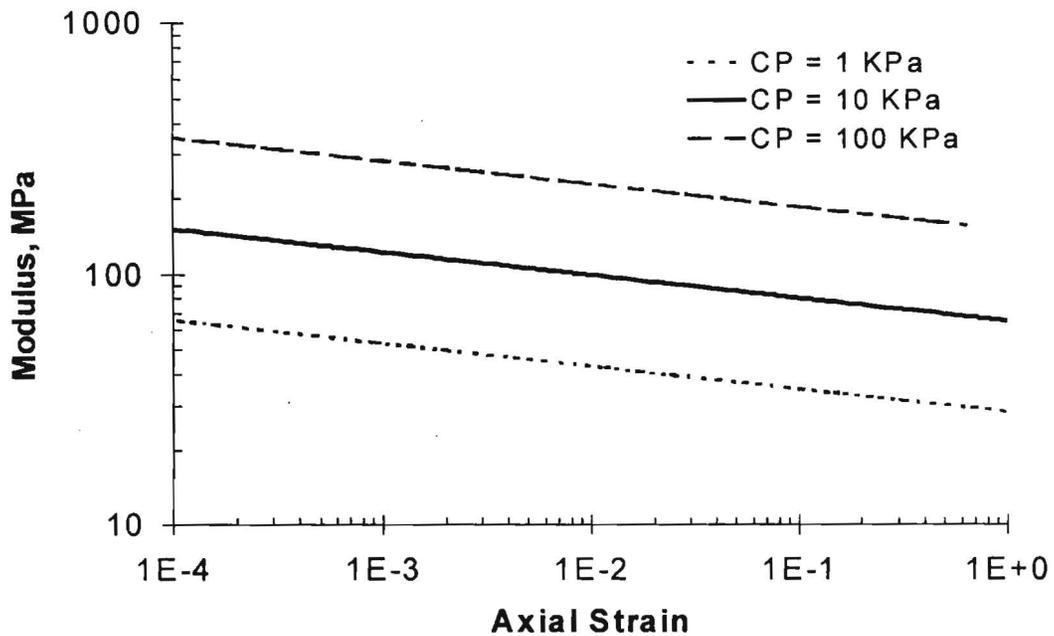
Figure 12 - Variation in Modulus with Axial Strain for a Typical Base
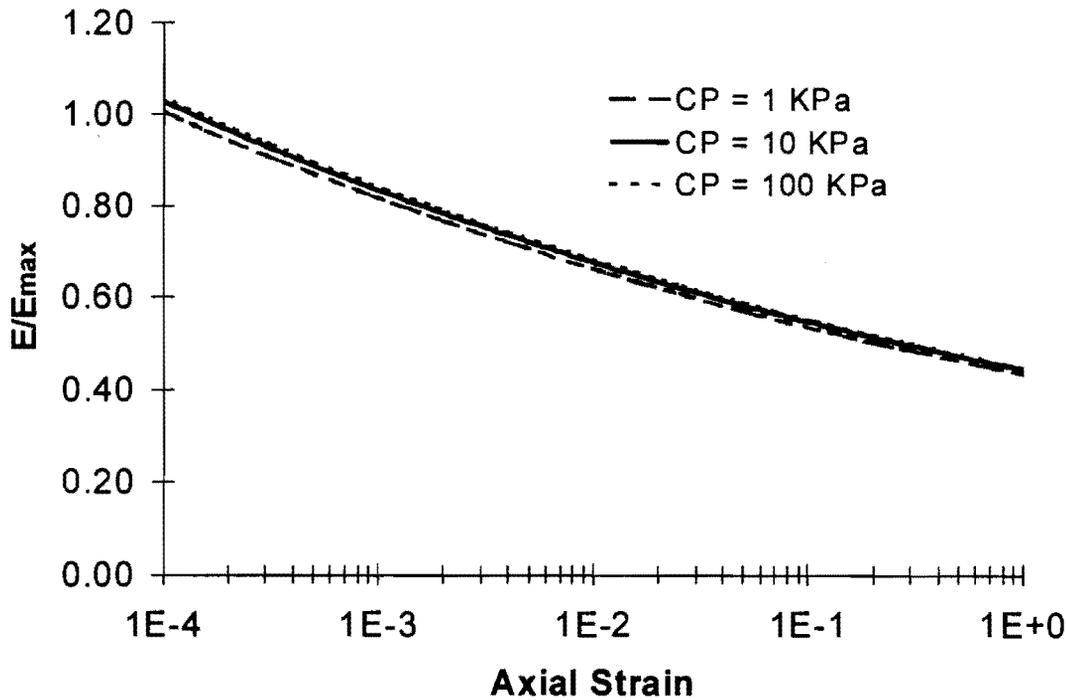
Figure 13 - Variation in Normalized Modulus with Axial Strain for a Typical Base

Another constitutive model that has been extensively used in the earthquake geotechnical engineering area is the Ramberg-Osgood model (Ramberg and Osgood, 1943). The general form of this relationship is

$$\frac{E}{E_{max}} = 1 + \alpha \left(\frac{\sigma}{\sigma_y}\right)^{R-1} \tag{9}$$

where α and R are model parameters determined from curve fitting, σ and σ_y are the applied stress and yield stress, respectively. Without actual test results, values of one and three are typically recommended for α and R, respectively (Anderson, 1974). The yield stress typically corresponds to a percentage of undrained shear strength. Typical variation in normalized modulus (E/E_{max}) with strain is shown in Figure 14. This curve and those shown in Figure 13 are similar. One big difference between the two curves, however, is that the Ramberg-Osgood model becomes asymptotic to E_{max} (initial tangent modulus) at small strain levels; whereas in the model shown in Figure 13, modulus values increase with decrease in strain. A similar trend is observed at high strain, where the high-strain moduli from the model shown in Figure 13 tend toward zero. In actuality the high-strain moduli become asymptotic to a finite value.

Several less known methods can also be used to define the behavior of materials. For the sake of brevity they are not included here. The reader is referred to Kramer (1996).

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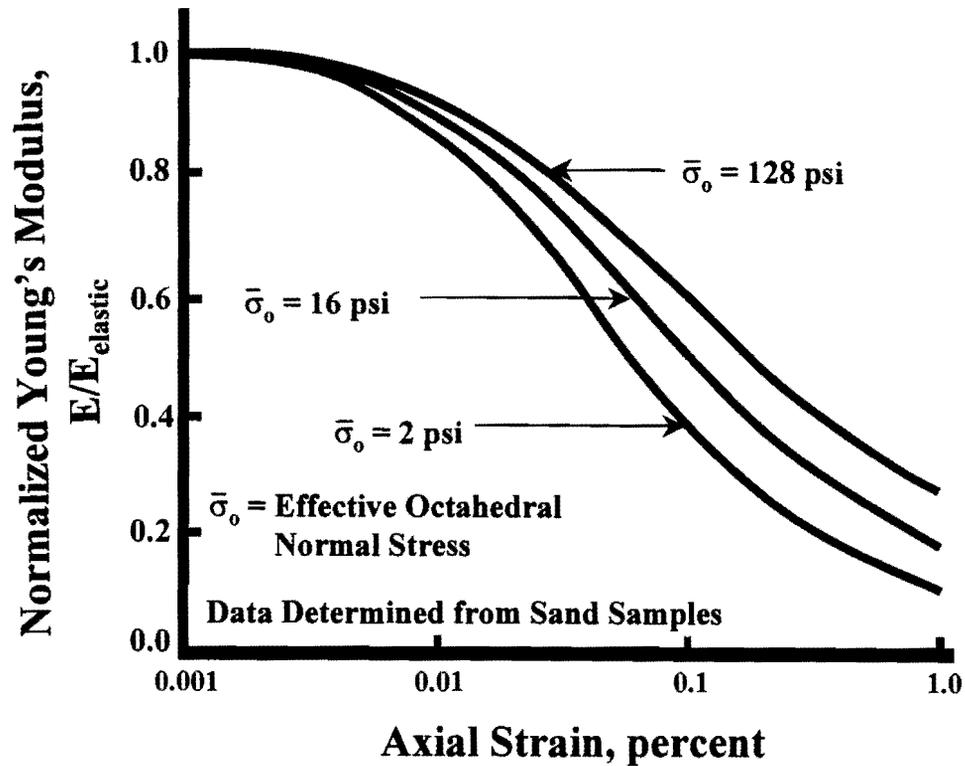
Figure 14 - Typical Variation in Normalized Modulus with Axial Strain Using Equation 9

The figure shows Normalized Young's Modulus, $E/E_{elastic}$ on the y-axis (0.0 to 1.0) versus Axial Strain, percent on the x-axis (0.001 to 1.0, logarithmic). Curves are labeled $\bar{\sigma}_o = 128$ psi, $\bar{\sigma}_o = 16$ psi, and $\bar{\sigma}_o = 2$ psi. $\bar{\sigma}_o =$ Effective Octahedral Normal Stress. Data Determined from Sand Samples.

Table 9 - Parameters that Impact Small-Strain Modulus (after Dobry and Vucetic, 1987)

Parameter	Impact on E_{max}
Void Ratio (e)	decreases with an increase in e
Age (t)	increases with an increase in t
Cementation ©	increases with an increase in c
Overconsolidation Ratio (OCR)	increases with an increase in OCR
Plasticity Index (PI)	increases with an increase in PI if OCR>1 small impact if OCR=1
Strain Rate ($\dot{\varepsilon}$)	increases with an increase in $\dot{\varepsilon}$ if plastic material small impact if nonplastic material
Number of High-Strain Loading (N)	increases with N for coarse-grained materials decreases with N and then recovers with time for fine-grained materials

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So far, the impact of the state of stress has been discussed. As shown in Table 7, other parameters also influence the modulus of a material. These parameters are divided into two groups, those that impact small-strain modulus, and those that affect the variation in normalized modulus.

The most popular relationship to estimate the small-strain shear modulus[2], G_{max}, of both coarse-grained and fine-grained materials is in the form of

$$G_{max} = 625 \ F(e) \ OCR^k \ p_a^{1-n} \ (\sigma_m')^n \tag{10}$$

where OCR is the Overconsolidation ratio, σ'_m is the mean effective principal stress (effective confining pressure), and p_a is the atmospheric pressure.

Parameter F(e) defines the impact of the void ratio (relative density). The most cited relationship for F(e) is in the form of (Hardin, 1978)

$$F(e) = \frac{1}{0.3 + 0.7e^2} \tag{11}$$

This relationship clearly shows the impact of compaction on obtaining high-quality material.

The value of n is typically assumed to be about 0.5 for coarse-grained materials. Baig (1992) performed a study to define this parameter for some Texas soils. His results are summarized in Table 10.

Parameter k is typically related to the plasticity index and varies from zero for a nonplastic material to about 0.5 for an extremely overconsolidated material (Hardin and Drnevich, 1972). For most base and subgrade materials, k is normally less than 0.2.

The rate of loading affects the modulus as well. The rate of loading can be addressed by performing tests with several devices including the resilient modulus tests at several different loading frequencies (between 1 Hz and 30 Hz), torsional resonant column tests (about 20 Hz to 200 Hz),

Table 10 -Typical Values of Parameter n (from Baig, 1992)

Material	Parameter n	
	Loading	Unloading
Clay	(0.15 - 0.20), mean = 0.17	(0.11 - 0.13), mean = 0.12
Sand (angular)	(0.37 - 0.47), mean = 0.44	(0.39 - 0.54), mean = 0.42
Sand (rounded)	(0.48 - 0.50), mean = 0.50	(0.44 - 0.48), mean = 0.46

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[2]G_{max} and be readily converted to E_{max} using Poisson's ratio (v) [i.e., $E_{max} = 2G_{max}(1+v)$]
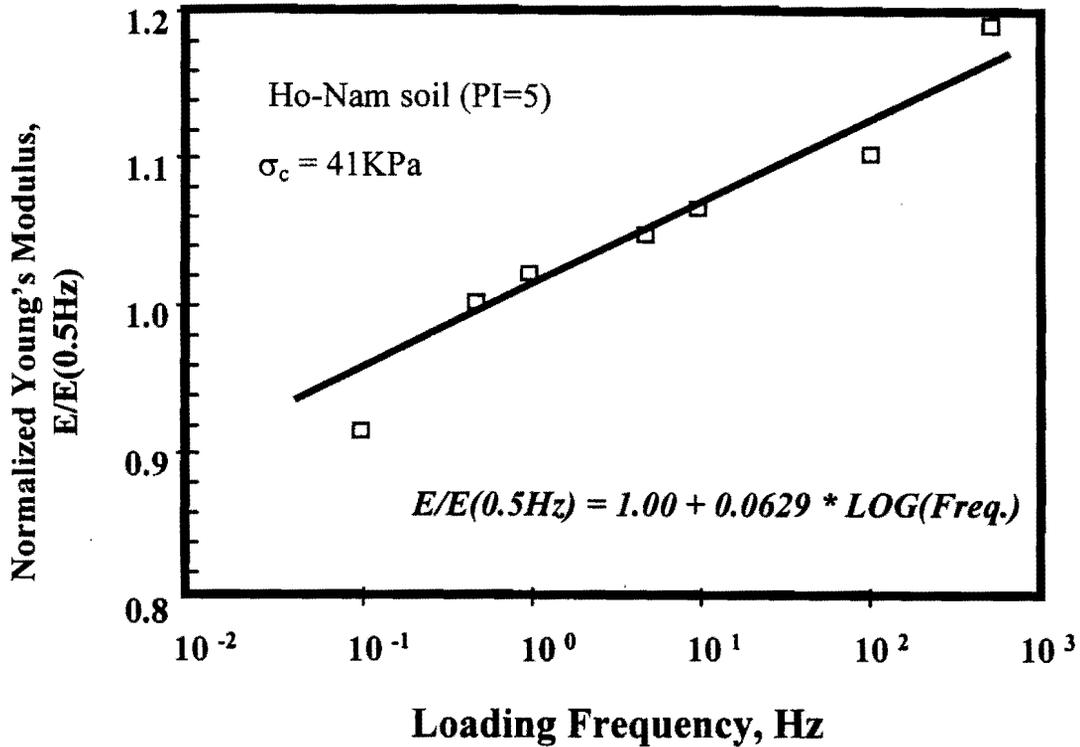
Figure 15 - Typical Variation in Modulus of Coarse-Grained Materials with Frequency (from Kim et al, 1997)

free-free resonant column tests (100 to 300 Hz), and ultrasonic laboratory tests (about 20 KHz). Kim et al. (1997) and Pezo (1993) exhibit how effectively these tests can be combined to determine the initial tangent and secant moduli of these materials.

With the seismic field methods, the typical frequencies at which the modulus is measured are about 1,000 to 5,000 Hz for the base, and about 100 to 500 Hz for the subgrade. Typical variation in modulus with frequency is shown in Figure 15. A correction of about 20 percent for base modulus, and about 5 to 10 percent for subgrade may be needed for the range of loading frequency.

Another parameter to be considered is the age of a layer. The stiffness of a layer typically increases with time. This phenomenon is attributed to either the secondary consolidation (creep) of the layer, or to thixotropic gains. The gain in stiffness is typically shown in the form of (Kramer, 1996)

$$\Delta G_{max} = N_G \ G_{1000} \tag{12}$$

where ΔG_{max} is the increase in stiffness over one logarithmic cycle of time, and G_{1000} is the modulus measured 1000 minutes past the primary consolidation. Parameter N_G, which increases with the

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plasticity index and decreases with OCR (Kokushu et al., 1982), for normally consolidated clays can be determined from

$$N_G = 0.027 \sqrt{PI} \qquad (13)$$

Given the typical PI for bases and subgrades, this parameter may not be of any significance for most bases and some importance for softer clayey subgrades.

The most comprehensive equation for determining the impact of important parameters on the normalized modulus has been developed by Ishibashi and Zhang (1993). That relationship can be written in the form of

$$\frac{G}{G_{max}} = \alpha \ (\sigma'_m)^{\beta - \beta_o} \qquad (14)$$

$$\alpha = 0.5 \ (1 + \tanh \ [\ln(\frac{0.000102 + \zeta}{\gamma})^{0.492}])$$

$$\beta - \beta_0 = 0.272 \ (1 - \tanh \ [\ln(\frac{0.000556}{\gamma})^{0.4}])\exp(-0.0145 PI^{1.3})$$

where γ is the shear strain. Parameter ζ, which varies with PI can be determined from the following table.

Plasticity Index	Parameter ζ	Type of Materials
0	0	Typical sandy or gravelly subgrade with little fine content
0<PI<15	$3.37 \times 10^{-6} \ PI^{1.404}$	Typical granular base, silty subgrade, clayey sand and gravel subgrade
15<PI<70	$7.00 \times 10^{-7} \ PI^{1.976}$	Typical plastic silt, or clayey subgrade
PI>70	$2.70 \times 10^{-5} \ PI^{1.115}$	fat clayey subgrade

This relationship for several PI's and several confining pressures is shown in Figure 16a and 16b. From this equation, as the confining pressure increases, and as the PI increases, the normalized modulus also increases.

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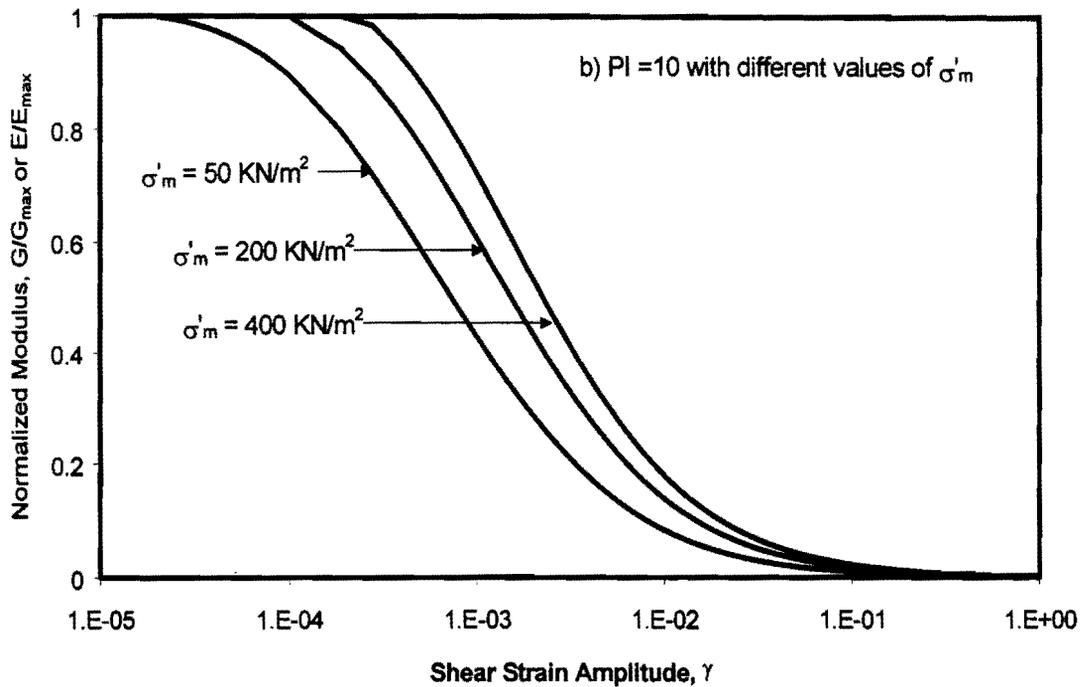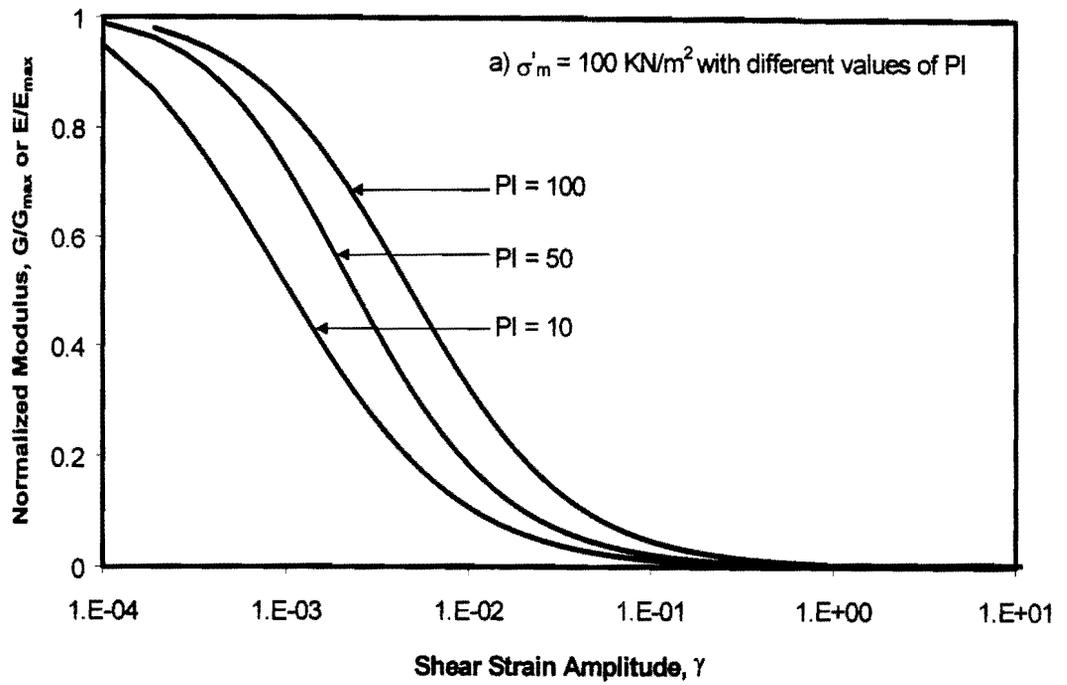
Figure 16 - Variation in Normalized Modulus with PI and Confining Pressure Using Equation 14

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5.2. Bituminous Materials

Modulus of HMA can be determined in several ways. The most common laboratory tests are the resilient modulus, creep, uniaxial frequency sweep, free-free resonant column, and ultrasonic wave velocity tests. The free-free resonant column and the ultrasonic wave propagation methods were described in the previous section, and will not be discussed any further. The main field tests are the FWD and wave propagation tests. These two tests were also described before.

Resilient Modulus tests have been used by many researchers to measure the modulus of HMA (Roberts et al., 1996). These tests can be performed either in compression (similar to soil specimens) or diameterally. The data collection and analysis are very similar to those shown for the resilient modulus tests on granular materials.

In the creep test, the specimen is subjected to a static load. The displacement of the specimen due to the applied load is measured with time. Using the variation in compliance (ratio of the strain and stress) with time, and time temperature superposition principle (Kim and Lee, 1995), the relaxation modulus can be determined.

The uniaxial frequency sweep test is very similar to the cyclic triaxial tests described in the previous section. The stresses and strains under sinusoidal loading are measured (ASTM D3497). Assuming that the material is linear viscoelastic, the dynamic modulus and viscous damping (or storage and loss moduli) are determined. By varying the frequency over a wide range, the variation in modulus with frequency can be determined. The method can be effective over a range of frequencies of 0.5 Hz to 50 Hz.

Several parameters affect the modulus of bituminous materials. The most important parameters to be considered are the rate of loading (i.e., frequency of loading), temperature, and air void content.

The typical frequency at which the AC moduli are measured with seismic methods is about 15 to 25 KHz; whereas the actual traffic load has a dominant frequency of about 10 to 30 Hz. Aouad et al. (1993) clearly demonstrated the importance of considering the rate of loading. As shown in Figure 17, depending on the temperature, the modulus measured with seismic methods should be reduced by a factor of about 3 to 15.

Daniel and Kim (1998), and Kim and Lee (1995) used the results from several laboratory and field tests (such as FWD, ultrasonic, uniaxial sweep, and creep) to show the frequency-dependency of modulus. The results from Daniel and Kim are shown in Figure 18. Again, the frequency-dependency is temperature related.

The AC modulus is strongly dependent on temperature. Von Quintus and Killingworth (1998) demonstrate the importance of temperature correction, and complexity involved in considering the temperature gradient within a pavement section. Aouad et al. (1993) and Li and Nazarian (1994) and several other investigators have studied the variation in modulus with temperature for seismic methods. Many relationships exist that recommend means for temperature adjustment. However, a comprehensive model that is universally accepted does not exist. With the advancement in
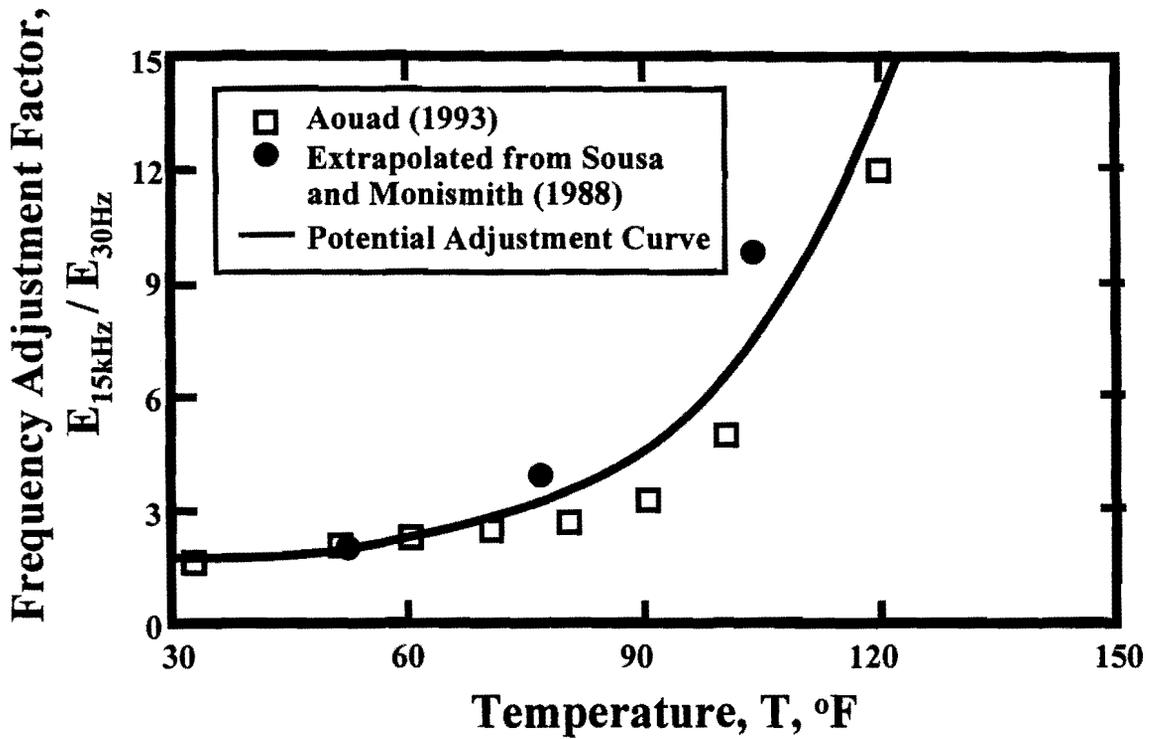
Figure 17 - Variation in Modulus with Frequency and Temperature (from Aouad et al., 1993)
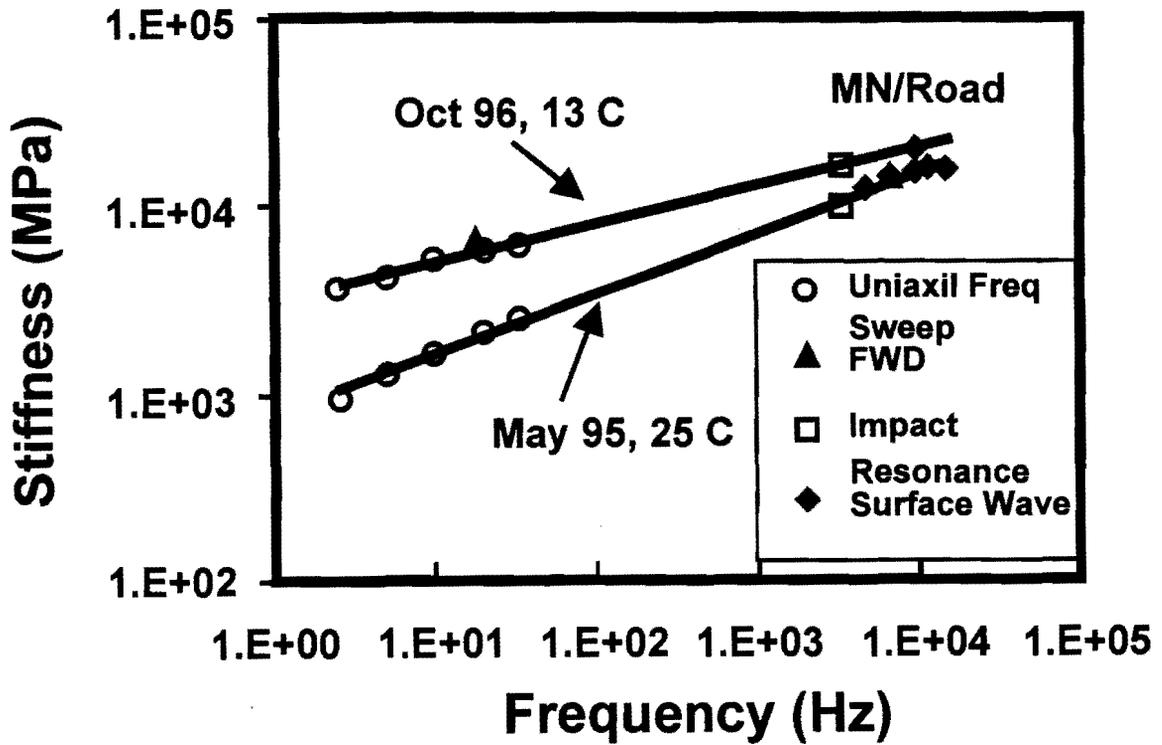
Figure 18 - Frequency and Temperature Dependency of AC Modulus
(from Daniel and Kim, 1998)

30

50
40 $y = -1.6472x + 36.172$
$R^2 = 0.9723$
Modulus, (GPa)
30
20
◆ V-Meter
10
0

0   2   4   6   8   10   12
Air Voids (%)
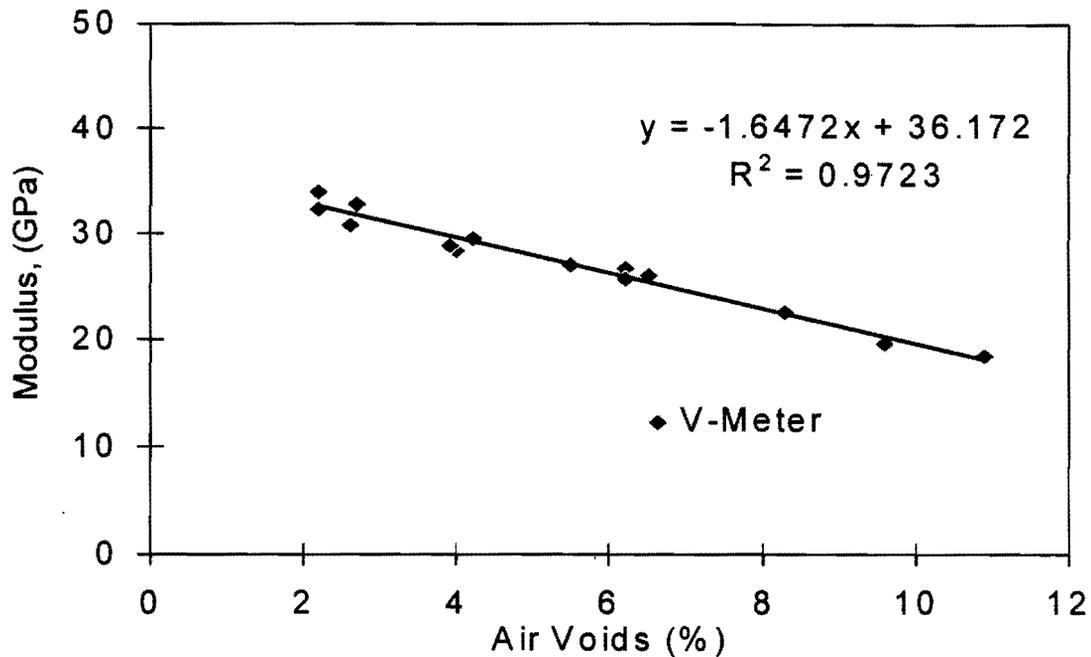
Figure 19 - Impact of Air Void Content on Modulus of AC (from Rojas, 1998)

measuring the modulus of pavements, methodology for temperature correction should be studied and improved.

Air void content has a significant impact on the modulus of AC as well. Rojas (1998) clearly demonstrated that the modulus of a mix is inversely proportional to the air void content of the mix (see Figure 19). He also showed that the aggregate gradation and the asphalt viscosity affect the modulus of the mix.

6. Modeling

Brown (1996) discusses a spectrum of analytical and numerical models that can be used in pavement design. The intention of these models is to find out the critical stresses, strains and deformations within a pavement structure. Many computer programs with different levels of sophistication exist. These programs are described below.

6.1. Linear Models

The simplest form of modeling the behavior of a pavement is by using layered elastic models. Most algorithms used in pavement analysis and design take advantage of this type of solution. The advantage of these models is that they are quite rapid and they are readily available. Three of the most popular programs in this category are MODULUS, WESDEF and KENLAYER.

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6.2. Equivalent-Linear Models

These programs are based on the static linear elastic layered theory. However, an iterative process is employed to consider the nonlinear behavior of the material in an approximate fashion. To implement these methods, one to two dozen points within the pavement structure at different depths and horizontal distances from the loading point are selected. An initial modulus is assigned to each point, so that the confining pressure and axial strain at that point can be determined. The modulus at each point is calculated using the calculated confining pressures and axial strains (Equation 8). The assumed and calculated moduli are compared until the differences are small. This method is rapid. However, the results are approximate because in a linear elastic layered solution, the lateral variation of modulus within a layer cannot be considered. If found reasonable, these programs have the highest chance for implementation. A program capable of this feature was not readily available. Therefore, WESLEA was modified to conduct this task. The results will be discussed later.

KENLAYER (Huang, 1994) also has a feature that considers the nonlinear behavior of the pavement layer in a reasonable fashion. However, the constitutive model used in that program is not compatible with the one assumed in this study. The author of the program can readily modify that algorithm. Unfortunately, the source code for that program is not available.

Several programs have used the constitutive models similar to those shown in Equation 8 in their linear elastic solutions. However, they do not incorporate a trial and error scheme to obtain more consistent results. This matter, as shown later, may not incorporate the nonlinear behavior of the pavement in an acceptable fashion.

6.3. Simplified Finite Element Models

Several simplified finite element models have been developed in the last few years (for example MICHPAVE). The main advantage of these models is that only a minimum interaction between the user and the model is required. The software virtually develops a finite element mesh, and conducts either a linear or a nonlinear analysis. The major limitations of these methods are that the mesh is too simplified and the material properties are set in a way that cannot be changed. Without access to the source code, the calibration of these programs for the use in Texas may not be feasible. Most of these programs also use constitutive models that are not in compliance with the three- parameter model of Equation 8.

6.4. Comprehensive Finite Element Models.

All purpose FE software packages (such as ABAQUS, NASTRAN or ANSYS) can be used for this purpose. These programs allow users to model the behavior of a pavement in the most comprehensive manner, and to select the most sophisticated constitutive model for each layer. The dynamic nature of the loading can also be considered. The analytical solutions are highly efficient and are quite advanced. The disadvantage of these programs is that a highly skilled engineer with expertise in finite elements should review the input and output to ensure that all aspects of modeling are considered.

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Custom-made comprehensive computer programs for pavement applications can be developed (see Brown, 1996). This should be done as a final solution, since the development, upgrade and modification of these programs are labor-intensive and expensive.

7. Previous Uses of Seismic Moduli in Geotechnical Earthquake Engineering

The process of determination of the behavior of pavements close to wheel loads is not much different from the process of determining the site amplification characteristics of a soil deposit subjected to earthquake loads. In both cases, the nonlinear behavior of the material should be considered. In both problems, in situ seismic moduli along with laboratory tests can be used to determine the stresses and strain within the soil medium. The main difference is that for pavements, a known load is applied at the surface; whereas in the geotechnical earthquake engineering, the motion of the bedrock at some depth is known.

To solve these problems, the equation of motion within a layered medium has to be used. In fact, if a linear elastic material is assumed, most of the algorithm used in either program will be practically identical. As the nonlinearity of materials is well known, the linear approach must be modified to provide reasonable estimates of the response for practical problems. The method most widely used to approximate the nonlinear behavior is known as the equivalent-linear method. In that method, as discussed before, the secant moduli are used in a linear model. Since the model is linear, the modulus of each layer should be assumed as a constant. The problem is then to determine a secant modulus that yields compatible strain induced at each point. Since the computed strain level depends on the value of the equivalent linear properties, an iterative procedure is required to ensure that the properties used in the analysis are compatible with the computed strain levels. The Ramberg-Osgood type model (see Figure 13 or 14) is used to define the nonlinearity of the material.

Several programs exist that can perform this task. SHAKE (Schnabel et al., 1972) is the most famous one. Although not satisfactory as a research tool, the program has been used with success in many design projects (Glaser, 1993), and perhaps is the state of practice in that area.

The same work frame can be readily employed with the existing elasto-static layered elastic programs (such as WESLEA) to predict the state of stress or strain within a pavement structure. This methodology can also be readily applied to any elasto-dynamic analysis program. In that case, the solution can be either in the time or in the frequency domain. If frequency-domain solution is used, care should be taken to ensure that the analysis is extended up to a frequency that appreciable energy exists in the impact (Kramer, 1996). The variation in properties with time during the impact is also not considered in the equivalent linear methods. However, since a well-designed pavement should experience small to moderate strain levels, this should not be of a great concern. The other limitation is that the method is not robust at large-strains. Again, most roads do not experience large strains under wheel loads.

Nonlinear approaches have also been used in the geotechnical earthquake engineering field to characterize site response. In this case, the solution has to be done in the time-domain. As such, the computation time may often be excessive. The reasonableness of the results is directly related to the adequacy of the nonlinear model used to define the properties of each layer, and the quality of the laboratory data. Arulanandan et al. (1994) describe the results of comparing analytical prediction

with model tests in a centrifuge under VELACS (Verification of Liquefaction Analysis Using Centrifuge Studies) research program. They concluded that the analytical models predict the behavior of the soil deposits reasonably closely.

All these models of course require a reasonable constitutive model. The consensus in this area is that the field and laboratory results should be reconciled. In the late seventies, several studies were carried out to combine laboratory and field moduli. A reasonable and widely-accepted procedure is to determine the small-strain modulus from field tests, and combine it with normalized modulus. Stokoe and Chen (1980) summarize some of the methods that are available. These methods include: percentage increase (Seed and Idriss, 1970), arithmetic increase (Richart et al., 1977), linear decrease (Taylor and Larkin, 1978), and reference strain (Drnevich and Massarsch, 1979).

In the percentage increase method, the laboratory moduli are adjusted upward by the ratio of field modulus and the small-strain laboratory modulus. The arithmetic increase method consists of upward translation of the laboratory moduli by the difference between the field and small-amplitude laboratory modulus.

In the linear decrease method, the modulus-strain curve is divided into three parts. For small strains (less than say 0.001 percent), an adjustment factor equal to the ratio of field and small-amplitude moduli is used. At high shearing strain (greater than 1 percent), a constant value equal to a fraction of the field modulus is assumed. Between these two limits, a linear relationship between the adjusted modulus and logarithm of strain is assumed. This model is particularly very attractive for use in our application, since the model defined by Equation 8 easily provides the slope of the line between the lower and upper strain limits.

In the reference strain method the strain axis is normalized with respect to a reference strain. The reference strain used is the ratio of the shear strength divided by the small-strain modulus of a specimen. The shear strength can be obtained by loading the specimen to failure after the completion of the resilient modulus tests. Assuming that the reference strain is the same for laboratory and field results, the modulus strain curve can be adjusted upward. One of the issues to be addressed in the future research is which one of these models is most appropriate.

One of the most significant studies in implementing seismic moduli in characterizing bases and subgrades has been recently completed by Florida DOT (Horhota, 1996). Horhota performed many SASW tests in a large-scale test pit operated by FDOT and at several actual field sites. His goal was to correlate seismic moduli with conventional test methods used in pavement design (such as static and dynamic plate load tests, resilient modulus test). A typical variation in moduli with axial strain and confining pressure for one site is shown in Figure 20. The results from the dynamic and static plate load tests (denoted as "Dynamic" and "Static" in the figure) provide moduli at large strain levels. The results from the resilient modulus tests (denoted as M(r) in the figure) provide moduli at intermediate levels. Horhota assumed that the modulus from the SASW tests corresponded to the small strain modulus of the material. He also assumed that value would be constant below a strain of 0.001 percent. As expected the seismic modulus is larger than the other moduli. Another significant matter is that the data readily conform to the model presented in Equation 8. The separate data sets correspond to separate confining pressures and deviatoric stresses.

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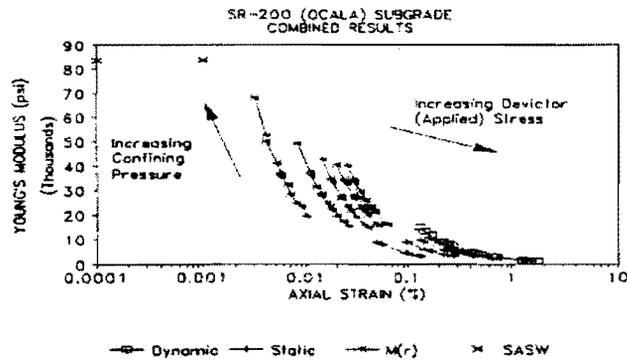
Figure 20 - Variation in Young's Modulus and Strain for a Subgrade Site (from Horhota, 1996)
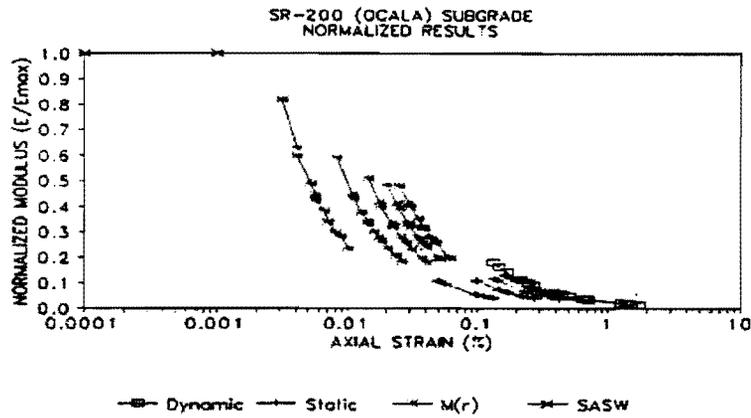
Figure 21 - Variation in Normalized Modulus and Strain for a Subgrade Site (from Horhota, 1996)
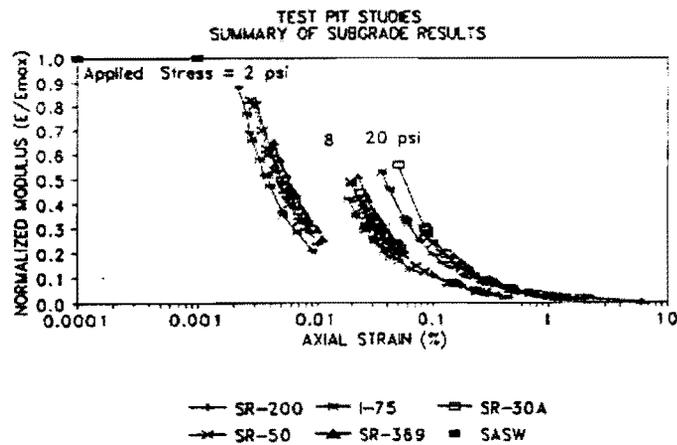
Figure 22 - Variation in Normalized Modulus and Strain for Several Subgrades in State of Florida (from Horhota, 1996)

In the next step, Horhota normalized the moduli with respect to seismic modulus (see Figure 21), so that the variations in moduli from different sites can be compared. The variations in normalized moduli as a function of strain for several of his sites are shown in Figure 22. These results clearly confirm that the normalized curve for a group of soils is material independent. The significance of this statement is that parameters k_2 and k_3 in Equation 8 can be determined for one type base and subgrade in a given region once. This will reduce the need for extensive laboratory tests. Therefore, the only parameters that should be measured are the seismic moduli of different layers.

In Figure 22, the normalized moduli in the intermediate and high strain regions favorably resemble the Ramberg-Osgood curve (see Equation 9 and Figure 14). Because small-strain moduli could not be measured in the laboratory, the shape of the curve cannot be defined. Since in only rare occasions the large-strain moduli can be measured, one can use Equation 8 for the intermediate strains. The normalized moduli for the small strains and high strains can then be set to 1 and a small value (say 0.05), respectively. Horhota also demonstrated the applicability of this procedure to granular bases and compacted embankments. These results are not shown for the sake of brevity.

8. Design of Pavements with Seismic Methods

As the goal of this project is to study the feasibility of incorporating seismic moduli in design process, defining a framework for pavement design (in general), and flexible pavement design using seismic modulus (in particular) would be desirable. This framework is then used to define the process that should be developed and the research that should be carried out to design based on seismic moduli.

8.1. A Framework of Understanding

The components of a mechanistic design procedure are summarized in Figure 23 (NCHRP, 1992). Each component is described briefly.

1. The inputs to the system are the material properties, traffic and climatic condition. The material properties entered into the model can be of any degree of sophistication (from very basic to very sophisticated and complex). The level of sophistication depends on the level of laboratory and field tests an agency is willing to partake. The laboratory and field-testing program should be balanced with the level of accuracy with which the traffic is classified.

2. The input parameters are used with a structural model to determine the pavement responses in terms of stress and strain developed within the pavement structures. The level of sophistication of the algorithm chosen by an agency should be harmonized with the level of sophistication assigned to the field and laboratory tests. For example, if the layered elastic model is used for determining the pavement responses, performing a sophisticated, but time-consuming, resilient modulus test on the base and subgrade may not be necessary. Such a test, will provide a nonlinear material characterization that is difficult to incorporate in a simple linear-elastic layered program. However, that does not mean that a simpler version of the test that would take much less time should not be performed.

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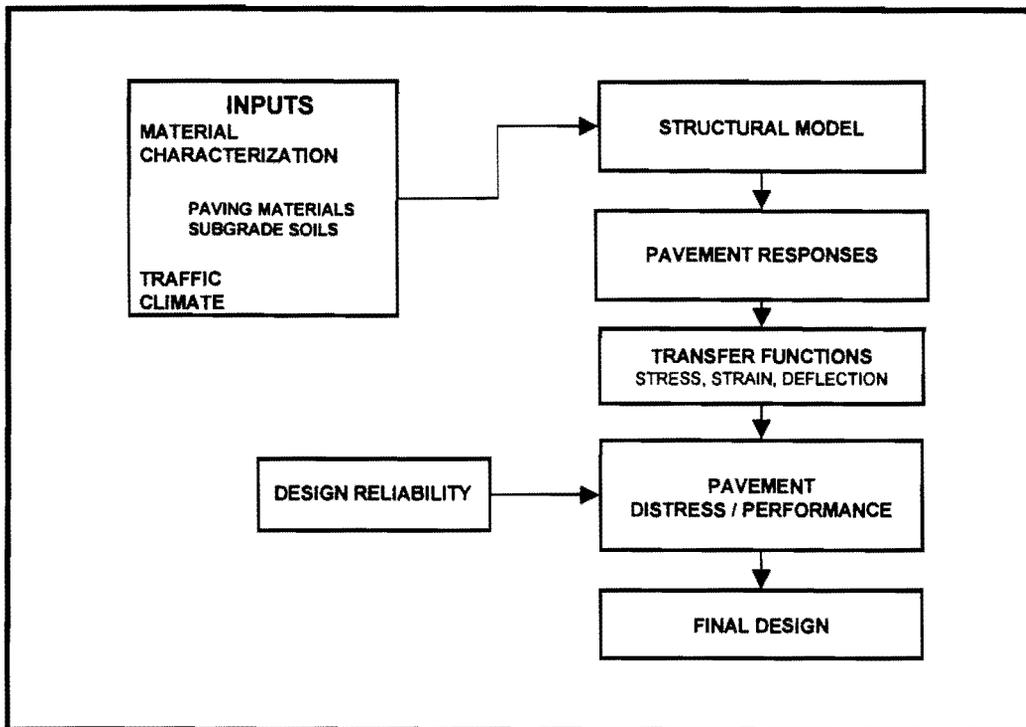
Figure 23 - Components of Mechanistic Design Procedure (from NCHRP, 1992)

3. The pavement responses determined from step 2 should be combined with models that relate structural response to the magnitude of structural distress (such as rutting, cracking, etc.). This is done through relationships that are typically called transfer functions. The reasonableness of the design is directly related to the accuracy of these models.

4. The results from step 3 along with other functional considerations are compared with the agency criteria and definitions of failure to determine the adequacy of the proposed design. Of course considering the reliability of the results, based on the uncertainties associated with the models and the input parameters, is very reasonable and prudent.

The success of this process as a whole is directly related to how well the input parameters, the structural models and the transfer functions are balanced. To do so, having different levels of sophistication associated with different types of roads managed by the highway agency may be reasonable. For example, performing a simple DCP test at regular intervals, and using simplified empirical models may be adequate for many tertiary roads in the network. Secondary roads can be simply designed using criteria developed based on FWD results and linear-elastic models. However, the major highways, should benefit from a thorough laboratory and field tests, with a reasonably sophisticated nonlinear elastic algorithm.

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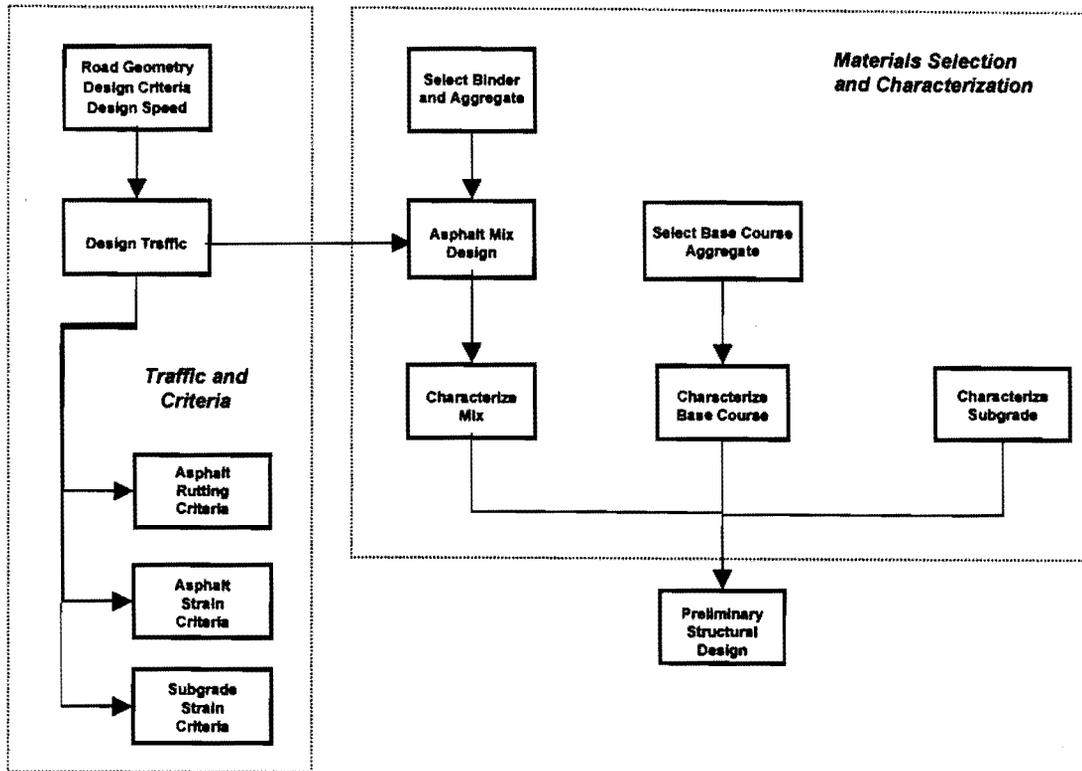
Figure 24 - Flow Chart of Thickness Design Methodology

8.2. Material Characterization

Our focus on this project is of course related to the input parameters. In that aspect, the flow chart included in Figure 24 demonstrates the process. With full consideration to the failure criteria of interest and the quality and quantity of traffic loading, the asphalt concrete, base and subgrade are selected and designed.

8.2.1. Asphaltic Concrete Layer

For a comprehensive design, the variations in the stiffness of AC with temperature, and with frequency of loading should be considered. The type of asphalt and the gradation and quality of aggregates, which affect these properties, should still be considered using volumetric mix design and local experience.

Advantages.   The state-of-practice in the laboratory or field testing has typically been shown to be ineffective in yielding repeatable and reliable modulus for AC layers. Seismic methods are currently the most accurate, rapid and repeatable methods for determining moduli in the laboratory (Rojas, 1998). In addition, rapid field methods exist that provide similar moduli to those measured in the laboratory. Therefore, the seismic methods are the only

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methods that can <u>directly</u> and in a <u>fundamentally-correct</u> manner provide similar moduli in the laboratory and in the field. With seismic methods, one can readily develop a modulus-temperature relationship with small effort (for instance Li and Nazarian, 1994).

Disadvantages. Seismic moduli are measured at small strains and high frequencies. As indicated before, other laboratory methods are available, that when combined with the seismic methods, can be used to describe the viscoelastic behavior of a mixture. As indicated in Section 5, several researchers have shown that developing relationships to adjust for the rate of loading is feasible.

Issues to be resolved. The main issues to be resolved are to develop a comprehensive model that relates the seismic moduli to moduli measured at strain levels and loading rate that are compatible with loads imposed by vehicles.

Another issue, which has broad impact in pavement characterization, is the method of temperature adjustment. A study should be carried out to define the best possible model for adjusting modulus with temperature, especially in the presence of a temperature gradient in the AC layer. It should be emphasized that this issue is prevalent to all methods used to characterize AC layers, and is not specific to seismic moduli.

8.2.2. Base Materials

For the base materials, the most important parameter to be considered is the nonlinear behavior of that layer. A model similar to the one presented in Equation 6 or 8 (or any other variation of it) should be used. The considerations for the nonlinear behavior of pavements when seismic modulus used in the design are schematically shown in Figure 25. As indicated in the figure, the modulus of base is measured at a relative small confining pressure (i.e., geostatic pressure) and small strain levels. However, under a wheel load, the confining pressure and strain level are increased. As indicated before, an increase in the confining pressure at a given strain will normally result in an increase in modulus. On the other hand, an increase in strain will generally result in a decrease in modulus. The combination of these two parameters will dictate if the modulus under the wheel load is larger, smaller or equal to seismic modulus. For a typical Texas base material, the modulus under a standard 40 KN axle load is about 1.5 to 3 times less than seismic modulus (Nazarian et al., 1998b).

To incorporate seismic modulus in a model, Equation 6 is reported herein another time.

$$M_R = k_1 \, \sigma_c^{k2} \, \sigma_d^{k3} \tag{15}$$

as indicated schematically in Figure 25

$$E_{seismic} = k_1 \, \sigma_{geo}^{k_2} \tag{16}$$
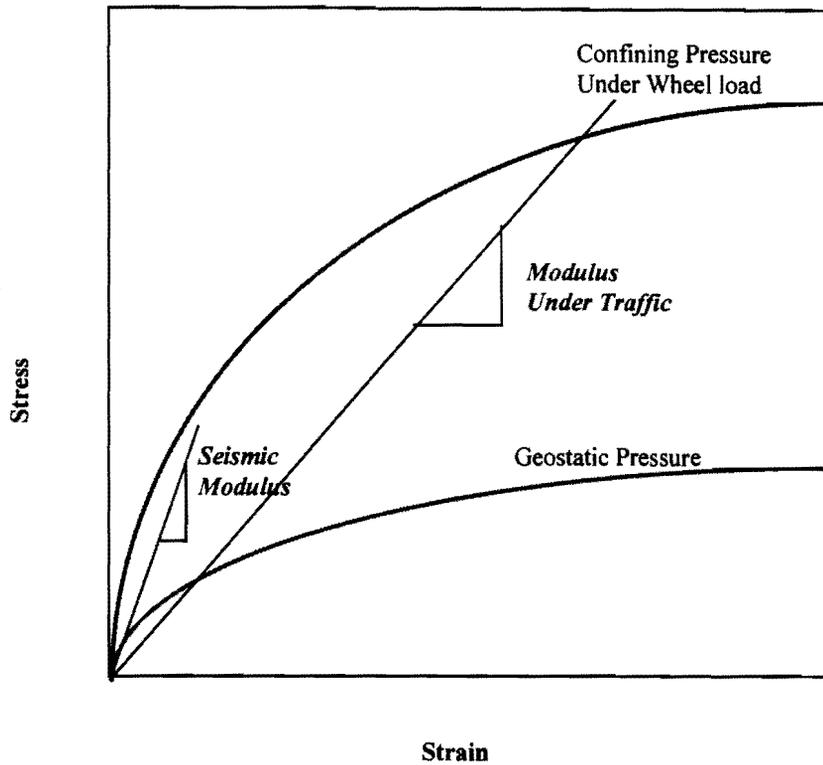
Figure 25 - Schematic Demonstration of Adjusting Seismic Modulus for Applied Loads

where σ_{geo} is the geostatic stress due to the weight of the overlying materials and is equal to $\sigma_v(1+2k_o)/3$. Parameter k_o = coefficient of earth pressure at rest, and σ_v = vertical stress due to the weight of the soil or pavement layers. From Equation 16 one can deduce that

$$k_1 = E_{seismic} / \sigma_{geo}^{k_2} \tag{17}$$

By substituting Equation 17 into Equation 15, for intermediate strains

$$E = E_{seismic} (\frac{\sigma_c}{\sigma_{geo}})^{k_2} \sigma_d^{k_3}, \tag{18a}$$

for small strains, where the material behaves in a linear-elastic manner

$$E = E_{seismic} (\frac{\sigma_c}{\sigma_{geo}})^{k_2} \tag{18b}$$

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and for large strains

$$E = E_{seismic} \ (\frac{\sigma_c}{\sigma_{geo}})^{k_2} \ \alpha$$ (18c)

where, as indicated before, α is a number about 0.05 to 0.10. Using Equation 7, Equation 18b can be written as in terms of strain in the form of

$$E = (E_{seismic})^{\frac{1}{1 - k_3}} \ (\frac{\sigma_c}{\sigma_{geo}})^{\frac{k_2}{1 - k_3}} \ \varepsilon_{ax}^{\frac{k_3}{1 - k_3}}.$$ (19)

The method to incorporate these relationships in pavement analysis was discussed before, and will not be repeated for the sake of brevity. A reasonable computer algorithm should also be considered. To incorporate the nonlinearity in a reasonable fashion, an equivalent linear or a finite element code has to be used.

Advantages.   The seismic methods can provide a <u>fundamentally-correct</u> modulus at a known state of stress (i.e., confining pressure equal to geostatic pressure, and strain in linear elastic range). Therefore, if a constitutive model is developed for a material, moduli at other states of stress can be estimated. Laboratory tests exist that provide similar moduli in the laboratory and in the field.

Disadvantages.   Seismic moduli are measured at small strains, and therefore, adjustment is always required. The moduli developed from inversion may be nonunique. TxDOT personnel has to yet be trained to use a new equipment.

Issues to be resolved. The main issues to be resolved are to develop a comprehensive model that relates the seismic moduli to moduli measured at strain levels and loading that are compatible with loads imposed by vehicles. This has shown to be feasible by Nazarian et al. (1987) and Horhota (1996). Many years of research in geotechnical earthquake engineering have shown that the parameters k_2 and k_3 are material independent, and that can be predicted using PI of the material. This point has not been proven for typical base materials used in Texas. Therefore, this matter should be pursued. If the values of k_2 and k_3 can be established for different bases in the state, the need for laboratory tests will be reduced.

Another issue is what type of model to be used to determine the state of stress under wheel loads. Equivalent linear models are quite attractive because of their speed and their ease of use. On the other hand, finite element analyses provide more comprehensive means of modeling the pavement, at the expense of complexity in operation. Balancing the complexity of nonlinear material model and the analysis model should be carefully studied.

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Another minor point is that some base materials that are rich in clay particles may require a correction for strain rate as discussed in Table 8. Adjustment should be small (roughly 10 to 20 percent in extreme cases) as discussed in previous sections. Nevertheless some effort is needed to ensure that this relationship is developed.

8.2.3. Subgrade

The advantages, disadvantages and issues that face the base are also relevant to subgrade. The material models described above can still be used for subgrades. The subgrade has to be divided into several layers. Ample evidence exists that subgrade next to base typically stiffer than deeper materials, and may behave more nonlinear.

9. Case Study

Several case studies are incorporated in this section to clarify some of the ideas explained before. In the first study, the impact of considering nonlinear behavior of a pavement is shown. A sensitivity study is then presented to show if considering the nonlinear parameters for bases and subgrades is necessary. In the final study, a case is included to demonstrate how the laboratory moduli and field moduli from (FWD and SPA) can be potentially combined. More work is in process to make these case studies more comprehensive, and will be included in the final report of this project.

9.1. Impact of Nonlinear Behavior of Pavement Materials on Response of Pavement Structure

To show the impact of assuming nonlinear behavior of pavements on the response of the pavement, the results from the response of a FWD on top of a pavement were simulated with the finite element program ABAQUS. A pavement section equivalent to the "Thin AC-Thick Base" pavement structure described in Section 3 and Table 1 was used. As a reminder, the thicknesses of the AC and base layers were 75 mm and 300 mm, respectively. The moduli of the AC, base and subgrade were 3.5 GPa, 350 MPa and 70 MPa, respectively. The nonlinear model shown in Figure 14 was applied to the base and subgrade.

The deflection basins under a load of 11 KN are shown in Figure 26. The deflections from the first two sensor locations (i.e., zero and 0.3 m) substantially differ from the two analyses, while the deflections from the last four sensor locations (spacings greater 0.45 m) are similar. This suggests that the assumption of a nonlinear material property will affect the response of the pavement. However, such effect is rather localized. Only the areas of the pavement "relatively" close to the load display any manifestation of the choice in model. Stated differently, areas that are several times away from the diameter of the loaded area are not affected by the nonlinear behavior of the pavement. More quantitative analysis of this matter is presented in the next case studies.

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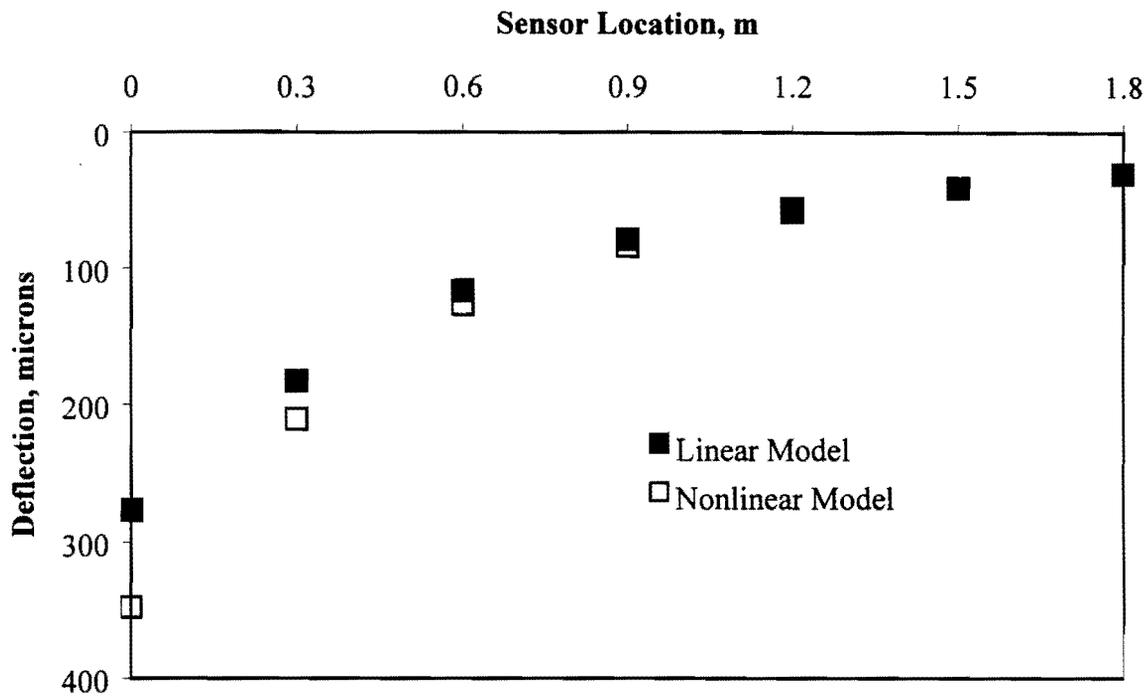
Figure 26 - Variation in Deflection Basins as a Function of Analytical Model Used to Calculate Them (Applied Load of 11 KN)
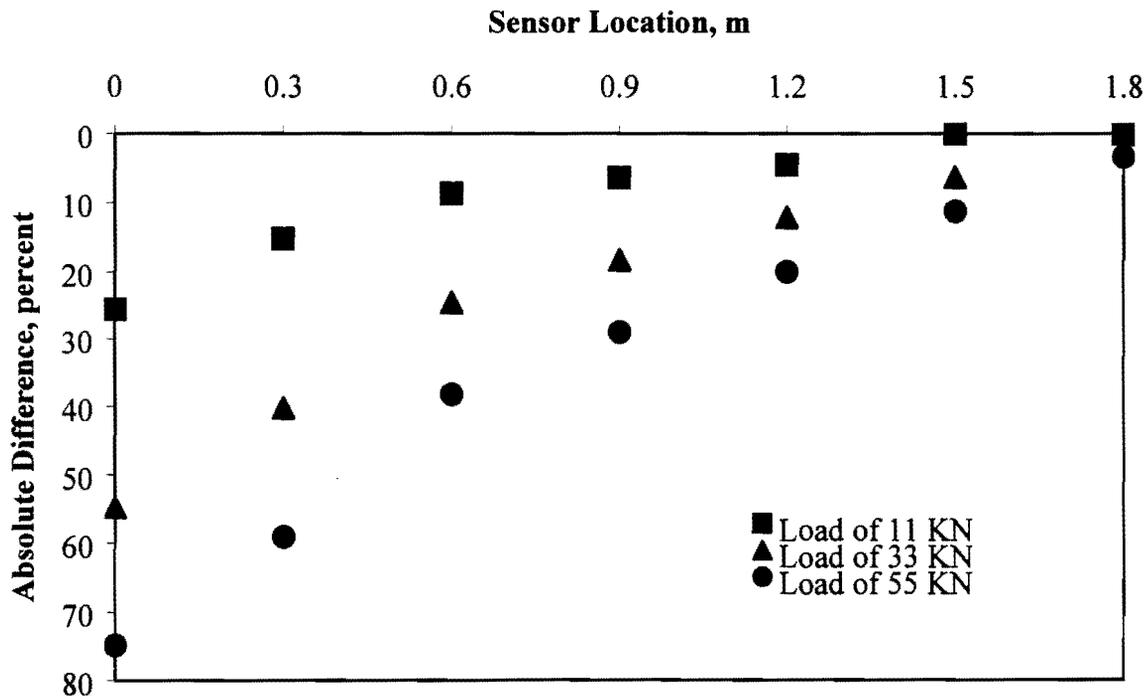
Figure 27 - Variation in Percent Difference in Deflection as a Function of Load Intensity

The difference between deflections calculated with linear and nonlinear models are compared in Figure 27. For a load of 11 KN that corresponds to the results from Figure 26, the differences are less than 10 percent for distances more than 0.4 m from the load. However, as the applied load increases, a larger area of the pavement is affected by the nonlinear behavior of the pavement. As an example, for a load of 33 KN, sensors placed farther than 1 m provide less than a 10 percent difference in deflections between linear and nonlinear models. By increasing the load to 55 KN, sensors placed less than 1.5 m from load produce deflections that are more than 10 percent different. This exercise demonstrates that the larger the load applied to the pavement is, the more important it will be to consider the nonlinear behavior of the materials.

Boddapati and Nazarian (1994) performed an extensive study to quantify the impact of the nonlinear behavior. They showed that if the nonlinear behavior is not taken into account, the mechanistic algorithms recommended for rutting and cracking by the Asphalt Institute will theoretically result in remaining lives that are 1/4 to 5 times the ones calculated.

9.2 Sensitivity Analysis of Remaining Life to Nonlinear Parameters

The second practical point to be considered is which of the nonlinear parameters that enter in Equations 15 through 19 will influence the remaining life of the pavement. A study very similar to that followed in Section 3 was carried out. However, instead as perturbing the modulus, thickness and Poisson's ratio of each layer, the three nonlinear parameters in Equation 15 (i.e., k_1, k_2 and k_3) of the base and subgrade were perturbed. As a reminder, a database containing 100 samples was developed for each parameter by assigning a coefficient variation of 10 percent, and by using the Monte Carlo simulation techniques (Ang and Tang 1984a, 1984b). For each sample, the remaining lives were then calculated and compared with the deterministic remaining lives.

The typical variations in remaining life as a function of the nonlinear parameters are shown in Figure 28 for the "Thin AC-Thick Base pavement." The baseline properties of this section are given in Section 3, and were reviewed in the previous section. Please note that only the base is considered nonlinear. The value of k_1 was selected in a manner so that the modulus at the middle of the base layer was the same as those reported in Table 1. The values of k_2 and k_3, were assumed to be equal to 0.37 and -0.4, respectively. These values were selected to be representative of a reasonable base in Texas.

As indicated in Figure 28a, a 10 percent variation in k_1, k_2 or k_3, will result in a variation of about 12 to 15 percent in the remaining life due to rutting. Using the levels of significance defined in Table 2, all these three parameters are moderately significant (i.e., they must be measured to limit errors in design).

For the cracking criteria however, a 10 percent variation in each of the three parameters results in a coefficient of variation of about 50 percent (see Figure 28b). Such a large standard deviation suggests that these parameters should be measured relatively accurately.

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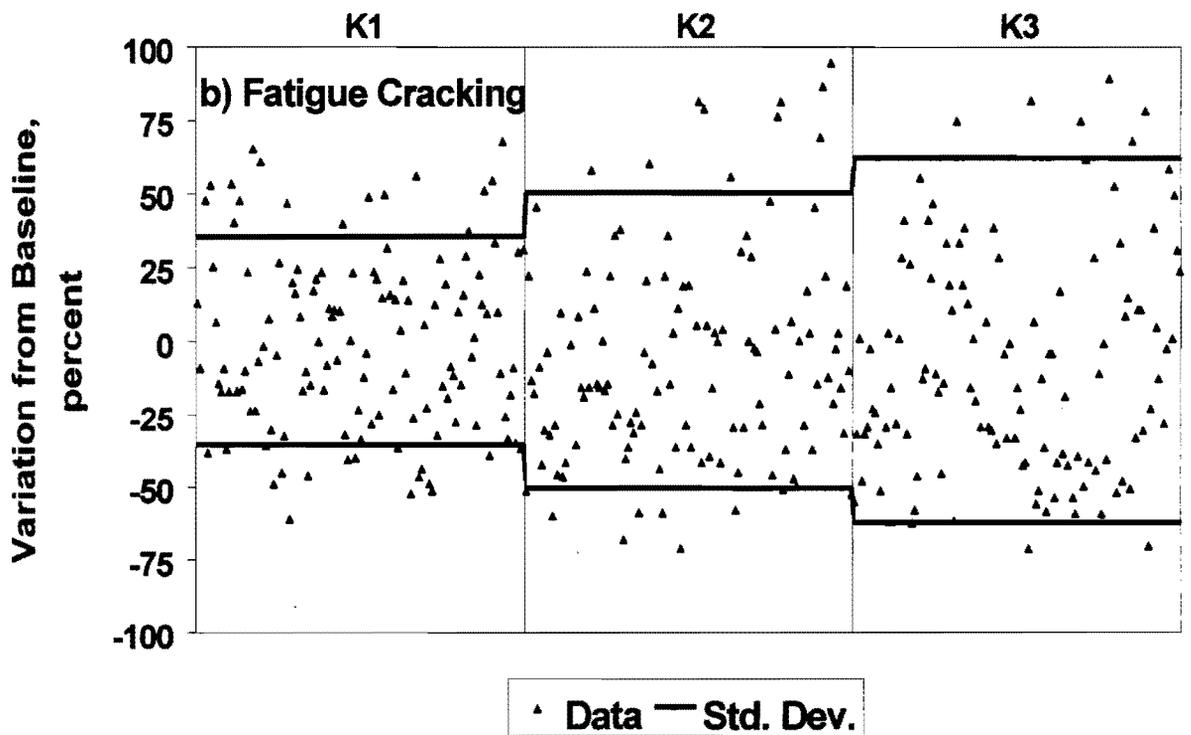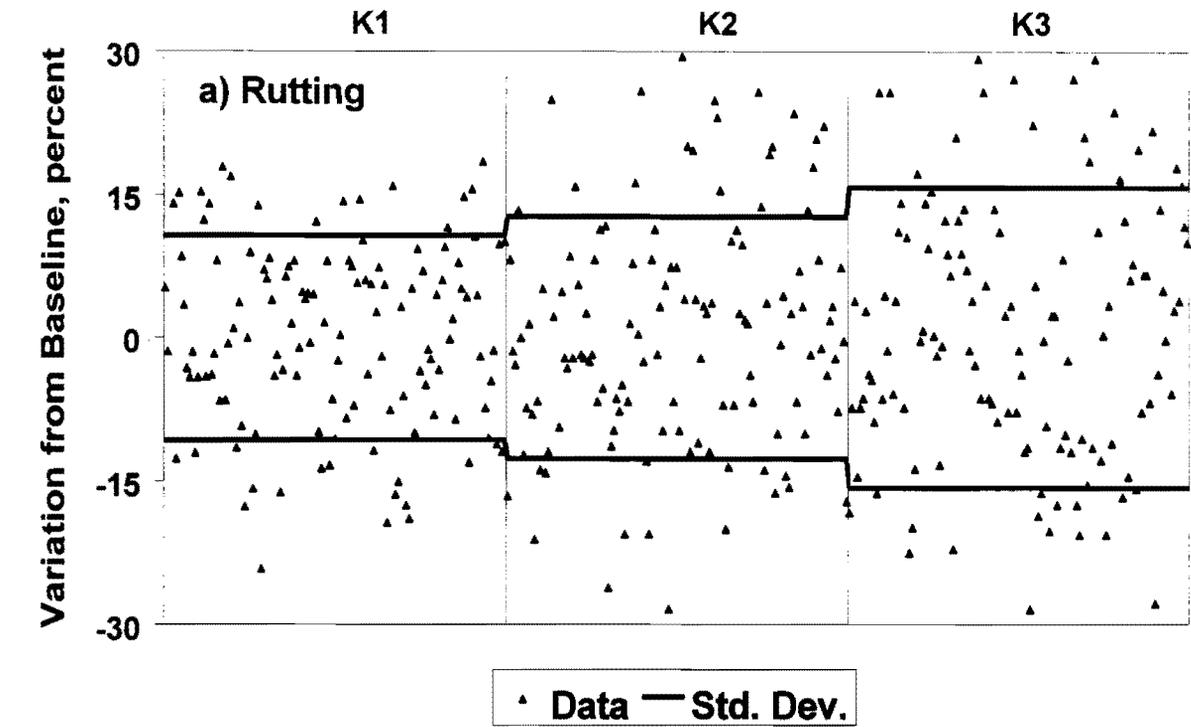
Figure 28 - Sensitivity of Different Pavement Parameters to Remaining Life of a Typical Pavement with Thin AC and Thick Base (Only Base is Considered Nonlinear)

The results from the four different pavements are summarized in Table 11. When only the base is considered nonlinear (Table 11a), the fatigue cracking criterion is more sensitive to the nonlinear behavior of base layers than the rutting criteria.

Similar exercise was carried out but this time both the base and subgrade were considered nonlinear. Typical results for the same pavement used in Figure 28 are shown in Figure 29. The value of k_1 for subgrade was selected as seismic modulus. The values of k_2 and k_3 of the subgrade were assumed to be equal to 0.35 and -0.3, respectively.

Table 11 - Significance of Pavement Parameters in Remaining Life of Pavement

a) when only base layer is considered as nonlinear

Parameter	Relative Significance							
	Rutting				Fatigue			
	Thin AC		Thick AC		Thin AC		Thick AC	
	Thin Base	Thick Base	Thin Base	Thick Base	Thin Base	Thick Base	Thin Base	Thick Base
k_1	M	M	I	M	V	V	S	S
k_2	M	M	I	M	V	V	S	S
k_3	M	M	I	M	V	V	V	V

b) when base and subgrade layers are considered as nonlinear

Layer	Parameter	Relative Significance							
		Rutting				Fatigue			
		Thin AC		Thick AC		Thin AC		Thick AC	
		Thin Base	Thick Base	Thin Base	Thick Base	Thin Base	Thick Base	Thin Base	Thick Base
Base	k_1	M	M	M	M	V	V	S	S
	k_2	M	M	M	M	V	V	S	S
	k_3	M	S	M	M	V	V	S	V
Subgrade	k_1	S	S	S	S	M	I	M	I
	k_2	I	I	I	I	I	I	I	I
	k_3	M	M	M	M	M	I	M	I

I = Insignificant; M = Moderately Significant; S = Significant; V = Very Significant
For the definition of the significance levels see Table 2

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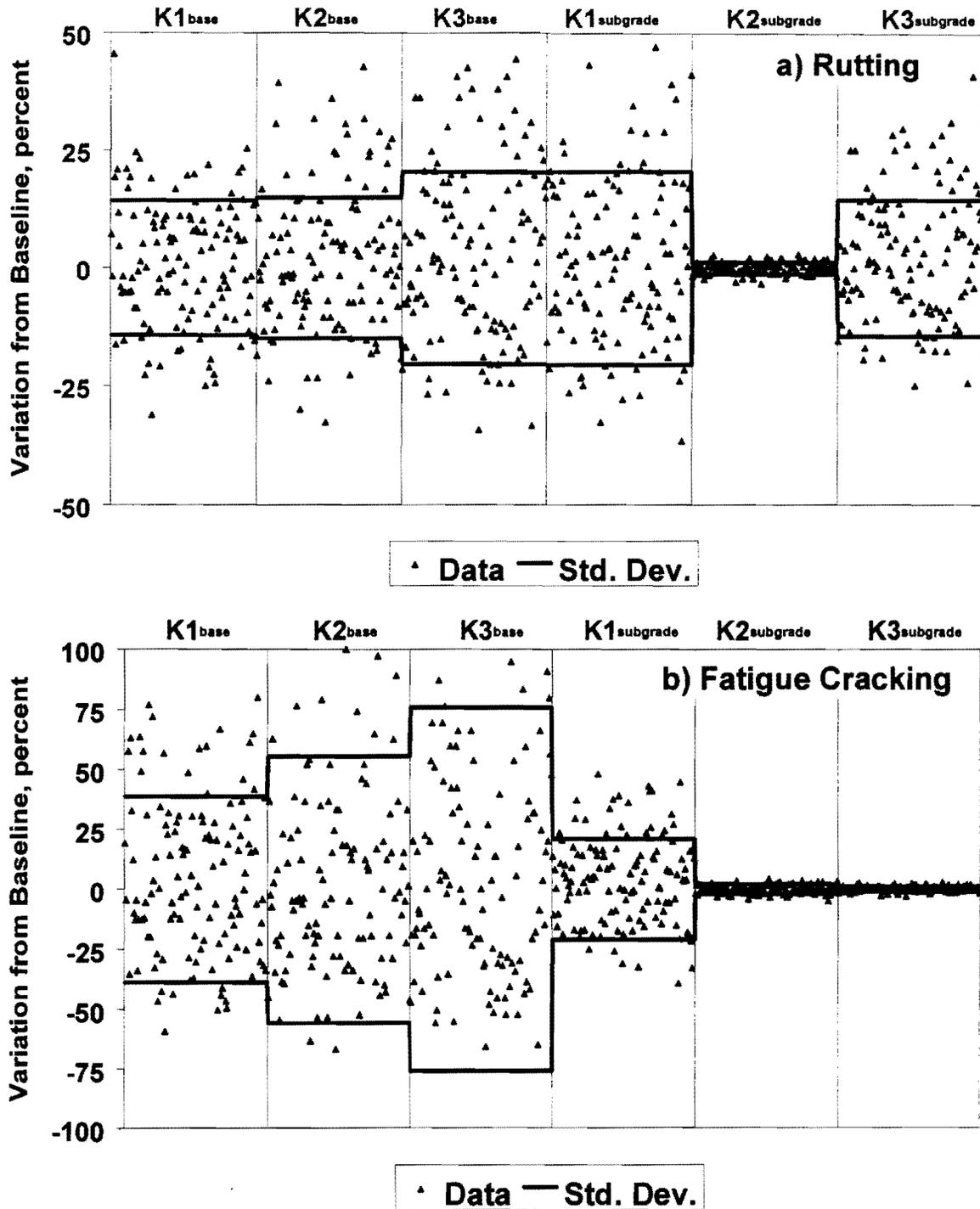
Figure 29 - Sensitivity of Different Pavement Parameters to Remaining Life of a Typical Pavement with Thin AC and Thick Base (Both Base and Subgrade are Considered Nonlinear)

The significance of different parameters on the remaining life is tabulated in Table 11b. For rutting, typically parameters k_1 of the subgrade is the most significant parameter. The three parameters of the base layer and parameter k_3 of subgrade should also be carefully considered. Parameter k_2 is of small significance, perhaps because of small changes in the confining pressure in the subgrade.

For fatigue cracking criteria, the importance of different parameters is somewhat different. As reflected in Table 11b, all three parameters of the base are very important. As the AC layer becomes thinner, the base parameters become even more significant. For thin bases, parameters k_1 and k_3 of the subgrade should also be seriously considered; whereas for thick bases, the impact of the subgrade is small.

This study confirms that the nonlinear behavior of bases and subgrades should be considered. In all these studies the AC layer has been considered as linear. In the remaining six months of this project, the impact of viscoelastic and nonlinear behavior of that layer will be considered and reported.

9.3 Predicting Response of FWD Using Seismic Modulus with Nonlinear Algorithm

A comprehensive case study that resembles the one shown here will be carried out shortly. However, existing data were used to demonstrate ways that the seismic moduli can be used in conjunction with laboratory tests to determine the design modulus.

Two sites, about 1 Km apart were tested on FM 2001 (Nazarian et al, 1987). The site nominally consisted of about 25 mm of AC, over 250 mm of granular base over a clayey subgrade. The variation in modulus with depth from the SASW tests at the two sites are shown in Figure 30. Site. 2 is clearly "softer" than Site 1.

Each of the modulus profiles shown in Figure 30 was simplified for this study to a five-layer system, as reported in Table 12. To calculate the theoretical deflections, WESLEA was modified so that it can perform equivalent-linear analysis. The algorithm added to the WESLEA to modify it to an equivalent linear code was described in Section 6. To avoid confusion, the traditional version of the WESLEA will be called WESLEA_STANDARD, and the modified version WESLEA_EQVLIN.

Table 12 - Variation in Modulus with Depth at Two Sites Used in Case Study

Layer No.	Material	Site 1		Site 2	
		Thickness, mm	Modulus, MPa	Thickness, mm	Modulus, MPa
1	AC	55	4,050	55	3,400
2	Base	125	950	150	540
3		100	310	75	190
4	Subgrade	250	140	250	110
5		Infinity	140	Infinity	110

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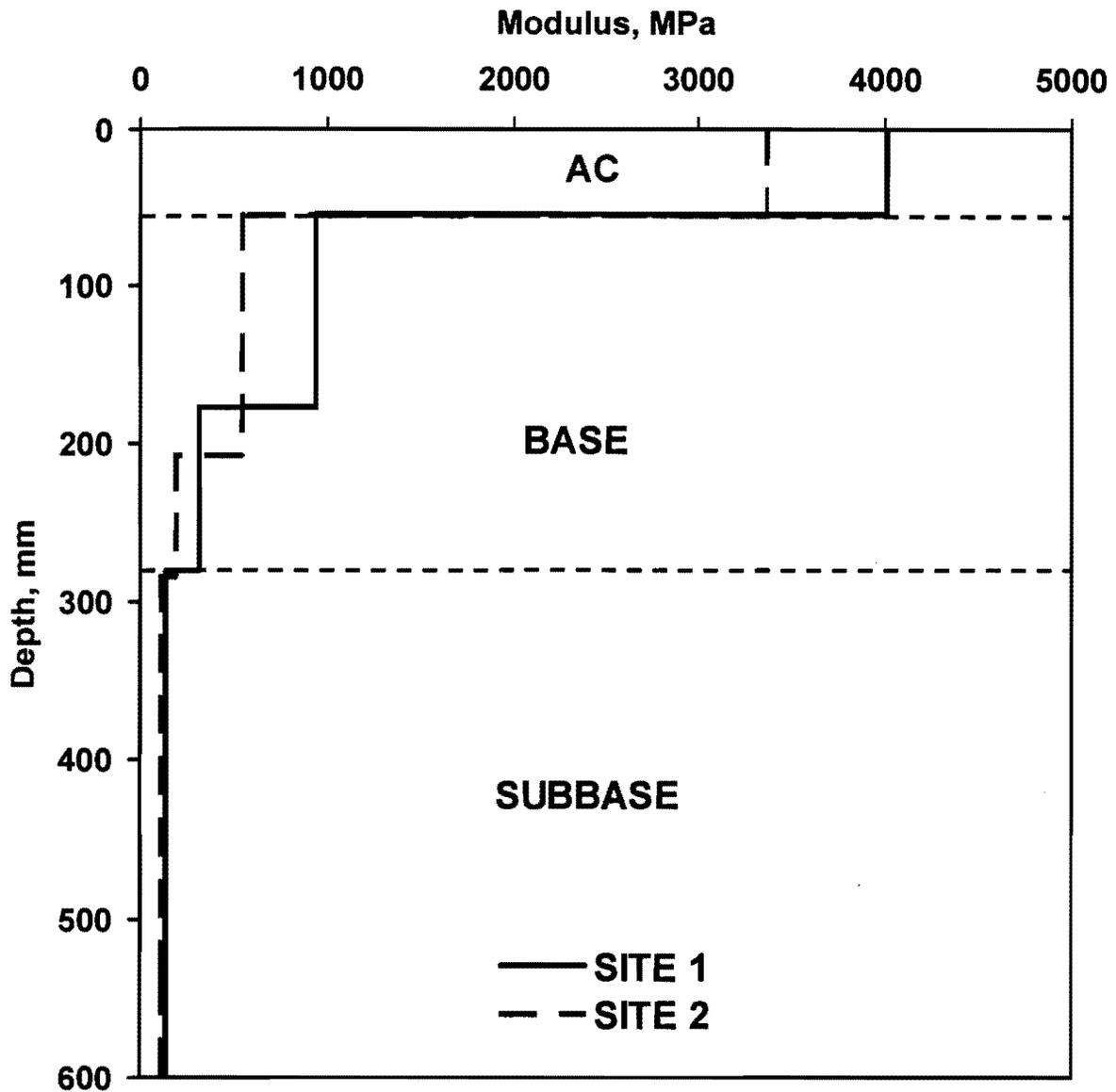
Figure 30 - Variation in Modulus from SASW Test with Depth Used in Case Study

The modification of the program was done for the sake of practicality and ease of use by TxDOT personnel. However, these problems are currently being solved using a finite element code.

The deflection basins measured at a nominal load of about 40 KN are included in Table 13. Again, Site 2 yielded higher deflections suggesting a "softer" site as compared with Site 1. Also shown in Table 13 are the theoretical deflection basins measured by entering seismic moduli in Program WESLEA_STANDARD. For Site 1, which is stiffer of the two sites, the differences between the

Table 13 - Comparison of Deflection Basins Obtained from Different Strategies

a) Site 1

Sensor Spacing, m	Deflection, micron						
	Measured	Linear WESLEA		Nonlinear WESLEA			
				Assumed k_2 and k_3		Optimized k_2 and k_3	
		Theoretical	Percent Difference	Theoretical	Percent Difference	Theoretical	Percent Difference
0	338	216	35.9	303	10.3	335	1.0
0.3	163	127	21.7	148	9.2	153	5.6
0.6	81	74	8.4	76	5.9	77	5.6
0.9	48	49	1.1	47	3.2	47	3.2
1.2	38	35	7.3	34	12.0	34	12
1.5	28	27	1.8	26	5.5	27	5.5

b) Site 2

Sensor Spacing, m	Deflection, micron						
	Measured	Linear WESLEA		Nonlinear WESLEA			
				Assumed k_2 and k_3		Optimized k_2 and k_3	
		Theoretical	Percent Difference	Theoretical	Percent Difference	Theoretical	Percent Difference
0	411	288	29.9	392	4.7	430	4.4
0.3	201	163	18.9	188	6.1	194	3.3
0.6	86	94	9.4	96	11.5	96	11.5
0.9	53	62	16.2	60	11.9	60	11.4
1.2	38	45	18.0	43	12.7	43	12.7
1.5	30	35	15.0	34	11.7	34	10.0

Percent Difference = abs(Measured Deflection - Theoretical Deflection) / Measured Deflection

measured and theoretical deflections from linear WESLEA vary from 2 to 36 percent; whereas for Site 2, the differences are between 10 percent and 30 percent. Several observations can be made

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from these comparisons. First, seismic moduli are higher than anticipated for these two sites since they provide deflections that are smaller than the measured ones. Second, the differences in deflections are much greater closer to the loading pad than away from the load. This indicates that, at least partially, the differences between the measured and theoretical deflection basins are due to load-induced nonlinearity of the material. Third, for the soft site (Site 2), the differences between the measured and theoretical deflections are greater than the stiff site (Site 1). This indicates that similar to the previous case study, when load-induced nonlinearity is increased, the lateral extent of the nonlinear zone expands.

Since laboratory data are not available, two strategies can be followed to incorporate the seismic data in the design. These two strategies are:

- ▸ Strategy 1.   Using Equation 19, one can use the seismic moduli and estimate appropriate values for k_2 and k_3 based on historical data to model the nonlinearity of the base and subgrade. This method is attractive since no laboratory tests are involved. If proven feasible, each district can perform tests on different base, subbase and subgrade once and use those values from then on.

- ▸ Strategy 2.   The seismic moduli and the FWD deflections are combined during the backcalculation process. As in Strategy 1, the seismic moduli are used as before in Equation 19. However, instead of estimating parameters k2 and k3, they are determined using a computer program that can provide surface deflection based on the constitutive model presented in Equation 19 (e.g., WESLEA_EQVLIN, or ABAQUS). The k_2 and k_3 of the base and subgrade are determined by minimizing the errors between the calculated and measured deflections. The obvious disadvantage of this method over standard deflection backcalculation method is that two parameters (k_2 and k_3) have to be determined for each layer, instead of modulus. However, since the values of k_2 and k_3 fall in a narrow range for granular and cohesive materials, we feel that the backcalculation process will be more robust with an added advantage that the nonlinear parameters of each layer are determined. This method is similar to Horhota's method.

One reasonable question is why not implement one of the two strategies with the FWD deflections. Although the model is conceptually applicable to FWD tests, one major practical problem exists. Referring to Equation 15, all three k values are unknown when FWD tests are performed. Therefore, for each layer three parameters (instead of two) have to be backcalculated. As for seismic moduli, parameters k_2 and k_3 are relatively well-constrained. However, parameter k_1 is highly variable and suffers from the same level of nonuniqueness experienced when regular backcalculation is carried out. Therefore, if the first strategy is used (i.e., k_2 and k_3 are estimated) one has to be careful about the backcalculation at a level similar to those usually experienced with standard backcalculation.
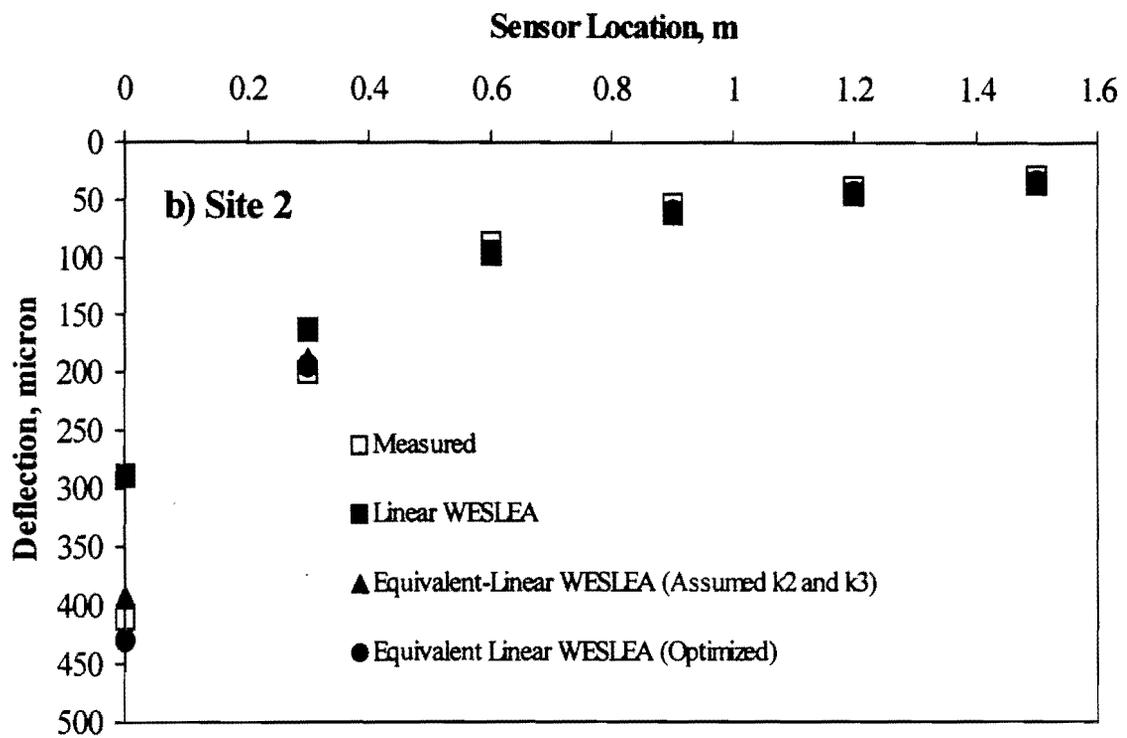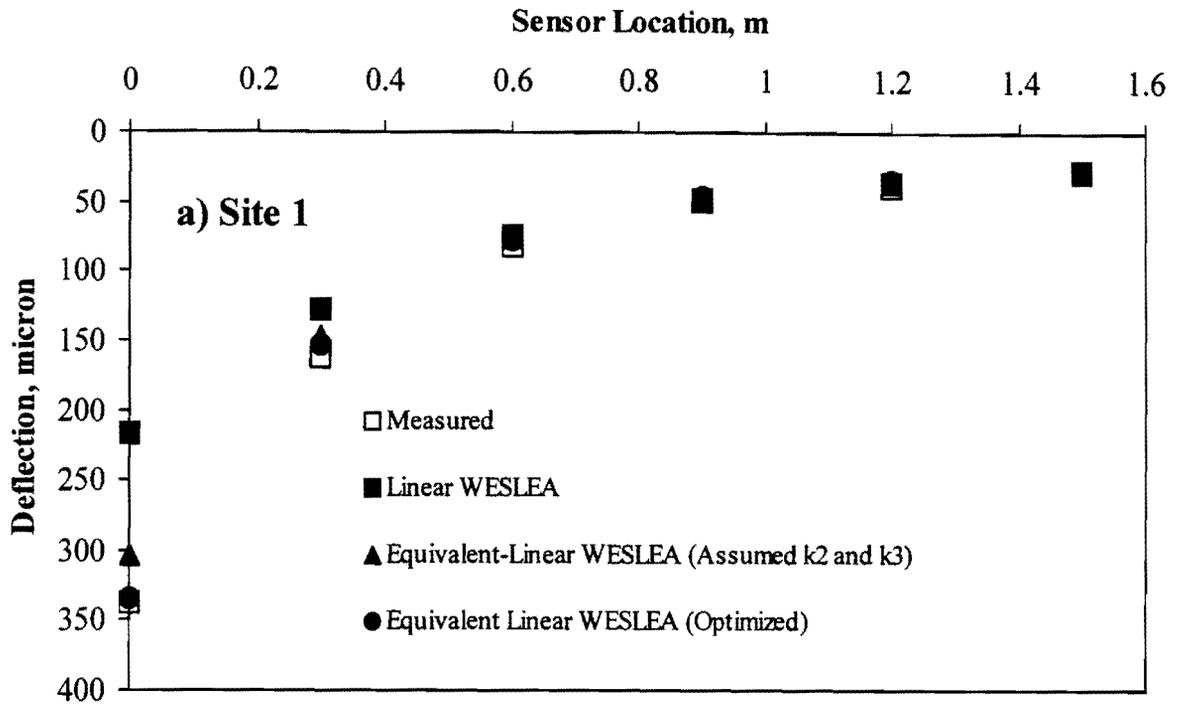
Figure 31 -Comparison of Deflection Basins Obtained from Different Strategies

The second scenario is usually out of the question because it would be highly nonunique to backcalculate three parameters per layer when only seven deflections are available.

The results from the first strategy are summarized in Figure 31 and Table 13 for both sites. In one case, the deflection basins are calculated by assuming "reasonable" values for k_2 and k_3. These values are selected from the literature search summarized in Section 5. From Table 10, values of k_2 equal to 0.37 and 0.15 were assumed for the base and subgrade, respectively. A value of k_3 equal to -0.4 was used for both base and subgrade. As reflected in Table 13, the average errors between measured and calculated deflections reduced substantially. At both sites, the maximum error reduced from about 36 percent to about 12 percent. This shows the promise of the method.

In the second strategy, the values of k_2 and k_3 were assumed to be variable within the ranges of reasonable values defined in Section 5, and with the values assumed in the first scenario as the seed values. The errors between the theoretical and measured deflections were minimized. The best results were obtained for Site 1 when values of k_2 were 0.33 and 0.15 for the base and subgrade, respectively. The value of k_3 was equal to -0.4 for both layers. For Site 2, k3 was again equal to -0.4, whereas k2 values were equal to 0.20 and 0.37 for the base and the subgrade, respectively. Here the average errors were again decreased for both sites. However, most of the reduction in error was associated with the sensors closer to the load (i.e., those that experienced the most load-induced nonlinearity).

Also to demonstrate the value of using seismic modulus (and Equation 19), the variation in modulus within a cross-section of site 1 under a FWD impact is shown in Figure 32. The values of k parameters determined in the second strategy were used. The imparted load is 40 KN. One can clearly see that the load-induced nonlinearity results in variation in modulus laterally and with depth. This may make the backcalculation of equivalent moduli difficult; whereas seismic moduli do not suffer from this problem.

This case study clearly shows that improving the determination of moduli and the modeling the nonlinear behavior of the pavement layers may be feasible. However, this is only an example of potential uses of this methodology. The sites selected were specially suited for this study since the pavement layers were thin and the subgrade was soft. A large number of sites with a variety of base and subgrade conditions should be considered before a definite conclusion can be drawn.
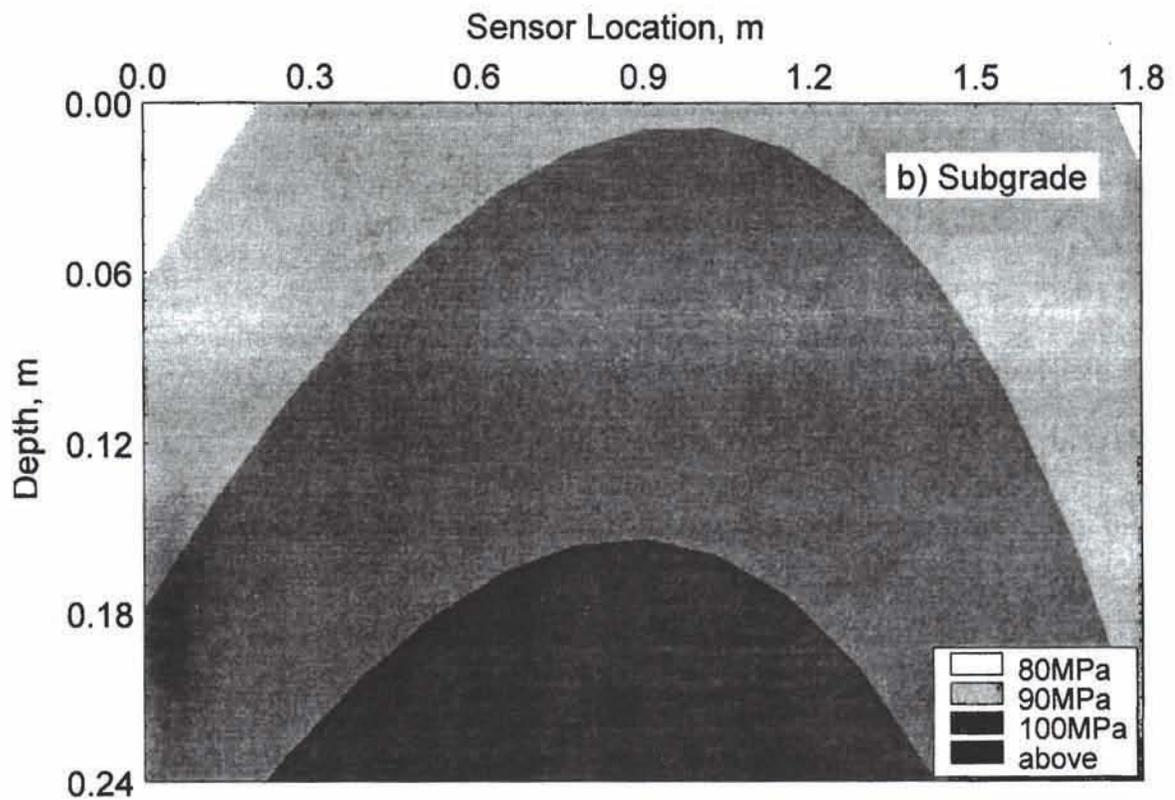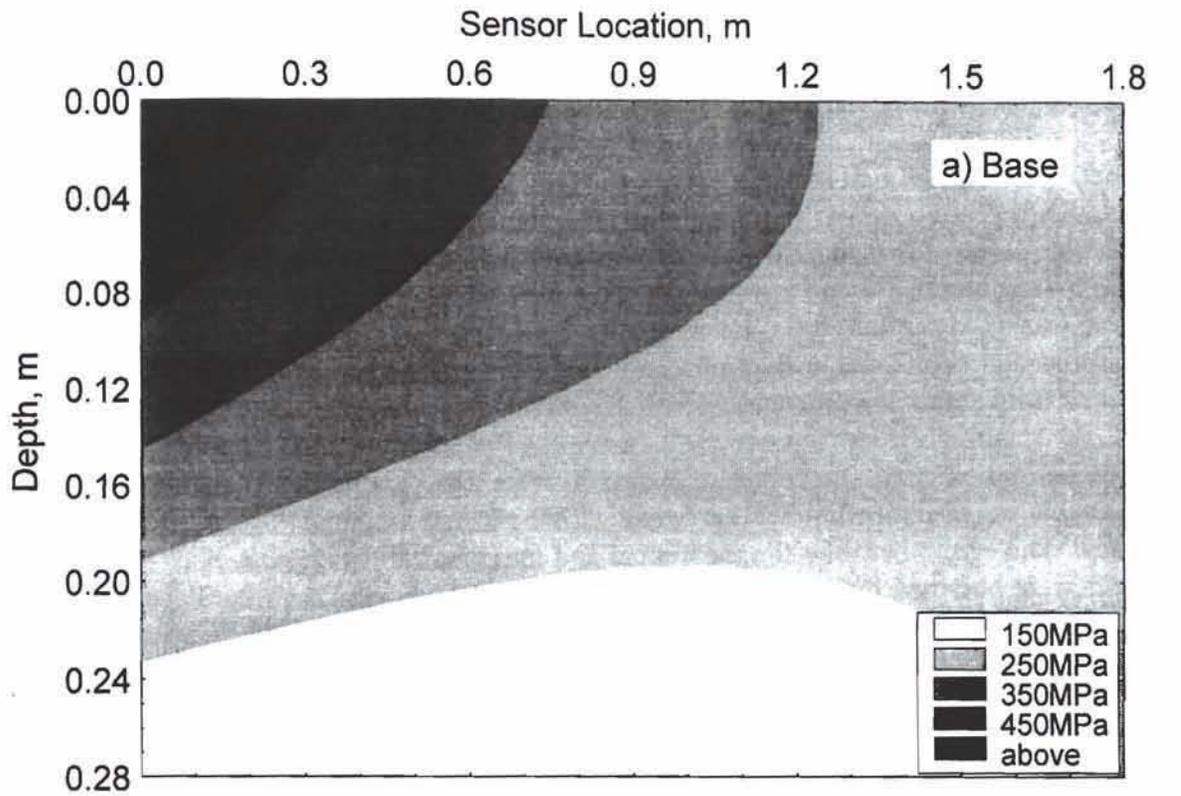
Figure 32 - Variation in Modulus under Site 1 Due to a FWD Impact of 40 KN

10. Closure

In this report a comprehensive literature search that covers different aspects of pavement design with not just seismic moduli is presented. Based on the experience of the researchers and practitioners in nondestructive testing, pavement design, geotechnical engineering, seismology and earthquake engineering, a comprehensive conceptual design methodology has also been suggested. The comprehensive methodology has been analyzed and simplified so that it can be practical enough for use by TxDOT. The research issues and unanswered questions are comprehensively detailed in Section 8. Several case studies are included to show the significance of the concept and the feasibility of it.

Based on this study, the use of seismic moduli in a mechanistic pavement design methodology seems reasonable and feasible. We feel that to implement a fully-mechanistic design procedure, or to develop performance-based specifications, the use of seismic moduli may be a better long-term alternative. This statement is substantiated by the fact that the state of stress and strain are much better understood and defined under seismic tests. Adjusting seismic moduli for the state of stress that the pavement is experiencing under the actual wheel load may be simpler than fully understanding the stress regimes developed during the FWD tests.

Several issues have to be resolved before seismic moduli can be used for design. In our opinion, the most important issue to be addressed is to define a balance between the sophistication in the field tests, with the number and the nature of laboratory tests to be performed, with the design algorithm to use.

Specifically, the following items have to be considered:

1. The simplest computer algorithm that can provide the capability of determining nonlinear properties of different layers should be identified. The choices are among equivalent-linear layered programs or equivalent linear or nonlinear finite element programs.

2. The most appropriate model to characterize the base and subgrade should be established. Some choices are a power model or a Ramberg-Osgood type model.

3. The possibility and the negative consequences of cataloging or estimating some of the parameters used in the material models should be explored.

4. The validity of the entire system, consisting of the model used for estimating stresses and strains within a pavement section and the nonlinear models used to determine moduli should be determined.

5. The proposed system should be fine-tuned until an acceptable compromise between the accuracy of the results, reasonableness of the laboratory and field tests, and ease of use of computer models are struck.

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