Laboratory Characterization of Materials & Data Management for Ohio – SHRP Projects (U.S. 23)

Final Report

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LABORATORY CHARACTERIZATION OF MATERIALS & DATA MANAGEMENT FOR OHIO-SHRP PROJECTS (U.S. 23)

FINAL REPORT

FOR

OHIO DEPARTMENT OF TRANSPORTATION and
FEDERAL HIGHWAY ADMINISTRATION

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16. Abstract

About a decade ago, the Federal Highway Administration (FHWA) set up a national study called the Long-Term Pavement Performance (LTPP) under the Stretegic Highway Research Program (SHRP) to extend pavement life through investigation of different pavement designs under various loading, environmental, subgrade soil, and maintenance conditions. The study involved many different materials and stressed the importance of collecting field and laboratory test data on their mechanistic properties.

In the current study, mechanistic properties of the pavement materials involved in the Ohio-SHRP project were measured according to the SHRP Protocols. The test program encompassed a wide array of materials and their properties, ranging from basic index properties of the subgrade soils to resilient modulus of soils and asphalt concrete to static modulus of Portland cement concrete and creep modulus of asphalt concrete. Any trends observed in the test results were pointed out to enhance our understanding of how each pavement material behaves. In some cases, previously published empirical relationships correlating basic and advanced material properties were reevaluated in light of the latest rest results.

A need for integrating a large volume of data that existed for the Ohio-SHRP project was recognized even piror to the initiation of the current study. As a result, a computer-based database was developed, packaged into a CD-ROM disk, and attached to this report. This user-friendly database allows a fast and easy access to all the mechanistic properties presented in this report as well as general information related to the Ohio-SHRP project.

17. Key Words
Mechanistic Properties, Pavement Materials, Laboratory Tests, Database, Ohio Test Road, SHRP, LTPP

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CHAPTER 1:
INTRODUCTION

1.1 Background

Highway design engineers in the U.S. have been relying on the 1986-1993 American Association of State Highway and Transportation Officials (AASHTO) Design Guide, which is based on the many empirical elements obtained in the 40-year old AASHO Road Test. Today, traffic volumes, traffic loads, and expectations for better pavement performance have outgrown the accuracy of the empirical design method. The performance and life of highway pavements have received increased concern across the U.S., since the maintenance and reconstruction of pavement systems cost the state and federal governments billions of dollars each year. Due to the great expense and effort often associated with roadway maintenance, many states are now behind schedule for highway repair. The inability to characterize material properties and their effect on pavement performance is believed to be a contributing factor to the pavement performance problems that exist.

About a decade ago, the Federal Highway Administration (FHWA) established a national study called the Long-Term Pavement Performance (LTPP) under the Strategic Highway Research Program (SHRP) to respond to the growing public concerns on the national level. The goal of the LTPP study was "to increase pavement life by investigation of various designs of pavement structures and rehabilitated pavement structures, using different materials and under different loads, environments, subgrade

soil, and maintenance practices" (FHWA, 1993a). To achieve this goal, the LTPP study was to establish a National Pavement Data Base (NPDB) that would contain inventory information, traffic data, climatological data, field monitored/test data, field sampling data, laboratory test data, and maintenance data for each pavement section. Therefore, major activities of the LTPP study included collection of inventory data, field test data, and laboratory test data on a large number of pavement test sections.

In parallel to the LTPP study, efforts have been continued by various research organizations to try to develop a new pavement design method, which is based more on principles of mechanics and less on empirical elements. The new method is generally labeled as the "mechanistic-empirical (M-E)" procedure. This is because the method, although based on sound engineering principles, still requires special transfer functions to translate predicted strains and stresses to the most likely pavement distresses. Effective implementation of the M-E procedure depends on complete and accurate input of the engineering properties of the pavement layer materials involved.

The Ohio Department of Transportation (ODOT) has funded many highway research projects in recent years, with the largest being the U.S. Rt. 23 project in Delaware County, Ohio. This highway project is also part of the Ohio Strategic Highway Research Program (Ohio-SHRP) Test Road. Ohio University coordinated the multi-university team assembled for this project and was responsible for most of the field instrumentations, field testing, and monitoring of the pavement sections. The Ohio-SHRP Test Road provides a great opportunity to implement and evaluate the M-E procedure, provided that the actual mechanistic properties of the pavement materials involved in the

project are measured according to the SHRP test protocols. The results from the application of the M-E procedure can then be compared to the actual sensor readings and pavement distress observations made at the Ohio-SHRP Test Road site.

In the last two years, personnel at Ohio University have conducted over one hundred experiments to characterize the properties of concrete for the SPS-2 and SPS-8 sections of the Ohio-SHRP project. These laboratory tasks were undertaken in consultation with the Ohio Department of Transportation (ODOT) personnel. The concrete testing conducted by Ohio University was part of the SHRP requirements. Although under the Long Term Pavement Performance Program (LTPP) it was required that the federally funded agency provide limited information on the characteristics of asphalt and subgrade, there is a need for more detailed characterization of all the materials that were used in the U.S. 23 project.

The mechanical properties of each component layer are an integral part of any design procedure. Structural responses of the pavement system due to load and/or environmental factors play a key role in development of a mechanistic design or verification of existing models.

In addition to the above, there is a need to integrate all the data from the Ohio-SHRP project. In a comprehensive pavement research project, one central source of data that contains construction sequence information, climatological data, material property data, etc., must be established.

1.2 Objectives

Objectives of this study are summarized below:

- Determine the mechanical properties of the materials that were used in the Ohio-SHRP (U.S. 23) project.
- Integrate and consolidate all the data for the Ohio-SHRP (U.S. 23) project that could be utilized for implementation in development of calibration of mechanistic design approach by the ODOT engineers and other designers and researchers.

1.3 Outline of Chapters

Chapter 2 describes past laboratory testing of pavement materials. The remaining Sections of this chapter are devoted to a literature review of laboratory tests performed on each different pavement material in the relatively recent past. In the section related to Portland cement concrete (PCC), the focus is more on the current state of knowledge than on research, since the art of concrete testing has well-established PCC properties.

Chapter 3 presents key information on the Ohio-SHRP Test Road project. The chapter begins with a description of the site conditions such as topography, geology, hydrology, etc. The focus then shifts to describe the makeup of the SHRP experiments within the Ohio-SHRP Test Road, such as pavement materials utilized, field instrumentations for seasonal (or environmental) responses and field instrumentations for load responses.

Chapter 4 summarizes currently available standard test protocols that were applied to measure the mechanistic properties of the pavement materials utilized in the Ohio-SHRP project. These include test methods established by the American Society for Testing and Materials (ASTM), AASHTO, and SHRP. Presentation of these test methods is grouped by the material types (e.g. subgrade soil, unbound granular base, stabilize base, concrete, and asphalt concrete).

Chapter 5 constitutes the heart of this report, presenting in detail every mechanistic property measured in the current study. The test results are categorized by the material types. Each section in this chapter is organized to present: (1) information on the test specimens, (2) descriptions of the test equipment utilized, and (3) specifics of the test procedure followed. The test results are then presented using tables and/or figures. Discussions follow the presentation of the test results to point out trends and findings observed in the results. In some cases, previously published empirical relationships among the material properties are also tested in light of the latest test results.

Chapter 6 describes a database developed as a logical next step resulting from the current study. The computer-based database, packaged into a CD-ROM disk, has been created to allow for fast and easy access to the information/data related to the Ohio-SHRP Test Road project. All the mechanistic laboratory test results presented in this report are accessible within the database. The installation procedure for the database is described in a step by step manner in this chapter. So, this chapter can serve as a users' manual for the database. The CD-ROM disk is located on the inside face of the back cover of the report.

Finally, Chapter 7 summarizes the laboratory test results and offers conclusions. A few helpful implementation plans are also suggested in this chapter.

CHAPTER 2:
LITERATURE REVIEW

2.1 Introduction

Testing and properties for materials such as concrete and soils, which are regarded as more traditional civil engineering materials, are well established. However, less information is available for relatively new materials, such as asphalt concrete and stabilized base materials. In the future, this information will become more readily available due to the continued advancement in computational and sensor technologies, which will allow for more sophisticated and realistic test methods for flexible or rigid pavement structures.

2.2 Laboratory Testing of Subgrade Soils

There have been a number of studies conducted in the past to understand the engineering behaviors of fine-grained subgrade soils under repeated load cycles. The most common approach utilized in these studies was to characterize the dynamic behavior of subgrade soils through a property called "resilient modulus". This is because after a series of repeated load applications the soil's deformation behavior becomes predictable, consisting of small permanent strain and larger recoverable elastic (or resilient) deformation. Resilient modulus (M_R) is defined simply as:

$$M_R = \sigma_d / \varepsilon_R \tag{2.1}$$

where σ_d = repeated deviatoric stress; and

ε_R = recoverable (or elastic) axial strain.

Thompson and Robnett (1976) performed a series of resilient modulus tests on subgrade soils in Illinois. They observed that the degree of saturation had a significant influence on the magnitude of resilient modulus at both 95 and 100% relative compaction.

Majidzadeh and Bayomy (1978) presented the correlation between soil properties and resilient modulus for nine silty and sandy soils from eight counties in Ohio. Each soil sample was prepared in the laboratory at various compaction moisture contents and dry unit weights. The samples were tested under a uniaxial dynamic loading environment, which involved a range of dynamic stress intensities to simulate the stress variations in the field, and to develop a correlation between dynamic stress levels and soil dynamic moduli. It was noted that as the deviator stress increased up to a certain level, the resilient modulus decreased rapidly. They concluded that resilient properties depended upon soil types, relative compaction, and moisture content.

Johnson (1986) measured resilient modulus of subgrade soils in the eastern Tennessee area. According to his study, the resilient modulus showed sensitivity to changes in moisture content, decreasing from about 97 to 62 MPa (14 to 9 ksi) when the degree of saturation was increased from 80% to near saturation.

Research performed by Figueroa et al. (1994) examined the seasonal variation of subgrade strength parameters. The test was conducted by collecting soil samples, which were classified into three fine-grained soil types (A-4, A-6, and A-7), from nine counties in Ohio. A bilinear model established by Thompson and Robnett (1976) was used to represent the behavior of fine-grained soils subjected to repeated loading. It was found that the higher moisture susceptibility of A-7 soils caused its resilient modulus at the break-point to decrease faster with the degree of saturation than those of A-4 and A-6 soil types. They concluded that resilient modulus of fine-grained soils depended upon the soil type, dry unit weight, and moisture content (or degree of saturation). Temperature had little effect on resilient modulus, as long as it was above the freezing point.

Li and Selig (1994) analyzed eleven sets of resilient modulus test data found in the research literature, and proposed a general method to estimate the resilient modulus of compacted fine-grained soils. They observed that three factors had a significant effect on the magnitude of the resilient modulus: 1) loading conditions (or deviatoric stress); 2) soil type and microstructure; and 3) the soil's physical state (e.g. moisture content, dry unit weight). They further observed that the resilient modulus could vary between 2 ksi and 20 ksi (14 MPa and 140 MPa) for the same soil due to variations in these factors.

Laboratory studies carried out within the last two decades by Barksdale et al. (1993) and by Pezo et al. (1992), on the subgrade and base materials served as a foundation for the standardization of the resilient modulus test procedure into the current SHRP P46 Protocol (1996). These studies combined with those conducted earlier by Tanimoto and Nishi 1970), Townsend and Chisolm. (1976), and Frendlund et al. (1975)

showed that resilient modulus of fine-grained soils could vary more than one-fold under the influences of stress state, moisture content, and dry density. In addition, the confining pressure had a smaller effect on the resilient modulus than the deviatoric stress for fine-grained soils.

In summary, these studies clearly show that the resilient modulus of fine-grained soils depends upon the soil type, loading conditions, and the soils' physical state. However, resilient modulus may also be affected by freeze-thaw cycles. Elliot and Thornton (1988) examined the effect of freeze thaw cycles on the resilient modulus of soil. Four seasonal variations in moisture content during the year on a similar subgrade soil were used. It was found that the resilient modulus decreased by about 50% due to the freeze-thaw action after 1 cycle. A major concern for highway design engineers is how much the subgrade stiffness or resilient modulus fluctuates seasonally as the subgrade soil undergoes numerous drying/wetting cycles and freeze-thaw cycles in each year. Controlled laboratory studies can certainly provide some insight into this fundamental question, and suitable in-situ testing methods, such as the Falling Weight Deflectometer (FWD), can provide actual data to answer this question.

2.3 Laboratory Testing of Unbound Base Materials

Traditionally, the study of engineering behavior of unbound granular soils under cyclic loading focused on their liquefaction potentials. The resilient behavior of granular soils has been examined only within the last three decades.

Chen et al. (1994) studied the variability of resilient modulus for six aggregate materials used for highway construction work in Oklahoma. Six resilient modulus tests, each under identical conditions for two aggregate types, and three tests each for four aggregate types, were conducted as per the AASHTO T292-91I test procedure. Two of the aggregate types were chosen for resilient modulus testing using AASHTO T292-92I. These values were then compared to those obtained from the AASHTO T292-91I, as well as with those reported by various agencies. These procedures were chosen since the AASHTO T292-91I test procedure starts with a higher confining pressure and deviator dynamic stress and ends with a lower confining pressure and deviator dynamic stress while the other (AASHTO T292-92I) starts in the reverse order. They found that the variability of resilient modulus values depended more on the testing procedure than on the aggregate source.

Nunes and Dawson (1997) observed the behavior of pavement foundation materials under repeated loading. Several types of unbound material were used, some of which were lightly treated with various binders. The samples were tested using the repeated load triaxial apparatus. They found that the stress/strain behavior of lightly treated material is similar to that of unbound materials. However, the non-linearity and stress dependency were reduced at higher levels of treatments. It was also noted that treatment improved resilient moduli, yet in some cases the enhancement only brought the material to the level expected for regular unbound materials.

Lekarp and Dawson (2000a) reviewed the current state of knowledge on resilient properties of granular materials. They found that factors such as stress level, amount of

fines, maximum grain size, gradation, density, aggregate type, and moisture content affected the resilient behavior of granular materials. Evidence indicates that the resilient modulus depends mostly upon the confining pressure and the sum of the principal stress, and slightly upon deviator stress. Conversely, Poisson's ratio increased directly with the deviator stress and inversely with the confining pressure. Both of these parameters were affected by the moisture content, which reduced the resilient modulus as the moisture content increased particularly at high degrees of saturation.

Lekarp et al. (2000b) also summarized several research papers regarding the permanent strain development of granular material. Factors such as stress level, principal stress reorientation, number of load applications, moisture content, stress history, density, fines content, gradation, and aggregate type were believed to contribute to the permanent strain development of the granular material. Mainly, permanent strain was related directly to the deviator stress and inversely to the confining pressure. Permanent strain was also affected by moisture content especially at high levels of saturation. At these levels, deformation resistance within the material reduced quite rapidly, probably with a positive pore water pressure being generated.

Uzan (1999) presented a procedure utilized for characterizing the granular material properties for the M-E procedure, which included repetitive testing under different confining pressures and axial stresses. Several specimens, with dimensions of 6 inches (152 mm) in diameter by 11.3 inches (287 mm) in height, were molded. Each sample was loaded at a different confining pressure over 10,000 to 100,000 load repetitions. The test results at different confining pressures and axial stresses showed that

the material response was non-linear, especially after a large number of load repetitions; therefore, the material may be considered elastic. They also noted that the material exhibited a lag of response behavior.

2.4 Laboratory Testing of Stabilized Base Materials

The use of stabilized base materials in pavement construction is a relatively new practice. However, the review of literature related to laboratory testing of stabilized base initially identified a few older studies. Terrel and Awad (1972) researched the behavior of the asphalt-treated base (ATB) materials under a range of test conditions by applying repeated load triaxial stress conditions. A triaxial test system was utilized for the resilient modulus tests in order to provide a wide range of stress-temperature conditions equivalent to those found in the field. Several specimens fabricated in the laboratory with dimensions of 4 inches (101.6 mm) in diameter by 8 inches (203.2 mm) in height, were subjected to a full series of stresses, including sinusoidal, repeated load, and creep loading at various frequencies to determine resilient modulus. Both tests were performed at the same stress levels and temperature. The temperature ranged from 25°F to 90°F (–4°C to 32°C). Several conclusions were drawn from the study:

- The stress-strain relationship of asphalt-treated base material was linear within the range of axial stresses and temperatures.
- The stiffness of the asphalt-treated base material depended on the shape of the aggregate material rather than the confining pressure at low temperature. At high temperature, ATB stiffness was dependent on temperature and asphalt content.

- Conversely, if the crushed aggregate is utilized, the interlocking of the particles will influence the behavior of the material by changing its shear behavior where the effects of temperature and asphalt contents on the material non-linearity may be slightly altered.

Resilient modulus tests on the asphalt-treated base were conducted by Terrel and Monismith (1968) using SM-K (CMS-2S) asphalt emulsion-treated base and MC-800 liquid asphalt-treated granular base course materials. Several specimens with dimensions of 4 inches (101.6 mm) in diameter by 8 inches (203.2 mm) in height were prepared in the laboratory. The specimens were tested by applying repeated loads with an approximate square-wave shape load having a 0.1 second duration and frequency of 20 cycles per minute (cpm). A series of sustained confining pressures from 50 psi to 40 psi (0.35 MPa to 0.28 MPa) and repeated deviator stresses from 5 psi to 30 psi (0.03 MPa to 0.2 MPa) were applied at 68°F (20°C). It was noted that the modulus of both the SM-K and MC-800 depended on the deviator stress and the amount of curing or age, rather than on the confining pressure. As the curing time increased, the stress dependence of the resilient modulus on confining pressure for the SM-K treated material decreased.

Several studies had indicated that by adding as little as 1% cement to asphalt-emulsion treated material a rapid gain in resilient modulus could be observed. Smith and Nair et al. (1972) also performed a series of tests to establish the resilient modulus and Poisson's ratio of asphalt-emulsion treated material. This test was conducted using a sandy soil treated with 6% CMS-2S (SM-K) asphalt emulsion. Cylindrical specimens

with dimensions of 4 inches (101.6 mm) in diameter by 8 inches (203.2 mm) in height, were tested by applying the same square-wave loading described earlier, at temperatures of 40°F (4.4°C), 70°F (21.1°C) and 100°F (37.8°C). Micro-Measurement strain gages, with a 2 inch (50.8 mm) active gage length, were bonded to the specimens using a special epoxy. It was concluded that the elastic properties of asphalt-emulsion treated base were dependent upon temperature and time, and therefore viseoelastic. Temperature and time effects can be considered through the selection of appropriate constitutive values. They observed that the viseoelastic constitutive relationship modeled ATB properties better than the elastic relationship.

Terrel (1967) studied the behavior of asphalt-emulsion treated materials by conducting resilient modulus tests using repeated load triaxial compression. Aggregate was treated in the laboratory with three types of asphalt: 1) asphalt emulsion (SM-K); 2) liquid or cutback asphalt (MC-800); and 3) asphalt cement (85-100-penetration grade). The results showed that the resilient modulus depended on both the confining pressure and deviator stress for treated aggregate with asphalt emulsion. For specimens containing asphalt emulsions or liquid asphalt, the resilient modulus increased as confining pressure increased during their uncured state or early stages of curing. Resilient modulus of uncured asphalt-treated materials was found to be slightly dependent on temperature.

Nair et al. (1972) researched the behavior of cement treated base coarse materials under repeated loads, using the same equipment as previously discussed. Both axial and radial strains were measured with the same type of strain gages used for all characterizations. Cement treated base Class A and Class B were used for this test. Type I

Portland cement was used to produce Class A (5.5% cement by weight of dry aggregate) and Class B (3.5% cement). It was concluded that a cement treated base could be treated as a linear elastic material.

Research undertaken by Lotfi and Witczak (1985) investigated the resilient modulus of cement stabilized base material types used by the Maryland State Highway Administration (MSHA). Typical cement-stabilized dense graded aggregate used by MHSA, which included limestone (LS) and Maryland State medium sand were used. Forty-eight specimens with dimensions of a 4 inches (101.6 mm) in diameter by 8 inches (203.2 mm) in length were tested using the University of Maryland's MTS Systems. Each sample was tested at five stress levels, three repetition levels, and three frequencies. Based on the analysis of the resilient modulus results, it was concluded that the cement content, the material type, and the dense graded aggregate (DGA) gradation had a major effect on the resilient modulus of cement-stabilized materials. From the test it was also found that the stress state had a minor influence especially at higher cement contents and longer curing periods. The load frequency and number of load repetitions appeared to have the smallest effect on the resilient modulus.

2.5 Laboratory Testing of Concrete

One of the most studied and understood pavement materials is Portland cement concrete (PCC). Civil engineers have been using PCC for many years, and have established standard mixture designs, standard testing methods, and determined relationships among its engineering properties. However, there is still a need for further

research on PCC, as evidenced by articles in publications of new technical materials on concrete technology such as ACI journals and ASTM STP publications in each year.

A book written by Mindess and Young (1981) is a very good source of information pertaining to PCC. The following is a summary of some useful information, taken from their book, related to engineering properties, mainly compressive strength, tensile strength, and elastic modulus of concrete.

Compressive strength is by far the most important engineering property of concrete, as its value corresponds to the overall quality of concrete. The compressive strength of concrete is dependent largely on porosity, which can be determined by the water/cement (w/c) ratio. Compressive strength decreases as this ratio increases; high w/c ratio represents large voids in inappropriately compacted concrete.

The tensile strength of concrete is much lower than the compressive strength, mainly due to the fact that stress cracks are created under tensile loads. The typical value of tensile strength of concrete is 400 psi (3 MPa). Failure of concrete will most likely occur from splitting tension at a much lower load than in compression, since concrete is less able to handle tension than compression; thus, permitting an estimate to be made of the tensile strength of the concrete.

The elastic modulus (or Young's modulus) of concrete is related to its compressive strength and density. The dominant factor in determining the elastic modulus of concrete is its porosity, since the elastic modulus decreases markedly as the (w/c) ratio is increased. The elastic modulus can be estimated from an empirical relationship between the modulus and the compressive strength. Therefore, factors that

affect the compressive strength should similarly influence the elastic modulus. It was determined that the strength-modulus relationship could become inconsistent due to the moisture dependency. The strength of saturated concrete is lower than that of dry concrete, while the resilient modulus of saturated concrete is higher than that of dry concrete.

One of the concrete properties that can be input into the M-E design is the mean flexural strength (or modulus of rupture) at 28 days. This concrete property is measured using the third point loading (AASHTO T-96; ASTM-78) rather than the center point loading (ASTM C-293). The third point loading procedure is generally thought to be more representative of conditions that develop in the pavement structure.

The concept of flexural strength is relatively simple and is based on the ability of the concrete to resist bending stress. However, since the test procedure for measuring flexural strength are complex and requires skilled technicians to achieve consistent results, many SHAs are now shifting to use the compressive tests.

The modulus of elasticity (E_c) can also be predicted from strength properties of sufficient accuracy for the proposed design procedure NCHRP 1-26 (University of Illinois at Urbana-Champaign, 1992). The relationship is based on measured strains during compressive strength testing conducted at the strain rate prescribed in the ASTM Test procedure C-469. The concrete properties increase with increasing strain rates. Poisson's ratio has very little effect on the results. The recommended value to be used as the standard input for design is 0.15.

Setunge et al. (1990) studied the static modulus of elasticity and Poisson's ratio of very high strength (2,900 to 12,035 psi or 20 to 83 MPa) concrete. They found that the properties of coarse aggregate played a major role on the static modulus of very high strength concrete. They also found that the static modulus of very high strength concrete was dependent upon the static loading rate as generally established for normal strength concrete. It was also noted that Poisson's ratio increased with an increase in compressive strength.

2.6 Laboratory Testing of Asphalt Concrete

The resilient modulus of asphalt concrete is another important material property input into the M-E design procedure. The basic assumption of the resilient modulus is that most paving materials are not elastic. Yet, the deformation under each repeated small load is almost completely recoverable. While this resilient behavior is taking place, the material also experiences some permanent deformation after each load application. Thus these materials can then be considered to be pseudo-elastic (Huang, 1993).

The resilient modulus test provides a basic relationship between stress and strain or deformation of pavement construction material for use in structural analysis of layered pavement systems. The permanent deformation occurs in the early stage of the repetitive loading, depicted in Figure 2.1. As the number of repetitive loads increases, the plastic strain due to each load decreases; and after a certain number of repetitions the strain is almost completely recoverable.

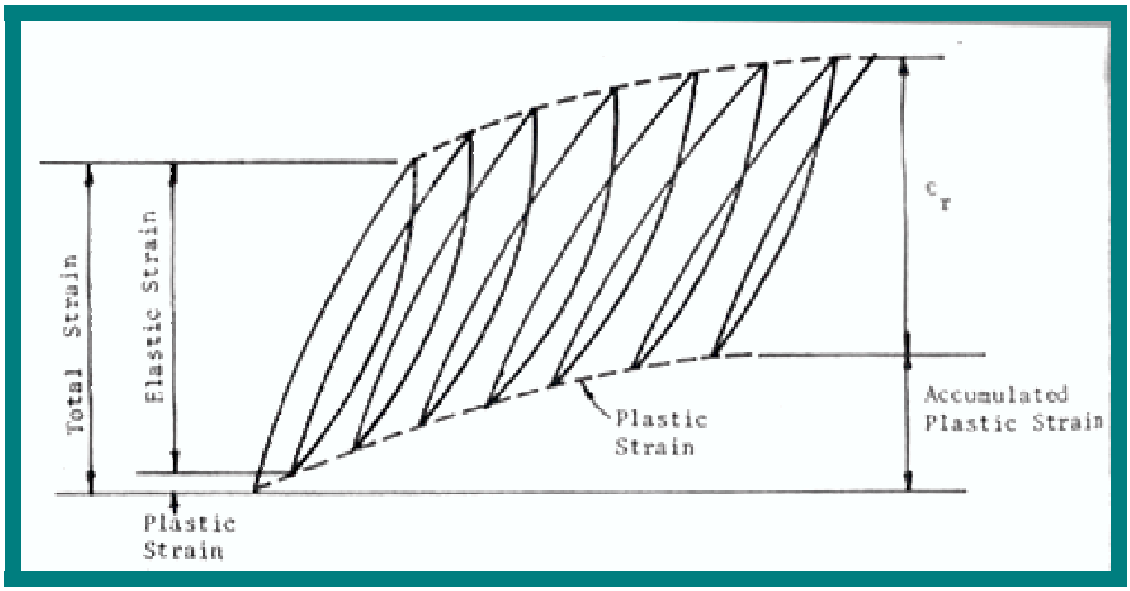


Figure 2.1 Strain Under Repeated Load During Resilient Modulus Test (Huang, 1993)

The resilient modulus can be obtained from field observation or from laboratory methods such as triaxial testing or indirect tension. In triaxial testing, the material is subjected to a repeated dynamic pulse-type loading in the axial direction. In the indirect tension testing, deformation is measured along the horizontal diameter of the sample while it is loaded through narrow strips along the vertical diameter direction.

A new experimental approach has been undertaken to determine the resilient modulus of an asphalt concrete mixture using indirect tension. Baladi and Harichandran (1989) utilized a repeated load indirect tension test method rather than the dynamic triaxial compression test methods. They determined that the repeatability of tests was poorer with the indirect tension test setup due to the equipment rather than the test mode. They also observed that the resilient characteristics of the AC mixture were dependent

upon the test temperature and number of load repetitions. Poisson's ratio was found to be almost constant within the range of 0.23 to 0.32 at the temperature range of 40 to 77°F (5 to 25°C).

The resilient modulus of asphalt concrete is another important input parameter in the M-E procedure for the flexible pavement as it reflects asphalt binder properties, temperature, time of loading, and mixture composition. The asphalt concrete modulus can be estimated using the asphalt binder properties, mixture composition, time of loading and temperature. Another approach would be to establish the modulus-split tensile strength relationship for typical asphalt concrete mixtures. This relationship can be utilized to predict the AC resilient modulus after measuring the split tensile strength obtained either from the laboratory or field (University of Illinois at Urbana-Champaign, 1992).

Nair et al. (1972) characterized asphalt concrete properties using elastic and viscoelastic constitutive laws. The experiments used for the elastic characterization were repeated load tests by applying a rectangular-waveform, which included the repetition of both axial and radial direction. Several cylindrical specimens with dimensions of 4 inches (101.6 mm) in diameter by 8 inches (203.2 mm) in height, were tested in the triaxial repeated load test setup under various axial and radial stress levels at controlled temperatures. The elastic response of the asphalt concrete was found to be a function of the stress and load frequency. It was also noted that temperature had a major influence on the elastic parameters for asphalt concrete.

In order to determine the rheological response of asphalt concrete specimens, they also carried out both creep and sinusoidal loading with load controlled haversine and sine-waveforms. The load was applied in the form of a step function and held constant for a twenty-minute period. The specimen was then unloaded and allowed to reach an equilibrium condition. This procedure was repeated over three cycles for each state tested. It was determined that viscoelastic model is a reasonable to use in characterizing the behaviors of asphalt concrete. Test results indicated that asphalt concrete had time-independent and time-dependent components of response under constant and repeated loads.

Mamlouk and Sarofim (1988) provided information about which moduli could be used for structural evaluation of asphalt mixtures. The three moduli discussed in their paper were complex, dynamic, and resilient moduli. The complex quantity (i.e. real and imaginary number) relating axial stress to axial strain in a cylindrical specimen subjected to sinusoidal loading was defined as the complex modulus. The real part of this quantity represents its stiffness, while the imaginary part characterizes the internal (material) damping property of the material. The dynamic modulus, which is the absolute value of the complex modulus, is a frequency dependent parameter, and ignores the phase lag between load and deformation. These moduli were measured by using an electrohydraulic machine capable of applying sinusoidal loading and strain-measuring devices.

The resilient modulus can be analyzed by computing the ratio of repeated stress to corresponding recoverable or resilient strain during a repeated loading. The resilient modulus can be determined in the laboratory using axial, triaxial, or diametrical methods.

Among the three different methods, the triaxial method was found to be more representative of the field conditions than the diametrical method due to the triaxial nature of the loading. The triaxial modulus was also found useful at high temperatures at which viscosity of the binder became small and the effect of confining pressure became significant. After analyzing each method, it was concluded that the triaxial or diametrical resilient modulus could be utilized to characterize the material. They concluded that the resilient modulus would be more realistic and useful than the static or dynamic modulus for the analysis of multi-layer pavement systems.

CHAPTER 3:
THE OHIO-SHRP TEST ROAD

3.1 Introduction

The Strategic Highway Research Program (SHRP) was a five-year, $150 million program funded by Congress to investigate the long-term field performance of various pavement sections. The Ohio-SHRP Test Road was constructed by the Ohio Department of Transportation (ODOT) in conjunction with the Federal Highway Administration (FHWA). Ohio University has lead the multi-university team assembled for the project and has taken on many responsibilities ranging from the installation of field sensors and testing of field pavements, to the analysis of environmental and dynamic response data.

The following provides descriptions of the site and project, the pavement materials used, the two types of field instrumentation methods applied. The initial report on the pavement performance at the site is also provided.

3.2 Site Description

The Ohio-SHRP test road is situated in Delaware County, Ohio, approximately 25 miles (40.24 km) north of Columbus. The 3.5 mile (5.63 km) long test road is located on a four-lane facility with a 170 ft. (51.8 m) wide median over an existing section of U.S. Rt. 23, which was constructed in the 1960s. This particular site was selected by the ODOT due to its flat topography, relatively uniform subsurface conditions, and uniform traffic and climate conditions. Prior to construction, design traffic loading of one million

ESAL per year was assumed. It was found that the soils existing at the site were mostly classified as A-4 or A-6 (fine-grained soils). This site location is known to experience multiple freeze/thaw cycles during the winter months. It was found from available climatological data that the annual precipitation and freezing index at the site were 38.1 inches (96.77 cm) and 116 degree-days, respectively. Also, it was reported that in any average year there were 16 days when the temperature rose above 90°F (32°C). Under these conditions, individual section performance could be directly compared.

3.3 Project Description

According to the SHRP Specific Pavement Studies (SPS) the pavement sections at the Ohio-SHRP Test Road were divided into four major experiments. The four experiments were defined as follows:

- SPS-1 Strategic Study of Structural Factors for Flexible Pavement
- SPS-2 Strategic Study of Structural Factors for Rigid Pavement
- SPS-8 Study of Environmental Effects in the Absence of Heavy Traffic
- SPS-9 Asphalt Program Field Verification Studies

SPS-1 and SPS-9 experiments were located on the southbound lanes and were constructed using asphalt concrete (AC). The SPS-2 experiment, located on the northbound lanes, was constructed with Portland cement concrete (PCC). The SPS-8 experiment was built on a ramp from the Village of Norton entrance onto the existing southbound lanes of U.S. Rt. 23.

A total of 40 test sections (14 for SPS-1, 19 for SPS-2, 4 for SPS-8, and 3 for SPS-9) were constructed for this project. This project also provided a good opportunity to obtain extensive data for the mechanistic-empirical (M-E) procedure. The ODOT incorporated the SHRP environmental instrumentation in 9 test sections of the rigid (or Portland cement concrete) pavement and 11 sections of the flexible (or asphalt concrete) pavement. Comprehensive response instrumentation for traffic loading and environmental changes were placed in 7 of the asphalt concrete test sections and 7 of the Portland cement concrete test sections.

Figure 3.1 depicts the layout of the Ohio-SHRP Test Road, with the six-digit numbering system established by the FHWA to identifying each section. The first two digits are fixed at 39, and indicate that the project is located in the State of Ohio. The next two digits indicate the SPS number. Finally, the last two digits identify each section specifically. For example, SHRP ID 390106 indicates that this is Section 6, under the SPS-1 experiment, located in the State of Ohio. The compositions of pavement materials used in this test are summarized in Tables 3.1 and 3.2.

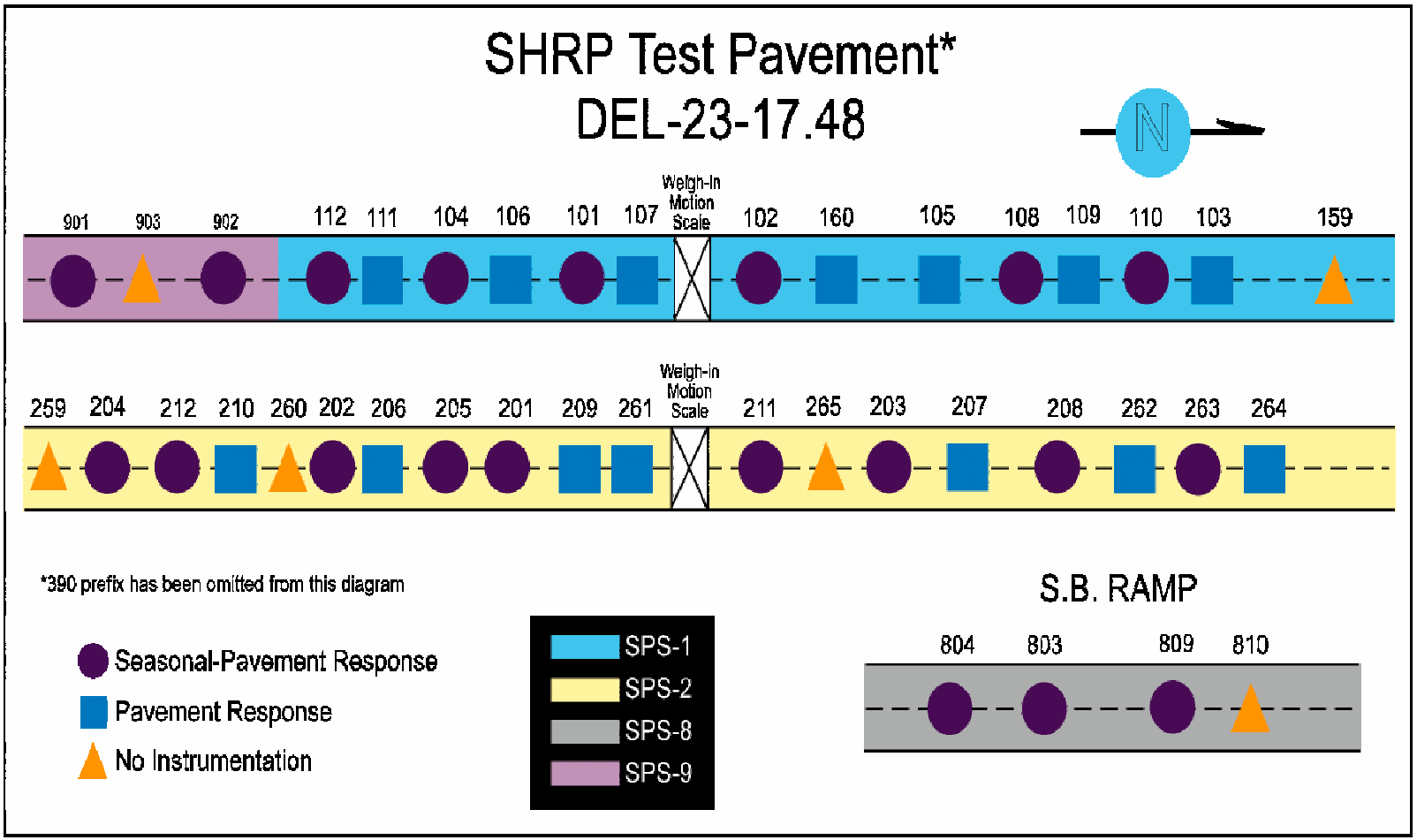


Figure 3.1 Layout of SHRP Test Sections at Ohio SHRP Test Road Site (Not to Scale)

Table 3.1 Project Layout for Asphalt Concrete Sections (ORITE, 1997)

Asphalt Concrete Studies				
SPS1				
Section	Thickness (in.)		Base Type	Drain
	AC	Base		
390101	7	8	DGAB	No
390102	4	12	DGAB	No
390103	4	8	ATB	No
390104	7	12	ATB	No
390105	4	8	4"ATB / 4" DGAB	No
390106	7	12	8" ATB / 4" DGAB	No
390107	4	8	4" PATB / 4" DGAB	Yes
390108	7	12	4" PATB / 8" DGAB	Yes
390109	7	16	4" PATB / 12" DGAB	Yes
390110	7	8	4" ATB / 4" PATB	Yes
390111	4	12	8" ATB / 4" PATB	Yes
390112	4	16	12" ATB / 4" PATB	Yes
390159	4	25	15" ATB / 4" PCTB / 6" DGAB	Yes
390160	4	15	11" ATB / 4" DGAB	Yes
SPS-8				
390803	4	8	DGAB	No
390804	7	12	DGAB	No
SPS9				
390901	4	22	AC-20 12" ATB / 4" PATB / 6" DGAB	Yes
390902	4	22	PG58-28 12" ATB / 4" PATB / 6" DGAB	Yes
390903	4	22	PG64-28 12" ATB / 4" PATB / 6" DGAB	Yes

Note : DGAB : Dense Graded Aggregate Base
ATB : Asphalt-Treated Base
PATB : Permeable Asphalt-Treated Base
PCTB : Permeable Cement-Treated Base
AC : Asphalt Concrete

Table 3.2 Project Layout for Portland Cement Concrete Sections (ORITE, 1997)

Portland Cement Concrete Studies						
SPS-2						
Section	Lane Width (ft.)	Flexural Strength at 14 days (psi)	Thickness		Base Type	Drain
			PCC (in.)	Base (in.)		
390201	12		8	6	DGAB	No
390202	14	900	8	6	DGAB	No
390203	14		11	6	DGAB	No
390204	12	900	11	6	DGAB	No
390205	12		8	6	LCB	No
390206	14	900	8	6	LCB	No
390207	14		11	6	LCB	No
390208	12	900	11	6	LCB	No
390209	12		8	8	4" PATB / 4" DGAB	Yes
390210	14	900	8	8	4" PATB / 4" DGAB	Yes
390211	14		11	8	4" PATB / 4" DGAB	Yes
390212	12	900	11	8	4" PATB / 4" DGAB	Yes
390259	12	900	11	6	DGAB	Yes
390260	12		11	8	4" PATB / 4" DGAB	Yes
390261	14		11	8	4" PCTB / 4" DGAB	Yes
390262	12		11	8	4" PCTB / 4" DGAB	Yes
390263	14		11	6	DGAB	Yes
390264	12		11	6	DGAB	Yes
390265	12		11	8	4" PATB / 4" DGAB	Yes
SPS-8						
390809	11	550	8	6	DGAB	No
390810	11	550	11	6	DGAB	No

Note : ATB : Asphalt-Treated Base
PATB : Permeable Asphalt-Treated Base
DGAB : Dense Graded Aggregate Base
PCTB : Permeable Cement-Treated Base
LCB : Lean Concrete Base

3.4 Pavement Materials

In order to examine the effect of the base type on the field pavement performance, the flexible pavement sections of the Ohio-SHRP Test Road were constructed with four types of base material, and the rigid pavement sections were also constructed with four types of base material. These base materials were asphalt-treated base (ATB), permeable asphalt-treated base (PATB), lean concrete base (LCB), permeable cement-treated base (PCTB), and dense graded aggregate base (DGAB). The following sections provide detailed information on each pavement material (including subgrade soils, asphalt concrete, and Portland cement concrete) utilized in the construction of the Ohio-SHRP Test Road.

3.4.1 Subgrade Soils

The subgrade soils at the project site consisted mostly of brown silty clay soils, classified as AASHTO A-4 or A-6 soil type. The subgrade soil was compacted with a sheepsfoot-roller and then carefully graded to bring its finished grade within 0.25 inch tolerance, and then verified by the ODOT inspectors at random locations. A summary of the soil classification test results is shown in Table 3.3.

Table 3.3 Subgrade Soil Types at Ohio SHRP Test Road

AASHTO Soil Classification	SHRP Section (Portland Cement Concrete)	SHRP Section (Asphalt Concrete)
A-4	390261	390109, 390110, 390160, 390809, 390810, 390901
A-6	390202, 390203, 390204, 390205, 390206, 390207, 390208, 390211, 390261, 390262, 390263	390103, 390104, 390105, 390111, 390112, 390803
A-7-5 or 7-6	None	390105 (A-7-5) 390101, 390107, 390901 (A-7-6)

[Note] Three soils samples, 390106, 390108, 390209 could not be classified into one of the soil groups due to insufficient data.

A nuclear moisture/density gage was used to measure the in-situ subgrade dry unit weight and moisture content. Samples were obtained every 150 feet (45.7 meter) along the centerline of the driving lane in each section at an approximately 12 inch (304 mm) depth below the finished subgrade surface by driving a Shelby tube. Specifications applied to the compaction of subgrade soils are listed in Table 3.4.

Table 3.4 Embankment Soil Compaction Requirements (Ohio DOT, 1997b)

Maximum Laboratory Dry Weight kg/cm^3 (lbs/ft^3)	Minimum Compaction Requirements Percent Laboratory Maximum
1440 - 1680 (90 - 104.9)	102
1681 - 1920 (105 - 119.9)	100
1921 and more (120 and more)	98

The key compaction properties are summarized in Table 3.5. Additional test results obtained on the soil samples are listed in Table 3.6.

Table 3.5 Average Standard Proctor Compaction Test Results on Subgrade Soils

AASHTO Soil Classification	Optimum Moisture Content (%)	Maximum Dry Density pcf (kg/m3)
A-4	13.5	117 (1,874)
A-6	14.6	115 (1,836)
A-7-6	15.8	112 (1,794)

Table 3.6 Additional Test Results on Shelby Tube Samples

Section No.	Moisture Content (%)	Dry Unit Weight (pcf)	UCS * (psi)	LL (%)	PL (%)	% Passing No. 200	AASHTO Classif.
390101	23.0	NA	30	47	32	83	A-7-6
390102	NA	NA	NA	34	6	77	A-4a
390103	14.9	112	29	38	19	77	A-6
390104	11.8	127	82	28	11	76	A-6
390105	18.3	113	36	73	40	80	A-7-5
390109	13.9	121	48	24	8	78	A-4a
390112	16.3	115	17	30	12	82	A-6
390203	14.3	125	82	29	12	80	A-6
390204	15.7	104	29	40	16	88	A-6
390206	19.8; 17.1	110; 114	14; 42	31; 33	12; 14	81; 81	A-6
390208	16.6	116	45	33	14	90	A-6
390212	26.8	96	17	NA	NA	NA	NA
390260	14.9	123	62	NA	NA	NA	NA
390261	14.5; 13.2	115; 118	37; 37	28; 29	10; 12	82; 76	A-4a; -6
390263	16.5; 15.4	111; 116	47; 33	30; 30	13; 12	79; 79	A-6
390803	17.1; 13.7	110; 129	24; 65	33	15	80	A-6
390804	16.5; 17.1	118; 115	35; 35	NA	NA	NA	NA
390810	17.1	120	59	27	9	64	A-4a
390901	14.9; 13.1	117; 119	53; 33	28; 27	10; 10	77; 76	A-4a
390901	20.3	100	19	42	21	88	A-7-6

[Note] * "UCS" = Unconfined Compression Strength.

3.4.2 Asphalt-Treated Base (ATB)

The Ohio Department of Transportation (ODOT) formulated the asphalt-treated base (ATB) to have 11.5% minimum voids in the mineral aggregate, 3 to 8% bitumen content, and 3 to 5% air voids. The mix design for the asphalt-treated base should meet the requirements of ODOT Item 302. The aggregate gradation data for the asphalt-treated base is presented in Table 3.7 and Figure 3.2. At lease 75% of the aggregate retained on the 3/8" sieve must have two or more fractured faces. The ATB was placed at variable depths, depending on the total thickness of the ATB for the section. Overall thickness of the ATB varied from 4 to 15 inches (102 to 381 mm), as shown in Table 3.1. Most lift thickness for this base was 2.5 to 3 inches (64 to 76 mm). The ODOT project notes required the ATB to be compacted to a minimum of 92% (90% for the first lift) of its theoretical maximum density. This base was used in the SPS-1 experiment sections (390103 through 390106, 390110 through 390112, 390159 and 390160) and in the entire SPS-9 experiment sections.

Table 3.7 Aggregate Gradation Data for ATB - ODOT Specification Item 301

Sieve Size Opening or Number	Percent Passing (%):	
	ODOT Specification	Average Test Results*
50 mm (2.0 in.)	100	100.0
38.1 mm (1.5 in.)	85 to 100	100.0
25.4 mm (1.0 in.)		74.0
19.1 mm (0.75 in.)	56 to 80	63.0
12.7 mm (0.5 in.)		55.0
9.5 mm (0.375 in.)	37 to 60	49.0
4.75 mm (No. 4)	22 to 45	33.0
2.36 mm (No. 8)	14 to 35	
2.0 mm (No. 10)		22.0
1.18 mm (No. 16)	8 to 25	
0.6 mm (No. 30)	6 to 18	
0.425 mm (No. 40)		12.0
0.3 mm (No. 50)	4 to 13	
0.18 mm (No. 80)		9.0
0.075 mm (No. 200)	2 to 6	6.7

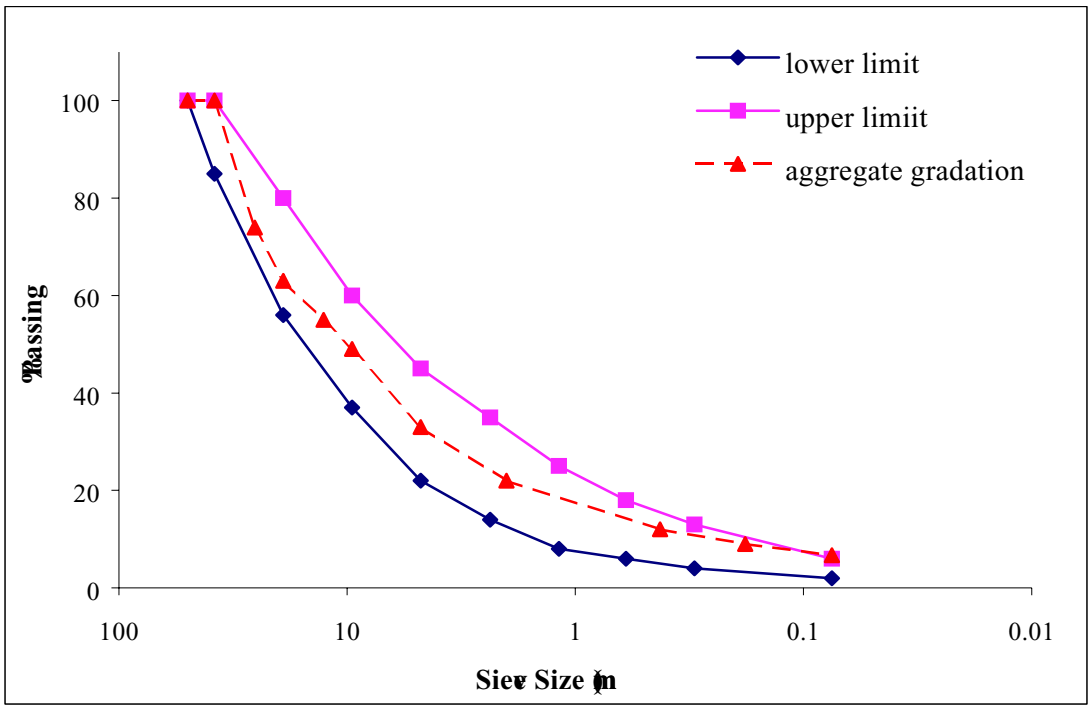


Figure 3.2 Typical Aggregate Gradation Curve for ATB

3.4.3 Permeable Asphalt-Treated Base (PATB)

Permeable asphalt-treated base (PATB), also known as asphalt-treated free draining base (ATFDB), is basically an open graded aggregate base bound with asphalt in the range of 2 to 2.5% binder by weight. It is currently covered by a supplemental specification. PATB has the same basic material requirements as ODOT Item 302, except that PATB uses #57 stone. The gradation of #57 stone is shown in Table 3.8. This base has a free draining nature due to the large number of voids, and may possess low strength.

A drainage system was installed on the right side of the pavement to prevent excess water from entering other base and subgrade materials. The drainage system consisted of a trench lined with a filter fabric filled with #8 gravel. The permeable asphalt-treated base was utilized on the SPS-1 experiment sections (390107, 390108, 390109, 390110, 390111, and 390112), on the SPS-2 experiment sections (390209, 390210, 390211, 390212, 390260, and 390265), and on the entire SPS-9 experiment.

Table 3.8 Gradation Data for #57 Stone Used in PATB

Sieve Size Opening or Number	Percent Passing (%):	
	AASHTO (1998) Specification	Average Test Results
25.4 mm (1.0 in.)	95 to 100	100
19.1 mm (0.75 in.)		85
12.7 mm (0.5 in.)	25 to 60	32
9.5 mm (0.375 in.)		14
4.75 mm (No. 4)	0 to 10	7
2.36 mm (No. 8)	0 to 5	
2.0 mm (No. 10)		5
0.425 mm (No. 40)		5
0.18 mm (No. 80)		4
0.075 mm (No. 200)		3

3.4.4 Lean Concrete Base (LCB)

The lean concrete base (LCB) is a Portland cement concrete meeting the requirements of ODOT Item 305, except as modified by plan note. This base used #57 crushed carbonate stone for coarse aggregate and the specification in Item 305 for fine aggregate. The LCB was designed for a 7 day compressive strength of 500 psi to 750 psi (3.4 MPa to 5.2 MPa). On the surface of the LCB, a double layer of wax-based curing compound was applied to resist bonding to the above PCC pavement material. The materials used in the mix design of the lean concrete base are provided in Table 3.9. The LCB was utilized in SPS-2 experiment sections (390205, 390206, 390207, and 390208).

Table 3.9 Mixture Design for LCB (Young, 2000)

Item	Description	Quantity (per cubic yd.)
Cement	Type I (by Southwestern)	160 lbs.
Water		235 lbs.
#57 Stone	Quartzite (by National Lime)	2,000 lbs.
Sand	Natural (by Prospect)	1,465 lbs.
Air Entrainment Agent	Daravair (by W.R. Grace)	4.1 oz.
Water Reducer	WRDA-82 (by W.R. Grace)	4.9 oz.
[Note] The above batch weights are based on surface-saturated dry (SSD) aggregate condition.		
Air Content	6%	
Slump	1 inch	
Unit Weight	140 lbs. Per cubic ft.	

3.4.5 Permeable Cement-Treated Base (PCTB)

The permeable cement-treated base (PCTB), also known as cement-treated free draining base (CTFDB), has physical characteristics similar to the PATB, with the exception that it is was bound by Portland cement rather than by asphalt cement. According to the ODOT Item 306 specifications, this base should have minimum cement content of 250 pounds per cubic yard (148 kilograms per cubic meter) with a water/cement ratio of approximately 0.36. Due to the fact that a uniform grain size coarse aggregate was used in the mix, the composition had to be compacted with a roller to achieve adequate density in the field. The mixture design used for the PCTB is summarized in Table 3.10. This particular type of base was applied to the selected SPS-1 (390159) and SPS-2 (390261, 390262) sections.

Table 3.10 Mixture Design for PCTB (Young, 2000)

Cement Type	Type 1 – Southwestern
#57 Stone	National Lime & Stone - Marion
(The cubic yard batch weight were established based on SSD aggregate condition)	
Cement	250 lbs.
Water	85 lbs.
#57 Stone	2515 lbs.
Unit Weight	105.5 pcf.
W/C ratio	0.34
Air	N/A
Slump	N/A

[Note] “SSD” = surface saturated dry

3.4.6 Portland Cement Concrete (PCC)

Three different mixture designs were used among the PCC pavement sections of the Ohio-SHRP Test Road. The first mixture, classified as the regular strength mixture, was used in twelve SPS-2 experiment sections (390201, 390203, 390205, 390207, 390209, 390211, 390260 through 390265). The second mixture, classified as the high strength mixture, was used in seven SPS-2 experiment sections (390202, 390204, 390206, 390208, 390210, 390212, 390259). The last mixture, classified as the low strength mixture, was applied to the entire SPS-8 experiment sections (390809, 390810). The high and low strength mixtures had a target 14 day flexural strength of 900 psi (6.2 MPa) and 550 psi (3.8 MPa), respectively. Details of the three mixture designs are summarized in Table 3.11.

Jointed plain concrete pavement (JPCP) slabs, with dimensions of 15 feet (4.6 meter) in length by 12 or 14 feet (3.7 or 4.3 meters) in width, were used in the rigid pavement sections at the Ohio-SHRP Test Road. Slab thickness was either 8 or 11 inches (203.2 or 279.4 mm). Dowel bars, spaced 12 inches (304.8 mm) apart at the mid-thickness, had the diameter of 1.25 or 1.5 inches (31.8 or 38.1 mm) depending upon the slab thickness. Tie bars with a 0.75 inch diameter were placed at 18 inches (457.2 mm) on the center along all longitudinal joints.

A drainage system was installed along all the sections containing a permeable base to disperse excess surface water. Three types of drains were used in this project, and are identified as details “A”, “B” and “ C” in Figure 3.3. More detailed information for each drain type can be found in a thesis by Macioce (1997).

Table 3.11 Concrete Mixture Designs for SPS-2 and SPS-8

	Mix Design 1	Mix Design 2	Mix Design 3
General Classification	Regular Strength	High Strength	Low Strength
Fine Aggregate (pcy): Natural Sand (SG 2.58)	1,300	950	1,316
Coarse Aggregate (pcy): Crushed Limestone (SG 2.62)	1,730	1,850	1,749
Portland Cement (pcy): Type I by Southwestern	510	750	350
Fly Ash (pcy)	90 (Class F)	113 (Class C)	52 (Class F)
Water (pcy)	240	270	235
Admixtures (oz.): - DARAVAIR - WRDA-82	 6 0	 10.8 43	 4 12
Slump (in.)	1	1	1
Air (%)	6 $\pm$ 1.5	6 $\pm$ 1.5	6 $\pm$ 1.5
Unit Weight (pcf)	141.9	142.0	141.6

[Note] "PCY" = lbs. per cubic yard; and "SG" = Specific Gravity.

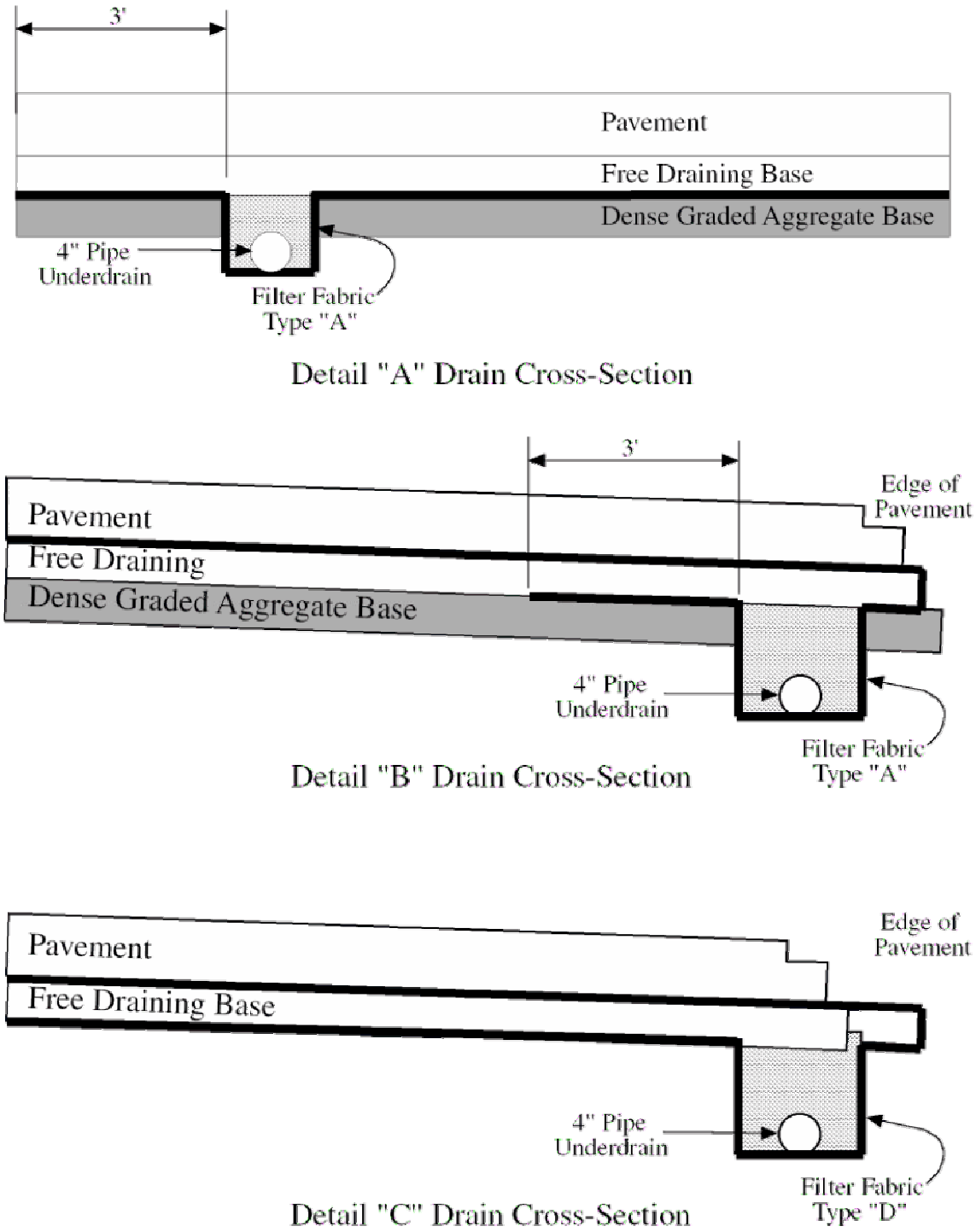


Figure 3.3 Drain Types (Macioce, 1997)

3.4.7 Asphalt Concrete (AC)

An asphalt concrete surface layer was applied to the SPS-1 and SPS-9 experiment sections. This layer consisted of 1.75 inches (44.5 mm) of the ODOT Item 446 (using Type 1 AC) applied over either 2.25 or 5.25 inches (57.2 or 133.4 mm) of the ODOT Item 446 (using Type 2 AC), depending on whether the total AC thickness was specified to be 4 or 7 inches (101.6 or 177.8 mm). Finer gradation aggregate was used in the top lift (Item 446) for a smoother surface finish.

The asphalt concrete in Sections 390901 and 390902 was designed with the standard AC-20 and PG58-28 asphalt cement, respectively. PG64-28 asphalt cement was applied in Section 390903. The AC-20 asphalt cement, with a viscosity of 2000 poises at 140°F (60°C), was classified based on the viscosity grade which was measured at 140°F as found in ASTM D3381.

Asphalt concrete in SPS-9 was designed using the Level 1 Superpave specifications. These specifications describe the tests used to classify asphalt binders, and state that performance graded binders are selected based on the climate in which the pavement will be used (Asphalt Institute, 1997).

PG 58-28 is the type of asphalt which will not rut at temperatures above 136 °F (58 °C), and will not crack when the temperature is below –18 °F (-28 °C). PG 64-28 is the type of asphalt that will not rut when the temperature is above 147 °F (64 °C) and will not crack when the temperature is below –18 °F (–28 °C).

3.4.8 Dense Graded Aggregate Base (DGAB)

The dense graded aggregate base (DGAB) used in this project should meet the requirements of the ODOT Item 304, and was composed of crushed limestone. The DGAB can be easily compacted to a very dense state due to its wide particle size distribution. The gradation specifications and typical gradation of the DGAB material found by the mechanical sieve method can be seen in Table 3.12 and Figure 3.4. This base material was utilized widely across the Ohio-SHRP Test Road, in the SPS-1 experiment sections (390101, 390102, 390105 through 390109, 390159, and 390160), SPS-2 experiment (except for 390205 through 390208, where LCB was specified), and the entire SPS-8 and SPS-9 experiments.

Table 3.12 Gradation Data for DGAB

Sieve Size Opening or Number	Percent Passing (%):	
	ODOT Specification	Average Test Results*
50.8 mm (2 in.)		100
38.1 mm (1.5 in.)	100	
25.4 mm (1.0 in.)	70 to 100	91.3
19.1 mm (0.75 in.)	50 to 90	81.3
9.5 mm (0.375 in.)		58.9
4.75 mm (No. 4)	30 to 50	40.1
2.0 mm (No. 10)		27.9
0.6 mm (No. 30)	7 to 30	
0.425 mm (No. 40)		13.2
0.18 mm (No. 80)		8.4
0.075 mm (No. 200)	0 to 10	

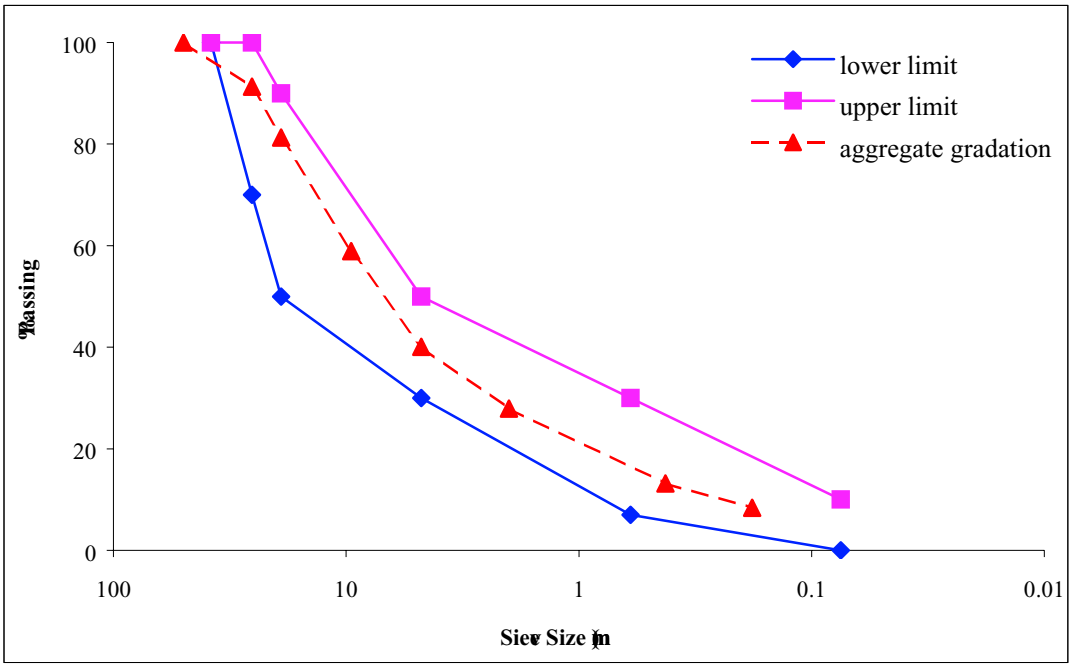


Figure 3.4 Typical Gradation Curve for DGAB

3.5. Field Instrumentation Methods

The Ohio-SHRP Test Road pavement sections received extensive sensor instrumentation, so that structural responses due to loading from controlled vehicles or non-destructive test devices, and environmental responses from moisture and thermal conditions could be measured in the field. In addition, a Weight-in Motion system and a weather station were installed to record traffic and climate data continuously. The following sections describe these field instrumentations.

3.5.1 Seasonal Parameters

Environmental sensors were installed to record ambient weather conditions, soil moisture in the base and subgrade, temperatures in the base and subgrade, frost depth, and depth to the water table.

An important parameter in pavement design is the moisture content of the soil, as it affects the resilient modulus, freeze-thaw capacity, and deflection. Time-domain reflectometry (TDR) probes, manufactured by Campbell Scientific, Inc., were installed every 6 to 12 inches (152.4 to 304.8 mm) below the top of the base layer to a depth of 6 feet (1.83 meter).

Temperature is also an important parameter to consider for stabilized pavement layers above the subgrade. It plays a major role in the fatigue life and settlement measurement, since it directly affects the resilient modulus and strength of the asphalt concrete materials. TP 101 thermistors or temperature sensitive resistors manufactured by Measurement Research Company (MRC) were utilized to monitor the temperature profile for each environmental response section as specified by the LTPP manual.

It is important to evaluate the frost penetration depth in the subgrade soil due to multiple freeze-thaw cycles during the winter season. Since soil stiffness has a tendency to decrease after each freeze-thaw cycle, the data on frost penetration depth will be required for mechanistic and overlay design procedures. A probe manufactured by the U.S. Army Corps of Engineers Cold Regions Research and Engineering Laboratory (CRREL) was selected for this purpose.

In order to measure the depth to the water table along the outside pavement shoulder, a total of nine 14.5 ft. (4.4 meter) long observation piezometers were installed. These instruments were made of two 1 inch diameter slotted PVC pipes connected together. The piezometers were threaded to a metal floor flange and anchored at the bottom of a borehole. An advantage of this instrument is that the pipes can also serve as a swell-free point of reference for the water surface level measurements.

3.5.2 Instrumentation for Load Response

Field instrumentation should provide information about the structural performance of both the flexible and rigid pavement sections in terms of horizontal strain, vertical deflection, and vertical pressure under dynamic loading conditions. The dynamic sensors were chosen based on input received from the FHWA and Sargand, 1999. All of the sensors were placed with 100 ft. (30.5 meter) long continuous lead wires to prevent potential problems at soldered joints from moisture. Table 3.13 summarizes the dynamic sensors utilized in this project.

Table 3.13 Dynamic Response Sensors (Sehn and Sargand, 1998)

Measured Parameters	Sensor
Horizontal AC Strain	Dynatest PAST-II AC Strain Gage
Horizontal PCC Strain	Dynatest PAST-II PCC Strain Gage
	TML KM-100B Strain Transducer
	TML PMR-60 Three Axes Rosette
	Carlson A-8 Strain Meter
	Geokon VCE-4200 Vibrating Wire Strain Gage
	Micro-Measurement EGP-5-120 Strain Gage
Vertical Deflection	Schaevitz GPD 121-500DC-LVDT
Vertical Pressure	Geokon Model 3500 Pressure Cell

More detailed information on the sensors listed in the above table can be found in a report prepared by Sargand (FHWA/OH-99/009, 1999).

CHAPTER 4:
CURRENT LABORATORY TEST METHODS

4.1 Introduction

In 1989, the SHRP issued a detailed guide to effectively carry out the LTPP study (FHWA, 1993a). In this document, one can find specific instructions on field sampling, material handling, and laboratory test methods. Pavement materials were divided into five groups (subgrade soil, unbound base, stabilized base, Portland cement concrete, and asphalt concrete). For each group, the LTPP developed protocols to determine a variety of engineering properties. Table 4.1 summarizes the laboratory tests to be performed on the pavement materials, along with the actual test procedures to be followed during each test.

According to the LTPP study (FHWA 1993a), both subgrade soil and unbound granular base/subbase samples shall be tested for natural moisture content, soil classification, moisture-density relations, and for resilient modulus. Tests should be conducted on stabilized base to determine its compressive strength and resilient modulus. If it is asphalt-treated base (ATB) material, resilient moduli must be measured at three different temperatures. PCC core specimens shall be subjected to the visual examination, thickness determination, and then to compressive strength/splitting tensile strength/static elastic modulus tests, after being air-dried for at least 40 hours under normal room conditions. AC core specimens should go through routine core examination, thickness

measurement, bulk specific gravity, maximum specific gravity, asphalt content (through extraction), resilient modulus, and split tensile strength at selected temperatures.

Table 4.1 List of Properties required by the SHRP procedure (FHWA, 1993)

Pavement Materials		Properties	Test Methods
Portland Cement Concrete (PCC)		Unit Weight	ASTM C642
		Split Tensile Strength	ASTM C496 or AASHTO T198
		Static Modulus & Poisson's Ratio	ASTM C469 or AASHTO T22
Asphalt Concrete (AC)		Bulk Specific Gravity	SHRP P07 or AASHTO T166
		Resilient Modulus & Poisson's Ratio	SHRP P07
		Indirect Tensile Strength	SHRP P07
		Creep Compliance	SHRP P06 or ASTM D3515
Stabilized Base	ATB & PATB	Bulk Specific Gravity	SHRP P07
		Resilient Modulus & Poisson's Ratio	SHRP P07
		Indirect Tensile Strength	SHRP P07
	CTB	Resilient Modulus & Poisson's Ratio	ASTM C3497 (modified) *
	LCB	Resilient Modulus & Poisson's Ratio	ASTM C3497 (modified) *
Unbound Granular Base/Subbase		Resilient Modulus	SHRP P46 or AASHTO T292
Subgrade Soil		Resilient Modulus	SHRP P46 or AASHTO T292

[Note] * Introduced due to a lack of guidelines by the LTPP.

As it is shown here, the total effort to characterize all the pavement material samples even from just one pavement test section is enormous. In the current research project on the U.S. Rt. 23 (Ohio), laboratory test responsibilities were divided between the ORITE and ODOT in order to expedite the full scope of the work. Table 4.1 lists for each pavement material type the laboratory tests that the ORITE laboratory was assigned to perform.

4.2 Laboratory Testing of Subgrade Soils

State-of-the-art test equipment was utilized to determine the resilient modulus of the subgrade soils. The test system featured an electroservo-controlled actuator, a large triaxial chamber, and a computerized command generator and data acquisition unit. This equipment was compatible with the current AASHTO test specification T-274 and the current SHRP test protocol P-46. The basic specifications of this equipment are summarized in Table 4.2. Figure 4.1 illustrates the overall set-up of this test system.

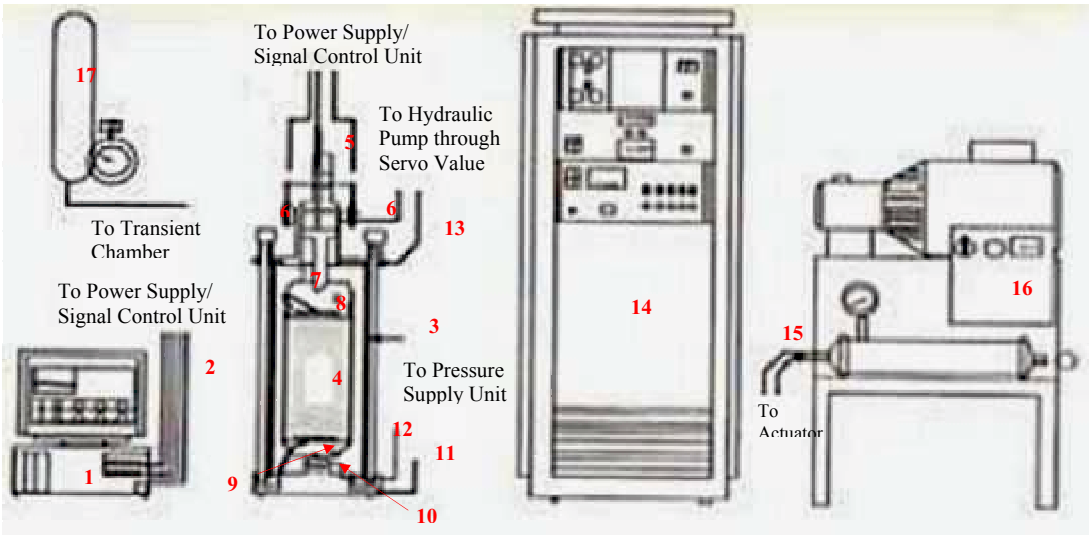
Table 4.2 Basic Information on Resilient Modulus Test Sensors (Sargand, 1999)

	Load Cell	Miniature LVDT's	System LVDT
Range	0 – 1,400 lbs.	± 250 mil	± 1,000 mil
Calibration Factor	140 lbs/V	25 mil/V	500 mil/V*
Accuracy	± 1.0% F.S.	± 1.0% F.S.	± 1.0% F.S.
Other Information	Temperature Range = 0 – 150 ^{o}F Excitation Voltage = 10V	Linearity = ± 25% F.S. Useful for Resilient Modulus Tests	Linearity = ± 25% F.S. Useful for Conventional Triaxial Tests

[Note] * Only 50% of the full range is being used.

Each soil sample was tested at more than three different moisture contents in order to reflect the variations in the field moisture conditions; the dry unit weight was kept relatively close to the average in-situ value. According to the SHRP P46 protocol (1996), the soil specimen shall be tested at a moisture content and dry unit weight close to those measured in the field at the time of subgrade preparation. Therefore, the approach adapted here exceeded the SHRP P46 minimum requirement. The samples

were prepared in the laboratory by statically compressing each lift inside a split mold with dimensions 2.8 inches (71.12 mm) in diameter by 6 inches (147.24 mm) in height. Detailed instructions for sample compaction and sample setup inside the triaxial chamber can be found in the SHRP P-46 Protocol (FHWA, 1996).



1. IBM PC
2. Interface Ribbon Cable
3. Triaxial Chamber
4. Test Specimen in Rubber Membrane
5. System LVDT
6. Miniature LVDTs
7. Actuator Piston
8. Top End Platen
9. Bottom End Platen
10. Load Cell
11. Pressure Air
12. Cable for Load Cell
13. Hydraulic Hose Attachment
14. Power Supply/ Signal Conditioning Unit for Servo Valve, Load Cell, and LVDTs
15. Hydraulic Hose Attachment
16. Hydraulic Pump
17. Air Pressure Source

Figure 4.1 Overall Set-up of Resilient Modulus Test System

At the beginning of the subgrade soil test, the preconditioning load cycles were applied over 500 repetitions of a deviatoric stress of 4 psi (27.58 kPa) using a haversine-shaped load pulse (consisting of a 0.1 second loading period, followed by a 0.9 second rest period), as illustrated in Figure 4.2. Initialization of all sensor readings was achieved at the end of the preconditioning load cycles (Sequence No.0, Table 4.3). The loading was then started by setting the confining pressure to 6 psi (41.4 kPa) and applying a deviatoric stress of 2 psi (13.8 kPa) (Sequence No. 2). Under any load sequence, the actual loading was repeated over 100 cycles, and the sensor output readings from the last 5 cycles were saved by the computer. The average of the last five recoverable strains was used as recoverable axial strains to compute the resilient modulus. The overall test involved a total of 15 load sequences, as shown in Table 4.3.

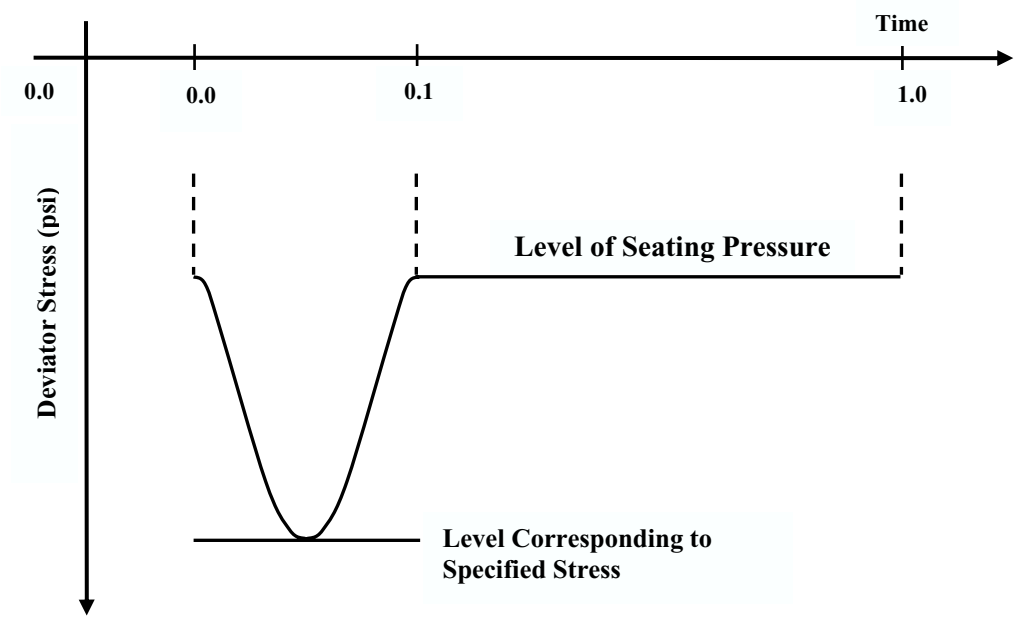


Figure 4.2 Haversine-Shaped Load Pulse

Table 4.3 Load Sequence Utilized in Subgrade Resilient Modulus Testing (SHRP P46, 1996)

Load Sequence No	Confining Pressure (σ_3)		Deviator Stress (σ_d)		Number of Repetitions
	kPa	psi	kPa	psi	
0	41.4	6.0	27.6	4.0	500
1	41.4	6.0	13.8	2.0	100
2			27.6	4.0	100
3			41.4	6.0	100
4			55.2	8.0	100
5			68.9	10.0	100
6	27.6	4.0	13.8	2.0	100
7			27.6	4.0	100
8			41.4	6.0	100
9			55.2	8.0	100
10			68.9	10.0	100
11	13.8	2.0	13.8	2.0	100
12			27.6	4.0	100
13			41.4	6.0	100
14			55.2	8.0	100
15			68.9	10.0	100

At the end of each test, the results were presented by the computer in a tabular form. Several options for the user to construct graphical plots of the test data were also provided. In most cases, the test results were represented in two styles of plot, resilient modulus vs. deviator stress and a logarithm of resilient modulus vs. a logarithm of deviator stress. Test results from a typical resilient modulus test are presented in Table 4.4 and Figures 4.3 and 4.4.

Table 4.4 Typical Result of Resilient Modulus in Tabular Form

12x1 * Ohio University *

Test #56

Resilient Modulus Test for material type 2

Data File rt23-56.dat Date: 08/01/97

Soil Sample type 2
Location oh23, us
Sample No. 390205
Specific Gravity 2.65

SOIL SPECIMEN WEIGHT:
Initial Wt. of Container
+Wet Soil-gms 0.0
Final Wt of Container
+Wet Siol-gms 0.0

Compaction Method static

Water Content After
Mr Testing % _____

SOIL SPECIMEN MEASUREMENTS:
Weight Wet Soil Used 0.0
Vertical Spacing Between LVDT Clamps(inch) 0.00

Diameter Top 2.78
Middle 2.78
Bottom 2.78
Average 2.78
Membrane Thickness 0.0140
Net Diameter (inch) 2.75
Ht Specimen+Cap+Base 7.00
Ht Cap+Base 1.00
Initial Length,Lo(inch) 6.00
Inside Diameter of Mold 2.80

SOIL SPECIMEN VOLUME:
Initial Area Ao
(inch²) 5.95
Volume Ao·Lo
(inch3) 35.69
Wet Density (pcf) 0.00
Compaction Water Content % 0.00
% Stauration 0.00
Dry Density (pcf) 0.00

LOAD ID:
SHRP P46 Soil type 2
Number of cycles per sequence 100
Load time 0.10 Cycle time 1.00 sec
Seating Load (lbs) 0.5
Waveform Type Havesine
Comments:
rt23-56.dat 8/1/1997

A Chamber Press. σ3 psi	B Nominal σd psi	C Mean Deviator Load lbs	D Standard Deviation of Load lbs	E Applied Deviator Stress psi	F Mean Recov Df LVDT #1 inch	G Mean Recov Df LVDT #2 inch	H Mean Recov. Def. inch	I Std. Dev. of Recov. Def. inch	J Mean of Resilient Strain in/in	K Mean of Mr psi	L Std Dev of Mr psi	M θ (σd+3σ3) psi
6.0	2.0	9.74	0.65	1.638	0.000122	0.000000	0.000061	0.000000	0.000010	~~160790~~	10817	19.638
6.0	4.0	22.05	1.41	3.706	0.000165	0.001318	0.000742	0.000079	0.000124	30137	2416	21.706
6.0	6.0	34.69	0.34	5.832	0.002631	0.003418	0.003024	0.000110	0.000504	11584	488	23.832
6.0	8.0	45.53	0.33	7.654	0.004425	0.005438	0.004932	0.000055	0.000822	9313	92	25.654
6.0	10.0	55.51	0.27	9.332	0.006476	0.007764	0.007120	0.000070	0.001187	7865	87	27.332
4.0	2.0	5.64	0.34	0.948	0.000000	0.000122	0.000061	0.000000	0.000010	93204	5649	12.948
4.0	4.0	22.49	0.14	3.781	0.000110	0.001880	0.000995	0.000054	0.000166	22850	1171	15.781
4.0	6.0	35.03	0.65	5.890	0.002747	0.004639	0.003693	0.000050	0.000615	9570	88	17.890
4.0	8.0	45.97	0.34	7.729	0.005042	0.006866	0.005954	0.000059	0.000992	7789	112	19.729
4.0	10.0	55.71	0.39	9.366	0.007184	0.009338	0.008261	0.000066	0.001377	6803	85	21.366
2.0	2.0	8.72	0.34	1.465	0.000000	0.001117	0.000558	0.000040	0.000093	15800	1244	7.465
2.0	4.0	21.19	0.00	3.563	0.000244	0.004053	0.002148	0.000000	0.000358	9950	80	9.563
2.0	6.0	32.98	0.34	5.545	0.003223	0.007098	0.005161	0.000097	0.000860	6449	117	11.545
2.0	8.0	43.37	0.36	7.292	0.005463	0.009796	0.007629	0.000050	0.001272	5735	30	13.292
2.0	10.0	53.66	0.68	9.022	0.008008	0.012592	0.010300	0.000120	0.001717	5256	86	15.022

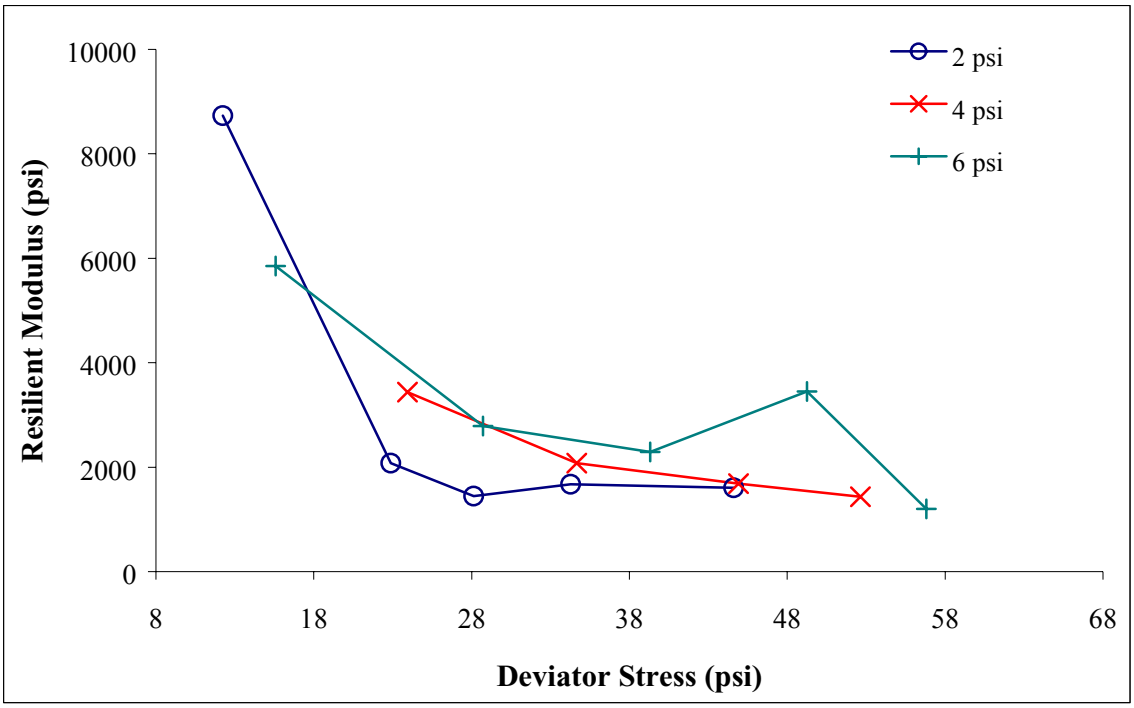


Figure 4.3 Plot of Deviator Stress Vs. Resilient Modulus for Subgrade Soil

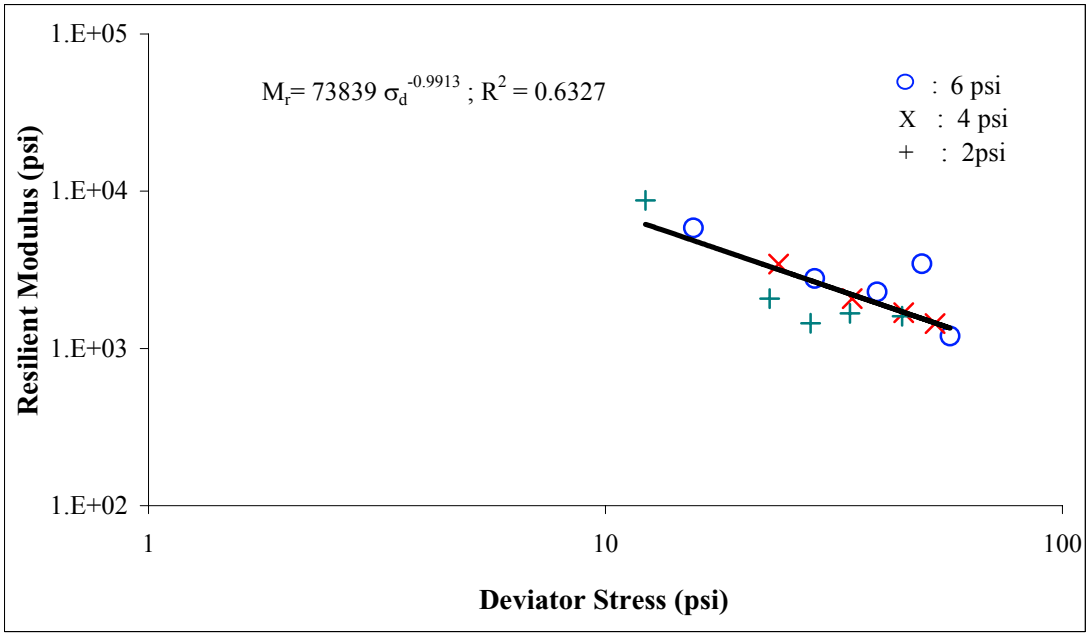


Figure 4.4 Plot of Deviator Stress Vs. Resilient Modulus of Subgrade Soil in Logarithmic Scale

Once a plot shown in Figure 4.4 is made, a correlation between the resilient modulus and deviatoric stress can be established through:

$$M_r = k\,(\sigma_d)^n \qquad (4.1)$$

where M_r = Resilient Modulus (psi); k = coefficient; n = exponent; and σ_d = deviatoric stress (psi).

Although the cumulative permanent deformation (or strain) is not mentioned in the final summary report, it can be determined easily from the miniature LVDT output readings taken at the beginning and end of the test.

4.3 Laboratory Testing of Unbound Base

The resilient modulus test protocol for the unbound base was similar to that specified for the subgrade soil (SHRP P46 protocol). The same type of repeated pulse loading was specified for testing the unbound granular soils (Figure 4.2). The only differences in testing methods between the fine-grained subgrade soil and unbound granular base material are found in their specimen preparation and load sequence specifications. In the protocol, fine-grained granular base and subgrade soil materials are classified as Type 1 soil and Type 2 soil, respectively. The specimen preparation specified for the Type 1 soil requires a dynamic compaction of the soil with an impact-drill in six lifts inside a larger split mold with dimensions 6 inches (152.4 mm) in diameter by 12 inches (304.8 mm) in height. The load sequences to be applied to the unbound granular soil sample, also begin with 500 conditioning load cycles (Table 4.5).

However, similarities between testing of the two different soil types end here. Levels of the confining stress are higher during the testing of unbound base materials. The confining stress is raised gradually during the testing of unbound base material, while it is lowered slowly during the testing of subgrade soils.

Table 4.5 Load Sequence Used in Resilient Modulus Testing of Unstabilized Base (SHRP P46 Protocol)

Load Sequence	Confining Pressure		Deviator Stress (σ_d)		Number of
No	kPa	psi	kPa	psi	Repetitions
0	103.0	15.0	103.4	15.0	500
1	20.7	3.0	20.7	3.0	100
2			41.4	6.0	100
3			62.1	9.0	100
4	34.5	5.0	34.5	5.0	100
5			68.9	10.0	100
6			103.4	15.0	100
7	68.9	10.0	68.9	10.0	100
8			137.9	20.0	100
9			206.8	30.0	100
10	103.4	15.0	68.9	10.0	100
11			103.4	15.0	100
12			206.8	30.0	100
13	137.9	20.0	103.4	15.0	100
14			137.9	20.0	100
15			275.8	40.0	100

4.4 Laboratory Testing of Concrete

In the current study, three laboratory tests (unit weight, split-tensile strength, and static modulus/Poisson's ratio) were performed on the Portland cement concrete (PCC). These PCC test methods have been standardized by the ASTM. The PCC specimens were cored from the actual rigid pavement sections at different ages to obtain variations in their basic mechanical properties with the age. The following sections describe each of these test methods in detail.

4.4.1 Unit Weight and Static Modulus/Poisson's Ratio

Prior to the static modulus test, a unit weight was determined for each core sample according to ASTM C567-91. After obtaining both the weight and volume, the unit weight of the specimen was computed by:

$$\gamma = \frac{W}{V} \tag{4.2}$$

where γ = unit weight (pcf); W = total weight of concrete (lbs.), and V = total volume of concrete (ft^3).

Static modulus (E) and Poisson's ratio (μ) of each selected concrete specimen were measured in an axial loading mode, according to ASTM Method C469-87a. This test was conducted at the ages of 28 days and 1 year. Before the test, the quality of each end of the test specimen was checked. If the specimen ends were not perpendicular to the axis with $\pm 0.5^o$ tolerance, and were not plane with 0.002 inch (0.05 mm) tolerance, the

specimen was to be capped by a sulfur-capping compound. Thin diametrical lines were drawn on the ends of the specimens to ensure that they were in the same axial plane. The diameter of the specimen was measured four times using a caliper to the nearest of 0.01 inch 0.25 mm). The length/height of the specimen, including the caps, was also measured and recorded to the nearest 1/16 inch (1.4 mm).

Compressometer and extensometer attached to a ring fixture (Figure 4.5) were installed on the specimen at the mid-height, of the test specimen in order to measure the axial and transverse deformations under a static compression load. Once the specimen was placed in the compression machine, a relatively small preload was applied to condition the specimen and verify that all dial gages were functioning. The specimen was then loaded up to about 40% of its ultimate strength at least twice. If this ultimate strength was unknown, the loading was continued until the axial strain reached the recommended maximum strain value (listed in Table 4.6), which depends on the unit weight and the age of the specimen.

The axial loading was applied continuously without shock at a constant rate within the range of 35 ± 5 psi per second (241.3 ± 34.5 kPa per second). The applied load and the longitudinal and circumferential deformation dial readings were recorded simultaneously when the axial strain was equal to 50 and 450 micro-strains. The data obtained during the test were then used to compute the static modulus (E) to the nearest 50,000 psi (0.35 GPa) and Poisson's ratio (μ) to the nearest 0.01, using Equations 4.3 and 4.4, respectively (ASTM C469, 1987):

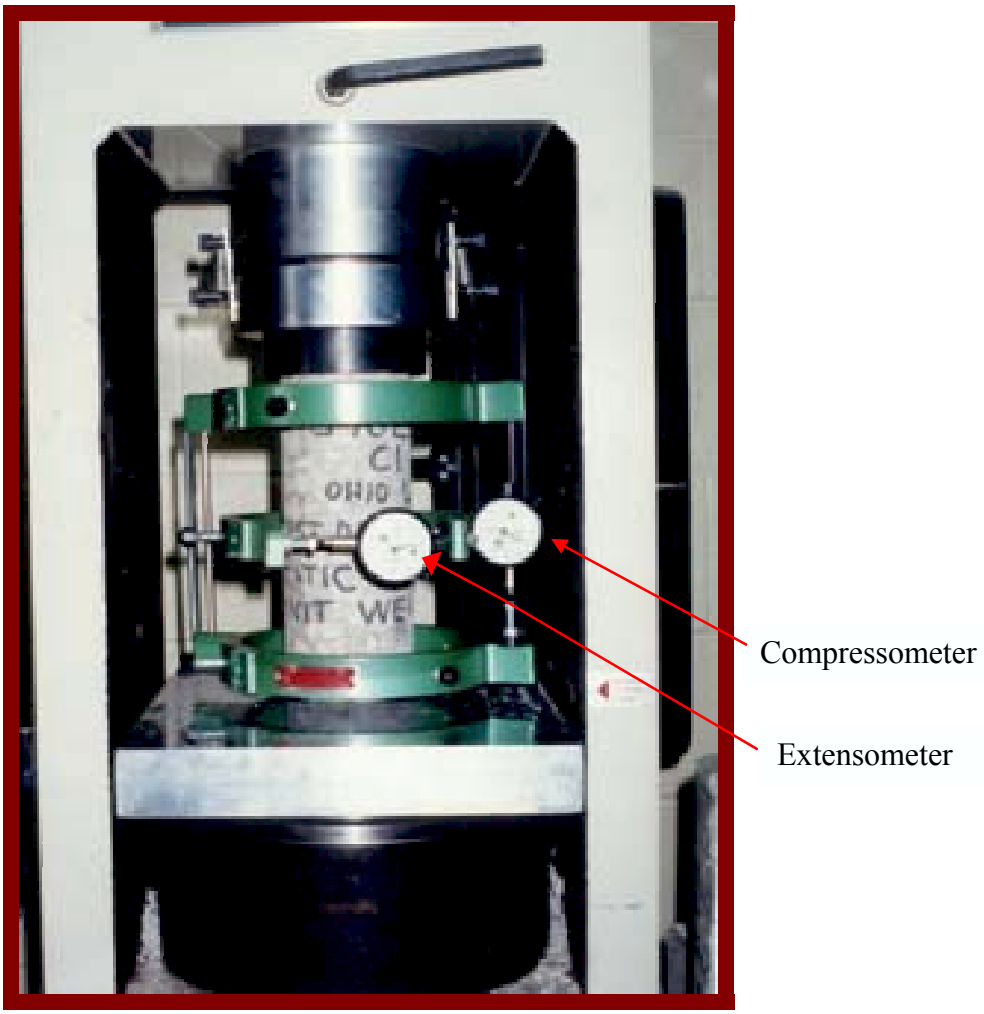


Figure 4.5 Compressometer/Extensometer Ring Device Attached to a PCC Test Specimen.

Table 4.6 Maximum Strain Values (ASTM C469-87a)

Unit Weight at Time, lb/ft^3 (kg/m^3)	Maximum Strain at Age Indicated (millionths)	
	Less than 7 Days	7 Days or More
205 (3280) and over	200	300
165 to 204 (2640 to 3264)	250	375
135 to 164 (2160 to 2624)	300	450
115 to 134 (1840 to 2144)	350	525
105 to 114 (1680 to 1824)	400	600
95 to 104 (1520 to 1664)	450	675
85 to 94 (1360 to 1504)	500	750
75 to 84 (1200 to 1344)	550	825

$$E = \frac{(\sigma_1 - \sigma_2)}{(\varepsilon_2 - 0.000050)} \tag{4.3}$$

where E = modulus of elasticity (psi); σ_2 = stress (psi) corresponding to the assumed maximum strain value obtained from Table 4.5; σ_1 = stress (psi) corresponding to a longitudinal strain ε_1 of 50 micro-strains; and ε_2 = longitudinal strain generated by stress σ_2.

$$\mu = \frac{(\varepsilon_{t_2} - \varepsilon_{t_1})}{(\varepsilon_2 - 0.000050)} \tag{4.4}$$

where μ = Poisson's ratio; ε_{t1}= transverse strain at mid-height of the specimen produced by stress σ_1; and ε_{t2}= transverse strain at mid-height of the specimen produced by stress σ_2.

4.4.2 Split Tensile Strength Test

The split-tensile strength test provides an alternative to the conventional unconfined compression strength test. In the Ohio-SHRP project, the unconfined compression tests were conducted on the PCC specimens by the ODOT. The split-tensile test was performed at the ORITE laboratory to measure the splitting tensile strength of the concrete core specimens. In this test, each specimen was placed on its side and subjected to a compressive line load applied diametrically, according to the ASTM 496-90 (Figure 4.6).

One long, thin strip of plywood was inserted at each contact between the specimen and the loading surfaces to ensure a relatively uniform distribution of the applied force. This small precaution, suggested in the ASTM method, can prove to be crucial, especially when testing cored specimens that tend to have imperfect shapes. The plywood strips, the supplementary bars and the test cylinder were all centered by means of the aligning jig. The load was then increased continuously at a rate of 100 to 200 psi/minute (0.7 to 1.4 MPa/minute) until the specimen developed a splitting crack along the vertical centerline. The maximum load that the specimen supports before it failed was used to calculate the splitting tensile strength of the specimen:

$$T = \frac{2P}{(\pi / d)} \tag{4.5}$$

where T = split-tensile strength (psi); P = maximum load sustained (lbs.); and D diameter of the specimen (inch).

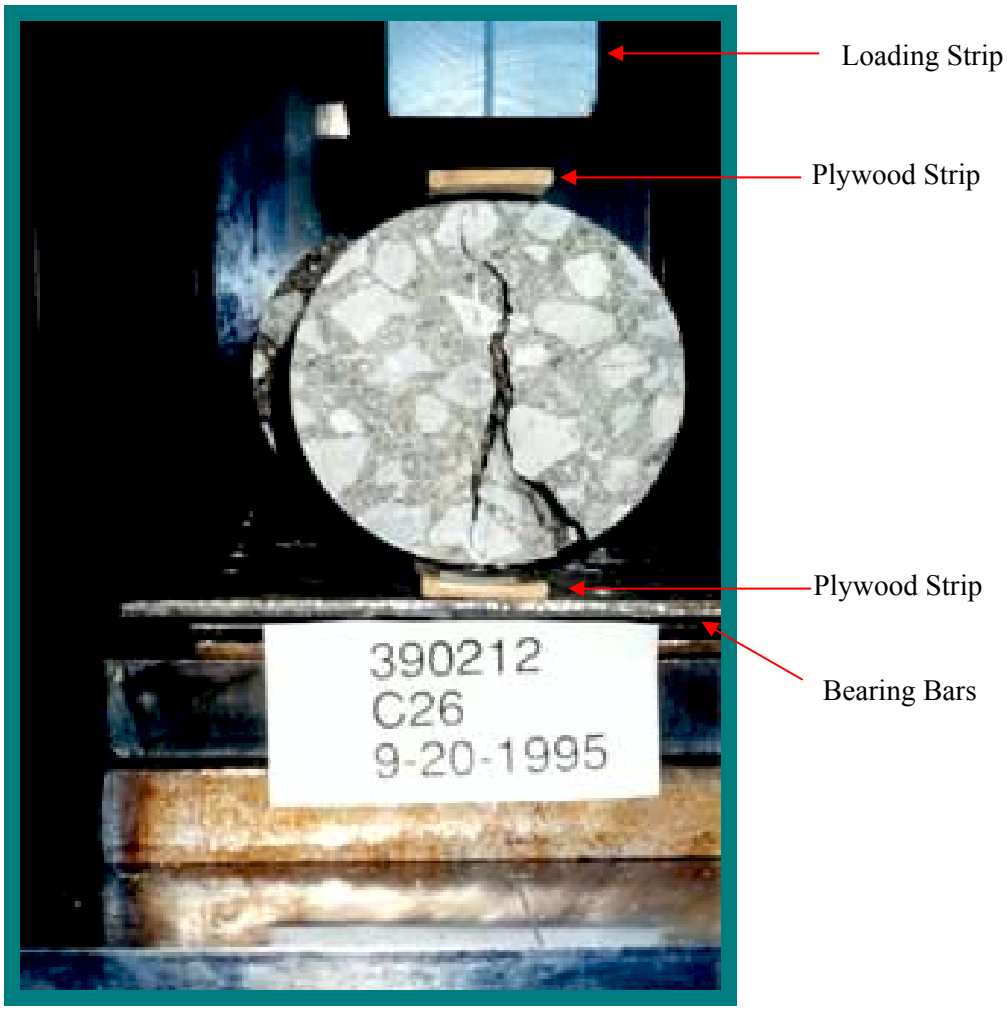


Figure 4.6 Split Tensile Strength Test Set Up (ASTM C496-90)

A sketch or a picture can be taken to record the location and nature of the splitting cracks. The split-tensile strength can be used to estimate the unconfined compression strength. The empirical correlation between them will be discussed later in Section 5.5.4.

4.5 Laboratory Testing of Asphalt Concrete

In the current study, asphalt concrete (AC) specimens cored from the flexible pavement sections received the most extensive laboratory testing. This was because the

AC material was known to behave in a much more complicated manner than the PCC. One major distinction between these materials is that the mechanical properties of the AC are very sensitive to temperature variations. Because of this temperature sensitivity, most of the available standard test methods require that the testing of the AC specimen be performed at three different temperatures. The laboratory tests performed on the AC specimens included: bulk specific gravity, resilient modulus, dynamic modulus, indirect tensile strength, and creep compliance.

4.5.1 Bulk Specific Gravity Test

Bulk specific gravity tests were performed by the AASHTO T166-93 Method A. First, the dry mass of a cored sample was recorded at standard room temperature of 77 ± 9°F (25 ± 5°C). The sample was then immersed in water at 77 ± 1.8^{o}F (25 ± 1°C) for 4 ± 1 minutes, to obtain the submerged mass of the specimen. The specimen was removed from the water and quickly dried by blotting with a damp towel. The surface-dry mass was then determined by weighing. The bulk specific gravity of the core sample was computed by using the following formula:

$$BSG = \frac{A}{B - C} \tag{4.6}$$

where A = dry mass (g); B = saturated surface-dry mass (g); and C = immersed mass (g).

4.5.2 Resilient Modulus Test

SHRP Protocol P07 (FHWA, 1993) describes a relatively new method to measure the resilient modulus of bound aggregate material, such as asphalt concrete. In this test, the AC specimen was placed on its side as in the indirect tension test mode, and subjected to repeated pulse loading along the vertical centerline of the sample (Figure 4.7).

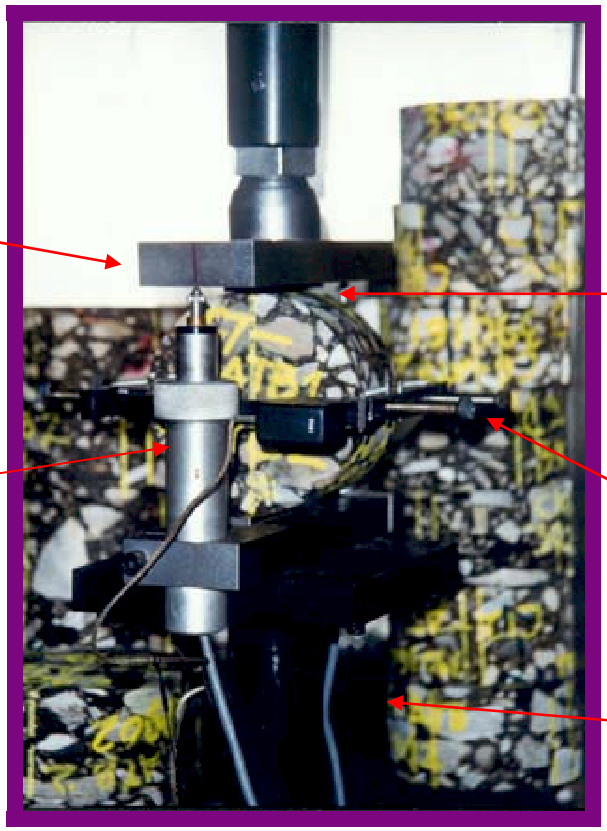


Figure 4.7 Testing Setup for Asphalt Concrete Using SHRP P07

Cross-hair lines were marked visibly on the faces of the cored sample to aid the positioning of the test specimen on the narrow loading strip and installation of horizontal

extensometer on the specimen. The typical dimensions of the core were 4 inches (101.6 mm) in diameter by 2 inches (50.8 mm) in thickness.

The latest laboratory test equipment was utilized to conduct the resilient modulus test. A fully computerized testing system, consisting of MTS 810 load frame and Test Star II computer data acquisition unit, made up the heart of the test equipment. This system included all the sensors (LVDTs, load cell, and horizontal extensometer) as well as the power supply/signal conditioners for the sensors. An MTS 651 environmental chamber was used in conjunction with the load frame, and was large enough to house the entire test setup inside and allow the loading of the specimen under a specified temperature condition. This chamber had the capability to maintain a constant temperature within 2 $^{\circ}$F (1 $^{\circ}$C) at settings ranging from -22 to 212 ± 2 $^{\circ}$F (-30 to 100 ± 1 $^{\circ}$C). The horizontal and vertical deformations were simultaneously recorded through LVDTs to measure the resilient modulus and Poisson's ratio.

The loading levels used in the resilient modulus test were determined by performing the indirect tensile strength test on the specimen taken from the same locations as those for resilient modulus tests. The strength test was carried out prior to the resilient modulus test as per SHRP P07 - Attachment A. The actual load repeatedly applied to the specimen was the same haversine-shape load as that specified in the test methods for the subgrade soil and unbound granular base material (shown previously in Figure 4.2).

From the resilient modulus test data, two separate resilient moduli can be computed. The instantaneous resilient modulus was computed using the recoverable

horizontal deformation that occurs during the unloading portion of one load-unload cycle. The total resilient modulus is calculated using the total recoverable deformation including both the instantaneous recoverable and the continuing recoverable deformation during the rest period of one cycle. The former represents the elastic modulus, while the latter (which is slightly lower) is believed to reflect more viscoelastic property. Figure 4.8 depicts the total and instantaneous resilient moduli.

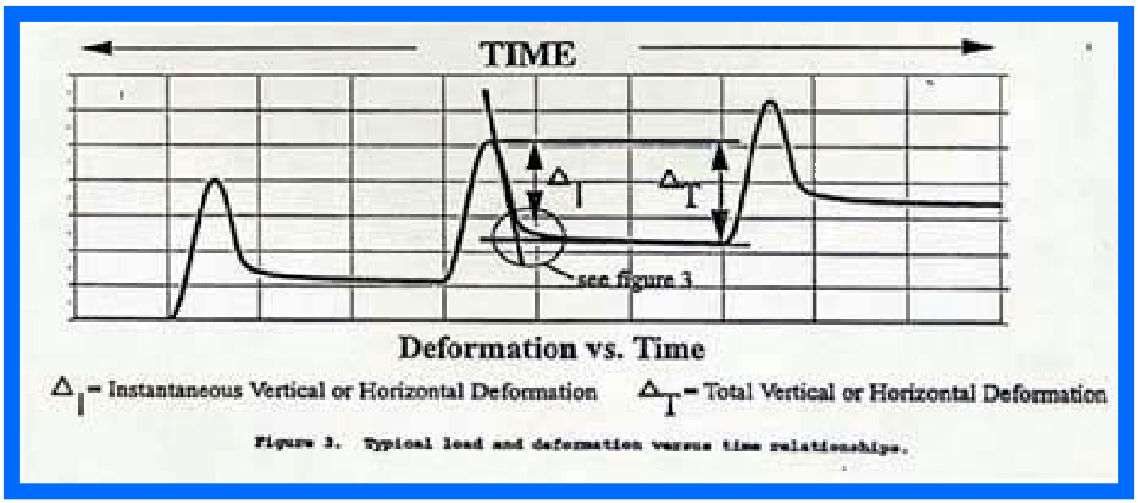


Figure 4.8 Total and Instantaneous Resilient Modulus (after SHRP P07, 1993)

The test specimens were tested along one diametrical axis at test temperatures of 41, 77 and 104 ± 2 °F (5, 25 and 40 ± 1 °C). Each specimen was kept in the environmental chamber for at least 24 hours prior to the tests at 41 and 77 °F (5 and 25 °C). The specimen needed to be kept in the chamber for at least 3 hours, but not exceed 6 hours prior to testing at 104 ºF (40 °C). After the completion of resilient modulus tests at 104 °F (40 °C), the test specimen was returned to 77 °F (25 °C) to perform an indirect

tensile test described subsequently. The total Poisson's ratio and resilient modulus were computed by using the following equations (SHRP P07, 1993b):

$$\mu_{Rt} = 3.59 * \frac{\Delta H_t}{\Delta V_t} - 0.27 \qquad (4.7)$$

where μ_{Rt} = total resilient Poisson's ratio; ΔV_t = total recoverable vertical deformation (mm); and ΔH_t = total recoverable horizontal deformation (mm).

$$M_{Rt} = P \frac{(\mu_{Rt} + 0.27)}{t \times \Delta H_t} \qquad (4.8)$$

where P = repeated load (N); M_{Rt} = total resilient modulus of elasticity (MPa); and t = thickness (mm).

The MTS Star II system program can also provide various plots of the test data, such as load versus time, horizontal deformation versus time, and vertical deformation versus time. Figures 4.9, 4.10, and 4.11 present some example plots. The computed Poisson's ratio must be between 0.1 and 0.5. Theoretically, the Poisson's ratio cannot be greater than 0.5. If it was less than 0.1 or greater than 0.5, it must be assumed to be equal to 0.1 or 0.5, respectively. Therefore, the computed total resilient modulus value needs to be adjusted accordingly.

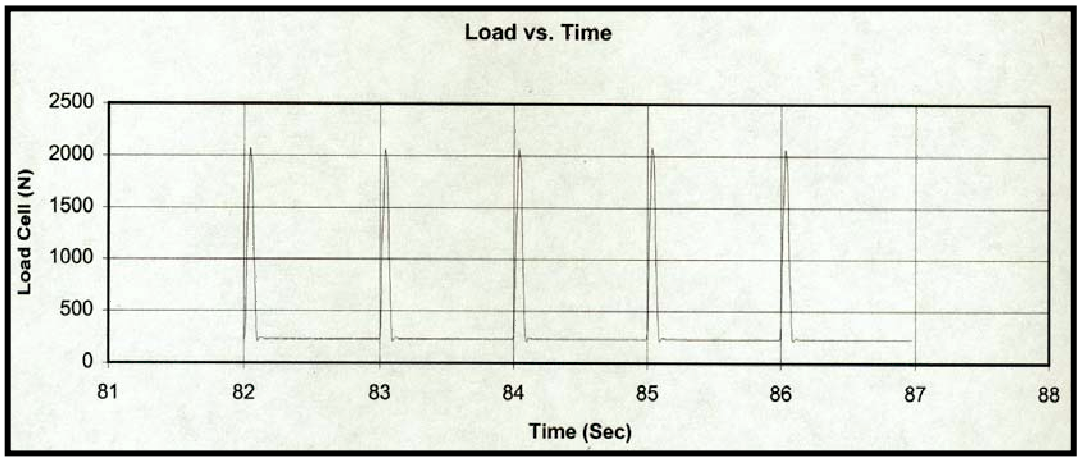


Figure 4.9 Typical Response of Load Vs. Time During Resilient Modulus Test

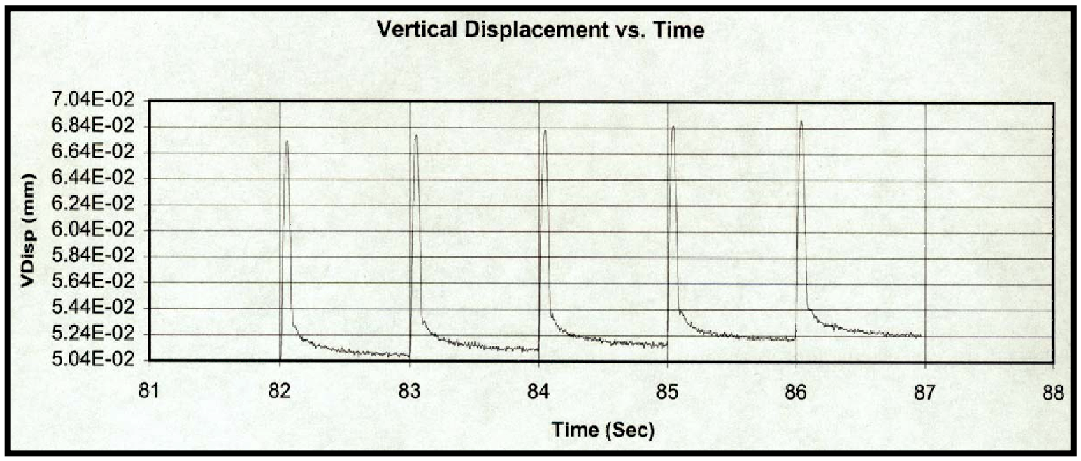


Figure 4.10 Typical Response of Vertical Deformation Vs. Time During Resilient Modulus Test

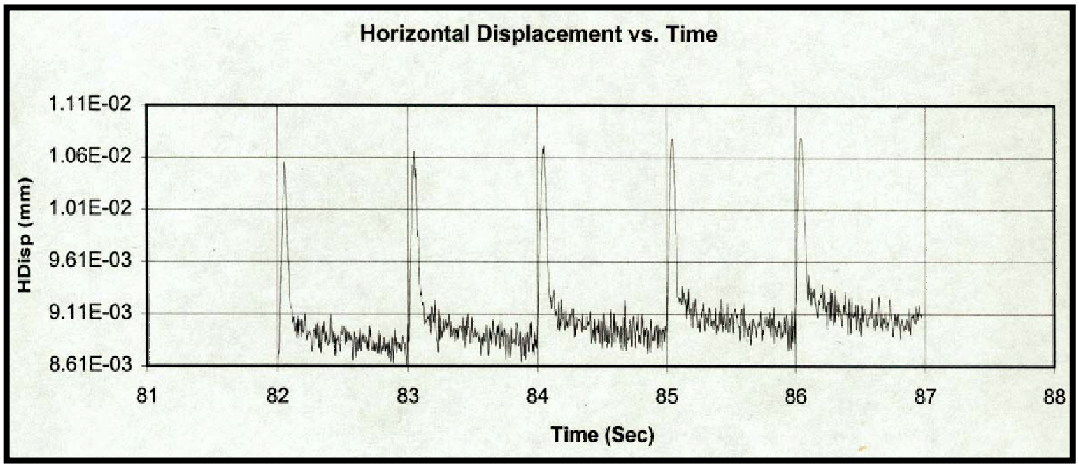


Figure 4.11 Typical Response of Horizontal Deformation Vs. Time During Resilient Modulus Test

4.5.3 Indirect Tensile Strength Test

This test is useful for establishing the tensile strength of the AC material, for comparing overall quality of different mix designs, and for specifying the load magnitude to be utilized during the resilient modulus test in the indirect tension mode. In addition, the indirect tensile strength value may be used to estimate the resilient modulus of AC, using a published empirical correlation between them.

After completing the resilient modulus tests on an AC specimen at the three temperature settings, the specimen temperature was gradually brought down to standard room temperature of 77 °F (25 °C) to perform the indirect tensile strength test. Specifications for performing this test are presented in the SHRP P07 Protocol-Attachment A. The specimen was positioned on its side on top of the narrow loading strip and loaded in compression along the vertical diametrical axis at a constant deformation

rate until it failed. Failure was defined as the point or deformation at which the load could no longer be increased. The test equipment used to perform this test was the same as that used in the indirect tension mode resilient modulus test.

The indirect tensile strength can be calculated using the following formula presented in the SHRP P07 protocol (1993c):

$$S_t = \frac{(50.127 \times P_o)}{t} \left[\sin\left[\frac{1455.313}{D}\right] - \left[\frac{12.7}{D}\right] \right] \tag{4.9}$$

where S_t = indirect tensile strength (kPa); P_o= maximum load sustained (N);

t = specimen thickness (mm); and D = specimen diameter (mm).

4.5.4 Creep Compliance Test

Creep compliance test results can be used to designate the overall quality of the asphalt material and to estimate the AC stiffness for pavement design and evaluation models. The data from this test can also be used to investigate the effect of temperature, load magnitude, and creep loading time on asphalt material properties. Although this test does not involve any applications of cyclic loading to the specimen, the test data can identify the mix stability at different temperatures. Data from the creep compliance test can also be used to calculate creep strain and creep stiffness for the AC specimen.

The creep compliance test requires the application of a constant static load to a cylindrical asphalt concrete specimen for a fixed duration of time along the centric longitudinal axis. During the loading duration, the total axial compressive deformation

response of the specimen is measured at specific time increments. The load is then removed from the specimen to record the time rate of rebound of the specimen deformations.

A core specimen, having dimensions of 4 inches (101.6 mm) in diameter by 4 inches (101.6 mm) in height was tested, according to the SHRP P06 protocol. This test used the same advanced test equipment as the resilient modulus test. An environmental chamber was integrated into the test setup to maintain each specific temperature during the test. Two LVDTs, positioned 180° apart next to the specimen, were used to measure the axial deformations of the specimen. Figure 4.12 shows typical test setup.

The test specimen was then placed in the environmental chamber for at least 24 hours prior to the test at 41 °F (5 °C) and 77 °F (25 °C) and 3 to 6 hours at 104 °F (40 °C) and 140 °F (60 °C). The test began by subjecting the specimen to a loading/unloading cycle at 41°F (5 °C), proceeding in the same manner at 77 °F (25 °C), then at 104 °F (40 °C), and finally at 140 °F (60 °C) on the same specimen.

Three cycles of preload conditioning were applied at one minute intervals followed by a one minute rest period for each cycle. The magnitudes of the conditioning stress were 80, 20, 10, and 5 psi (551.6,137.9, 69.0, and 34.5 kPa) at the test temperature of 41 °F (5 °C), 77 °F (25 °C), 104 °F (40 °C), and 140 ° F (60 °C), respectively.

After the initial conditioning, the actual static load was applied for a period of 60 minutes ± 15 seconds, during which the axial deformation readings from the LVDTs were recorded at elapsed times of 1, 10, 100, 1000, 1800, 2700, and 3600 seconds. The

load was then released, and the rebounding deformation data was recorded for an additional 60 minutes under no load.

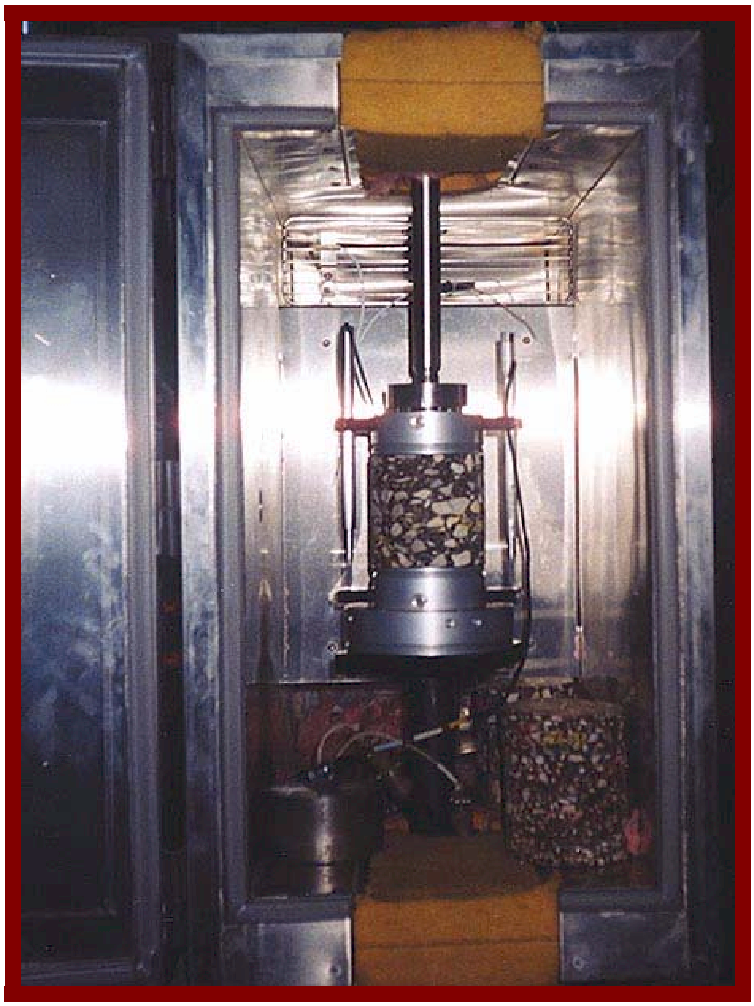


Figure 4.12 Creep Compliance Test Set-up

The raw data were plotted in terms of the vertical deformation versus time to analyze the response of each LVDT. Typical responses from the two LVDTs during the test are shown in Figures 4.13 and 4.14. Creep compliance was calculated at times of 1,

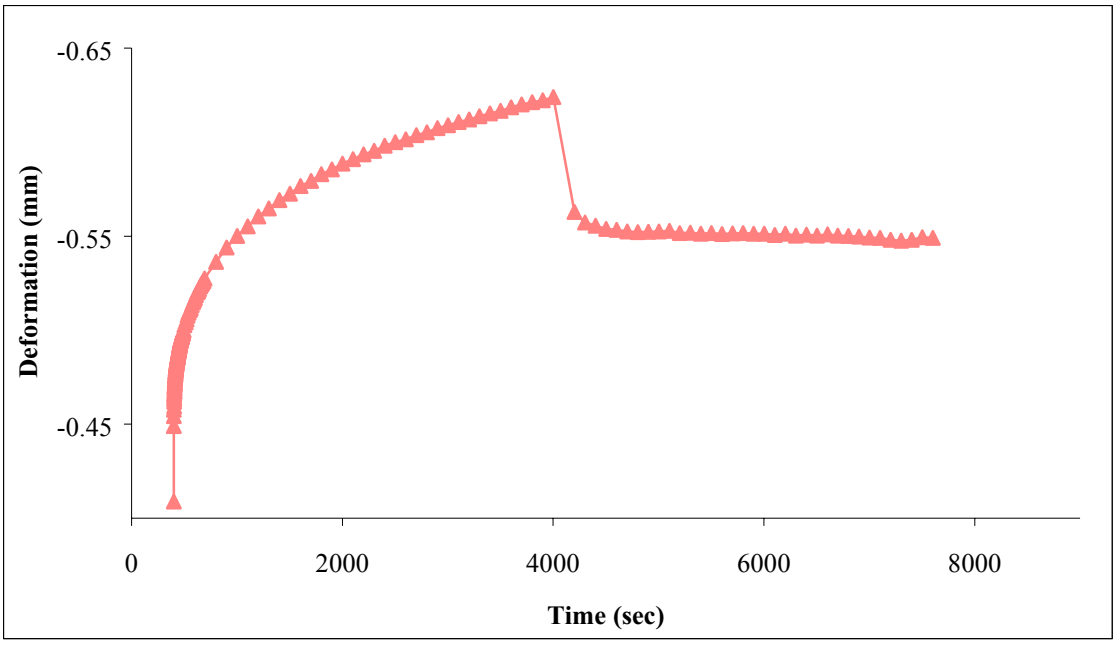


Figure 4.13 Typical Response of LVDT1 Deformation Reading Vs. Time During Creep Compliance Test

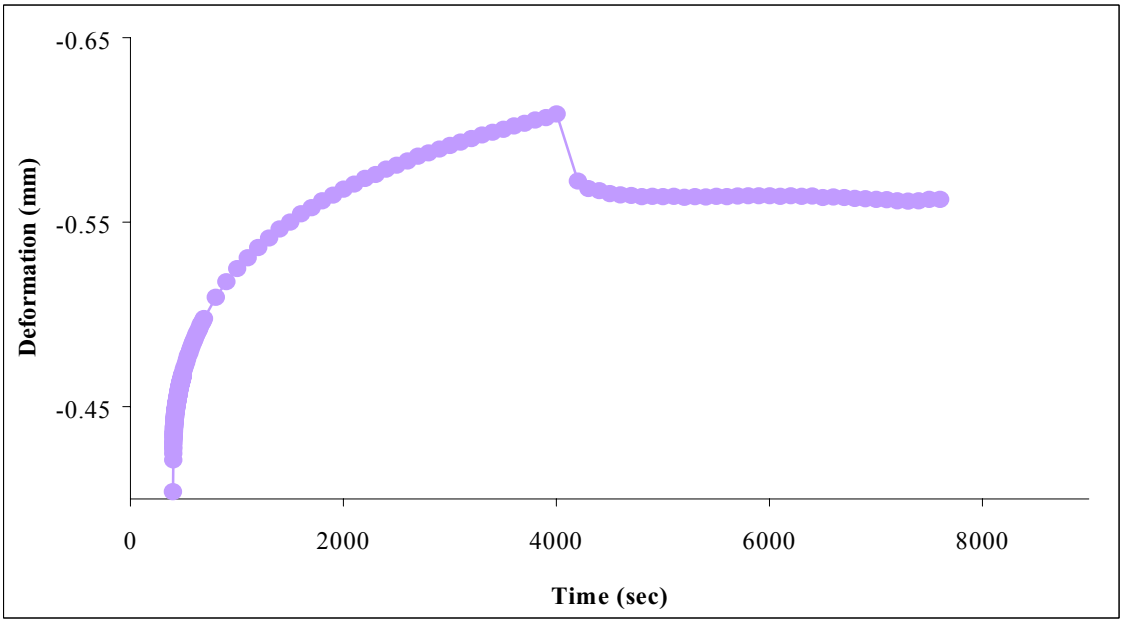


Figure 4.14 Typical Response of LVDT2 Deformation Reading Vs. Time During Creep Compliance Test

10, 100, 1000, 1800, 2700, and 3600 seconds for each specimen by using the following formulas (FHWA, 1993d):

1. ***Axial compressive strain***

$$\varepsilon_t = \frac{\delta_t}{H_0} \tag{4.10}$$

where ε_t = axial compression strain at time t (mm/mm); H_o = initial height of the specimen (mm); and δ_t = average axial deformation at time t, based on two separate LVDT measurement (mm).

2. ***Adjusted axial compressive stress***

$$\sigma_{adj} = \frac{(4 \text{ x } P_{adj})}{(\pi d^2)} \tag{4.11}$$

where P_{adj} = final adjusted load used during the test (N), and d = diameter of the specimen (mm).

3. ***Creep compliance***

$$D_t = \frac{\varepsilon_t}{\sigma_{adj}} \tag{4.12}$$

where D_t = creep compliance at time t (mm/mm/kPa).

4. Creep stiffness

$$S_{ct} = \frac{\sigma_{adj}}{\varepsilon_t} \tag{4.13}$$

where S_{ct} = creep stiffness at time t (kPa/mm/mm).

5. Permanent Strain

$$\varepsilon_p = \frac{\delta_p}{H_o} \tag{4.14a}$$

$$\delta_p = \frac{\delta_{pL} + \delta_{pH}}{2} \tag{4.14b}$$

where ε_p = permanent axial strain (mm/mm); δ_p = permanent axial deformation (mm); δ_{pL} = lowest permanent axial deformation (mm); δ_{pH} = highest permanent axial deformation (mm).

4.6 Laboratory Testing of Stabilized Base Materials

Currently, there is no standard laboratory test method proposed by ASTM, SHRP, or AASHTO for the resilient modulus of stabilized base material. Therefore, in the present study it was decided to:

1) Measure the resilient modulus of the ATB specimens by the SHRP P07 protocol (indirect tension mode), since the ATB material was physically similar to the AC.

2) Measure the resilient modulus of most of the stabilized base materials by an alternative approach, similar to the ASTM D3497-85 (dynamic modulus test method).

This alternative method is based on research done on ATB for the FHWA by Smith and Nair (1972). Each specimen, with typical dimensions of 4 inches (101.6 mm) in diameter by 8 inches (203.2 mm) in height, was instrumented with four uniaxial strain gages for measuring both the axial and lateral strains (Figure 4.15). The strain gages had a resistance of 120 Ω, a gage factor of 2.090, and a transverse sensitivity of –1.3 ± 0.2% at 75.2 °F (24 °C).

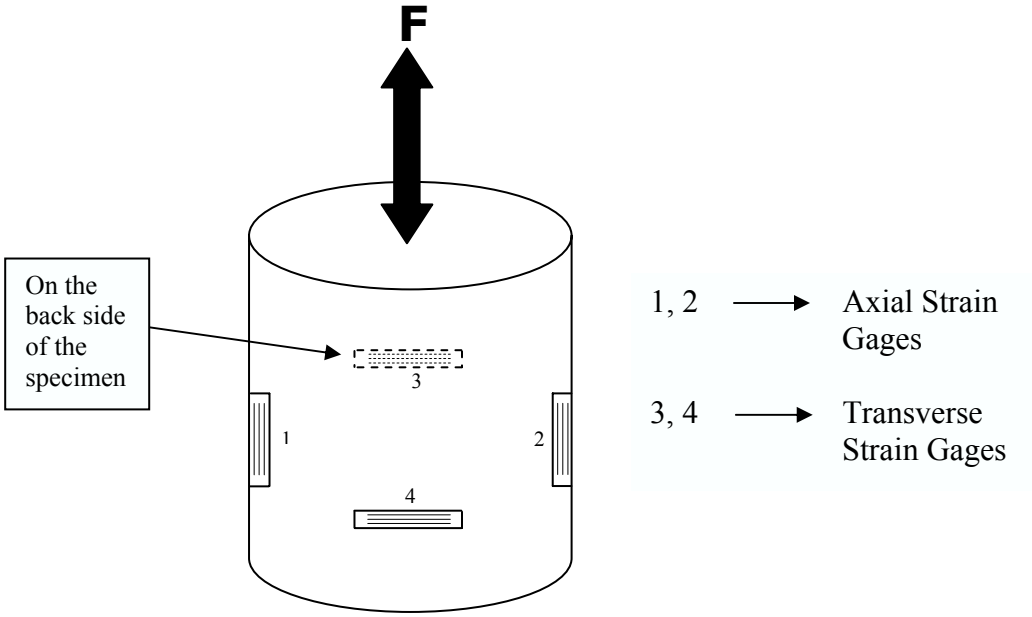


Figure 4.15 Strain Gage Locations on Stabilized Base Specimen

These gages were connected to the MEGADAC 5108 AC data acquisition system through an interface element called Screw Terminal Blocks (STB), which was connected to 810 MTS loading frame and 458.20 Micro Console. The MEGADAC has the capability to read sensor outputs at a rate up to 200 data points per second, four megabytes of memory, and a capacity of 80 input channels. An interface card, IEEE 488, allowed it to be controlled by a host computer using Test Control Software (TCS). The test data were collected during each test using the MEGADAC system, and the resulting data were saved on the host computer's hard drive.

Each specimen was subjected to repeated loading with an approximate square pulse waveform consisting of a 0.1 second load period and a 2.9 second rest period (Figure 4.16). Tests were carried out at a standard room temperature of about 77 °F (25 °C) for deviator stress varying from 5 psi to 30 psi (34.5 kPa to 206.8 kPa) with 5 psi (34.5 kPa) intervals. These specimens were tested at the room temperature only, because field thermocouple readings monitored at the Ohio-SHRP site indicated that the temperature within the stabilized base remained fairly constant at mild temperatures throughout the year. Each sample was preconditioned under 50 load cycles before the sensor readings were collected at a rate of 200 readings per second over a minimum of 5 load cycles. Once the test was completed, a number of graphical plots were generated. Figures 4.17, 4.18, and 4.19 present typical plots prepared from a single test.

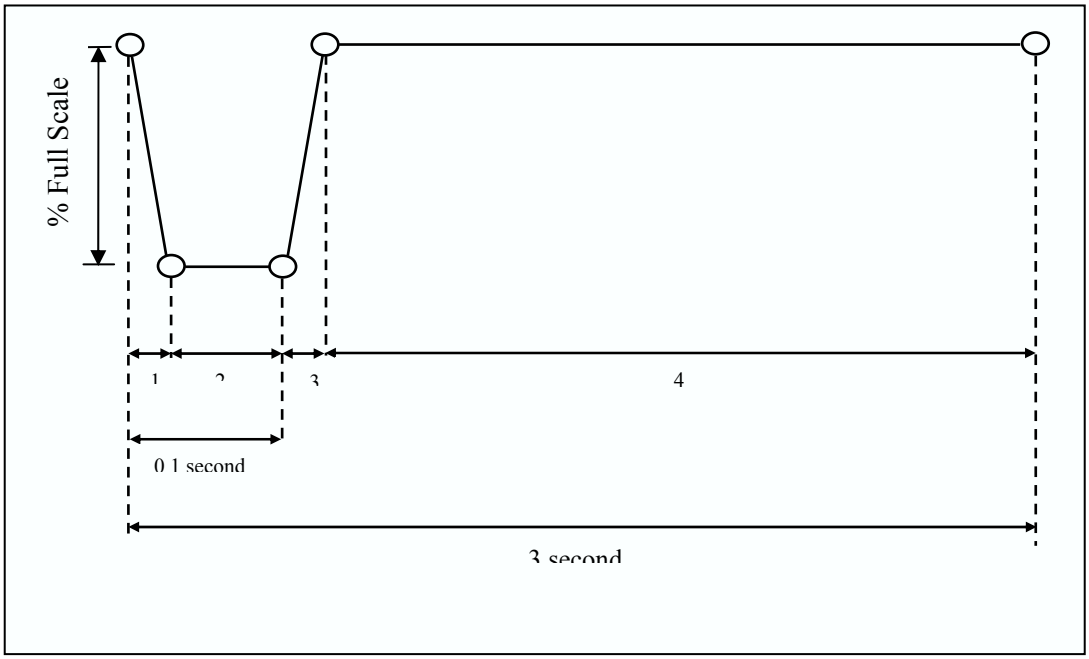


Figure 4.16 Approximate Square Pulse Waveform (Not to Scale)

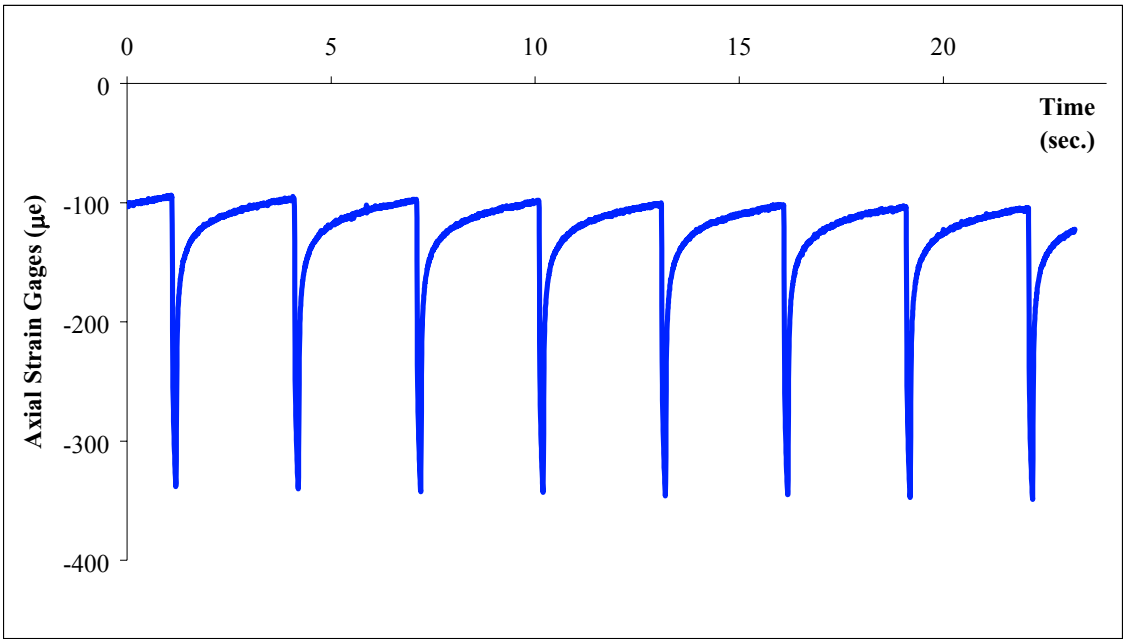


Figure 4.17 Typical Response of Axial Strain Gages During Alternative Test

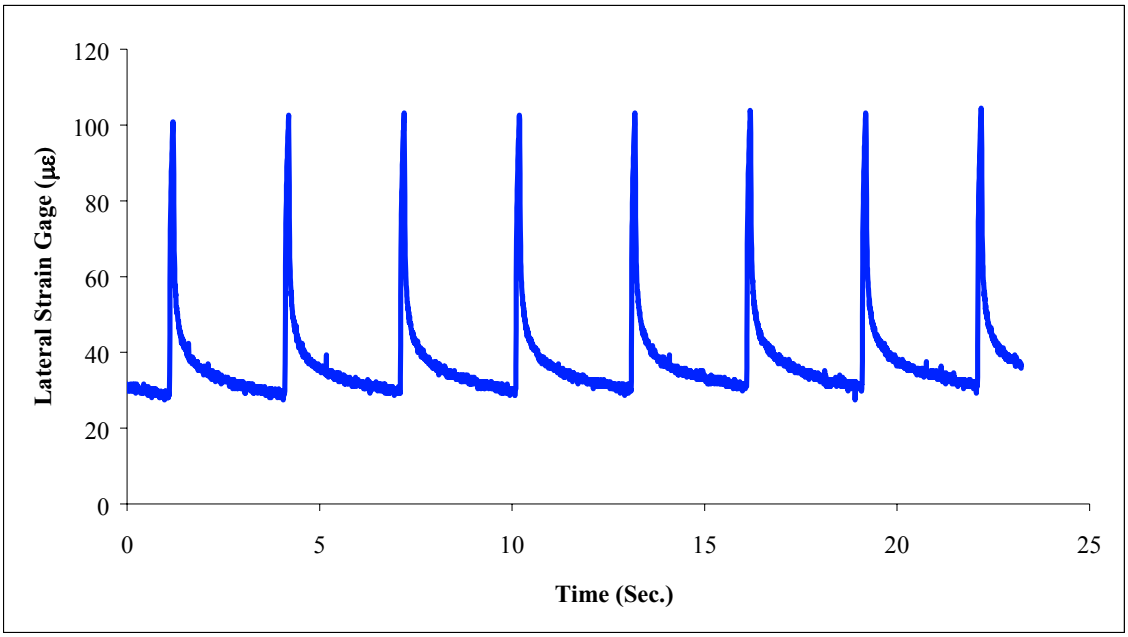


Figure 4.18 Typical Response of Transverse Strain Gages During Alternative Test

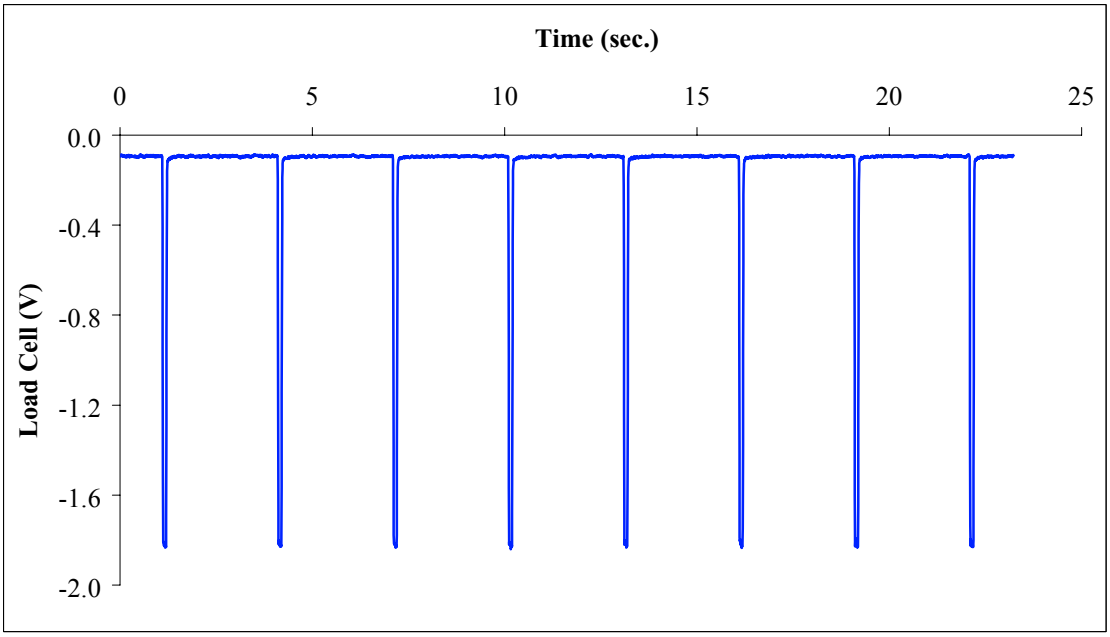


Figure 4.19 Typical Response of Load Cell

The data collected from each test were then analyzed by using the following equations:

$$M_r = \frac{\sigma}{\varepsilon_{axial}} \tag{4.15}$$

$$\mu = -\left(\frac{\varepsilon_{trans}}{\varepsilon_{axial}}\right) \tag{4.16}$$

where M_r = resilient modulus (psi); σ = axial stress (psi); ε_{axial} = axial strain; ε_{trans} = transverse strain; and μ = Poisson's ratio.

An obvious advantage of the alternative test method is that it simulates field loading conditions more directly, since each specimen is loaded in the same direction as in the field. A disadvantage of the alternative approach is that it cannot measure the resilient modulus of relatively thin samples and/or individual layer samples. The alternative approach also requires careful installation of multiple strain gages on each test specimen.

CHAPTER 5:
LABORATORY TEST RESULTS

5.1 Introduction

This chapter is dedicated to the presentation and discussion of the laboratory test results obtained for pavement materials from the Ohio-SHRP Test Road site. An effort is also made to integrate additional test results supplied by the ODOT and any other results obtained through nondestructive in-situ testing conducted in the field. Trends observed among the test results are pointed out whenever possible to enhance the understanding of how each pavement material behaves. In some cases, empirical relationships applicable to the test results are evaluated in light of the current test results. It will be very convenient if the pavement material properties that require expensive test set-ups and time-consuming procedures can be estimated relatively accurately from more basic material properties that can be measured more easily. The results are presented in a bottom-to-top sequence, from the subgrade soils to the base materials to the paving materials.

5.2 Laboratory Test Results on Subgrade Soils

Soil samples were utilized to measure the resilient modulus of each subgrade soil type. A total of fifteen bag samples of subgrade soil were recovered from the project site. Theses included six samples from the SPS-1 experiment sections (390106, 390107, 390108, 390110, 390111, and 390160), six samples from the SPS-2 experiment sections

(390202, 390205, 390207, 390209, 390211, and 390262), two samples from the SPS-8 experiment sections (390809 and 390810), and one sample from the SPS-9 experiment sections (390902). The in-situ moisture content and dry unit weight data were also obtained at a depth of 12 inches (304 mm) with a nuclear gauge over the entire site at the time of subgrade preparation work. Table 5.1 summarizes the average accepted, field moisture content and dry unit weight data obtained from the in-situ nuclear moisture-density tests.

Table 5.1 Average Accepted Field Moisture Content and Dry Unit Weight Data

Section	Dry Unit Weight pcf	Moisture Content %
390101	116.8	8.9
390102	124.6	8.3
390103	119.8	7.7
390104	119.7	9.2
390105	117.6	9.7
390106	124.4	9.8
390107	120.6	7.3
390108	117.8	8.1
390109	119.7	9.7
390110	118.6	9.5
390111	122.5	9.4
390112	121.9	8.7
390159	118.1	12.0
390160	123.4	8.3
390201	119.6	11.1
390202	124.2	10.5
390203	120.4	8.4
390204	125.9	9.2
390205	118.2	10.7
390206	120.0	10.1

Section	Dry Unit Weight pcf	Moisture Content %
390207	120.7	8.0
390208	115.2	9.3
390209	118.8	10.5
390210	116.1	9.6
390211	118.7	9.6
390212	126.0	9.2
390259	115.0	8.7
390260	121.4	11.6
390261	120.7	9.0
390262	117.8	8.8
390263	119.4	11.3
390265	121.9	8.6
390803	114.6	14.5
390804	113.8	13.2
390809	109.6	16.4
390810	117.4	11.7
390901	124.1	10.8
390902	122.3	10.8
390903	126.1	8.8

The ODOT (1997) specification for earthwork, states that the subgrade soil must be compacted to 100% of the maximum dry unit weight determined in the laboratory, when the maximum laboratory dry unit weight is within 105 pcf and 119.9 pcf (or within 1,684 kg/m^3 and 1,920 kg/m^3). For the soils encountered at the Ohio-SHRP Test Road, the maximum dry unit weight values are listed in Table 3.5, and the measured dry unit weight values can be found in Table 5.1. The percent relative compaction was calculated using the following formula:

$$\% \text{ Compaction} = \left(\frac{\text{Field Dry Unit Weight}}{\text{Maximum Laboratory Dry Unit Weight}} \right) \text{ X } 100 \qquad (5.1)$$

As seen in Table 5.2, all of the sections met the ODOT requirement for compaction. The percent relative compaction ranged between 101.37 and 108%, with an average of 104.7%. The field moisture contents varied between 7.3 and 10.8%, with an average of 9.05%. The ODOT specifications did not address any requirements for the compaction moisture contents.

The ODOT laboratory performed a series of grain size analysis and the Atterberg limit tests to classify the soils at the Ohio-SHRP Test Road. Table 5.3 summarizes the test results presented previously in Section 3.4.1. Three fine-grained soil groups of A-4 (moderately plastic silty soil), A-6 (plastic clay soil), and A-7-6 (clay soil with high plasticity index) were identified among the soil samples, according to the AASHTO Soil Classification System. The A-6 soil group appeared most extensively over the site, with the A-7-6 soil group found only in an isolated area.

Table 5.2 Assessment of Field Subgrade Compaction Work

Soil Type	Section	Max. Dry Unit Weight (Laboratory) (pcf)	Dry Unit Weight (Field) (pcf)	% Compaction	Moisture Content (%)	
					Field	Optimum
A-4	390110	117	118.6	101.37	9.5	13.5
	390160		123.4	105.47	8.3	
	390902		122.3	104.53	10.8	
A-6	390202	115	124.2	108.00	10.5	14.6
	390205		118.2	102.78	10.7	
	390207		120.7	104.96	8.0	
	390211		118.7	103.22	9.6	
	390262		117.8	102.39	8.8	
	390111		122.5	106.52	9.4	
A-7-6	390107	112	120.6	107.68	7.3	15.8

Table 5.3 Average Atterberg Limits of Subgrade Soil Types

Soil Type	Atterberg Limit	
A - 4	PL = 8.8	LL = 28.0
A - 6a	PL = 12.7	LL = 30.6
A - 6b	PL = 17.0	LL = 37.0
A - 7 - 6	PL = 26.5	LL = 44.5

The SHRP Protocol P46 for the resilient modulus of subgrade soils divides soils into two types; Type 1 (less than 70% passing No. 10 sieve, less than 20% passing No. 200 sieve); and Type 2 (more than 70% passing No. 10 sieve, more than 20% passing No. 200 sieve). From the grain size analysis, it was found that the subgrade soils at the project site could all be considered as Type 2 material. Therefore, relatively small size

specimens of 2.8 inches in diameter by 5.6 inches in height (7.1 cm in diameter by 14.2 cm in height) were utilized in the resilient modulus test program. The SHRP Protocol specifies each soil to be tested at a dry unit weight and moisture content similar to those accepted in the field. In the current study, each soil sample was tested at a minimum of three moisture contents, in order to examine any effects that moisture content has on resilient properties of the subgrade soils.

The resilient modulus was computed using Eq. 4.1, which depended on the deviator stress. The resilient modulus test results for the A-7-6 soil group samples are summarized in Table 5.4. The relationship between resilient modulus and moisture content for this soil type is presented in Figure 5.1.

Table 5.4 Summary of Resilient Modulus Test Results for A-7-6 Soil Group

Test No	Sample I.D	Dry Unit Weight (pcf)	Moisture Content Lab. (%)	Values of k	Values of n	Values of R^2	Resilient Modulus (psi) @ σ_d (psi) of: 2	4	6
15	390107	115.2	10.5	35,018.8	-0.475	0.982	25,198	18,131	14,956
16		114.7	13.5	30,983.8	-0.424	0.964	23,102	17,225	14,507
18		111.0	21.8	9,056.9	-0.971	0.777	4,622	2,358	1,591
20		116.2	16.1	17,624.8	-0.843	0.968	9,827	5,479	3,893
Maximum		116.2	21.8				25,198	18,131	14,956
Average		114.3	15.5				15,687	10,798	8,737
Minimum		111.0	10.5				4,622	2,358	1,591
Std. Deviation		2.3	4.8				10,037	8,054	6,988

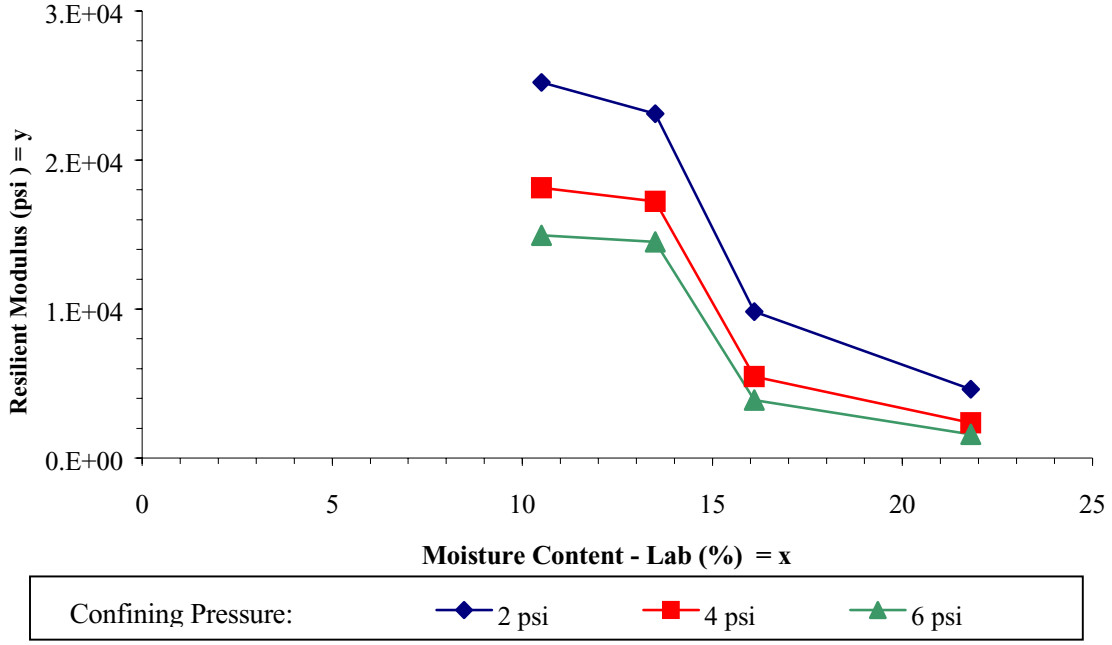


Figure 5.1 Resilient Modulus Vs. Moisture Content for A-7-6 Soil Group

Figure 5.1 indicates that the resilient modulus decreased as the moisture content increased, while the dry unit weight remained almost unchanged. The resilient modulus decreased by more than 80% under the deviator stress levels considered. The curve became flatter and shifted into a lower position as the deviator stress level increased.

Table 5.5 summarizes a similar set of resilient modulus test results obtained for the A-6 soil group samples. The moisture content of this soil type varied between 6.3 and 19.9% in the laboratory, while the dry unit weights remained relatively unchanged.

Figures 5.2 (a) through 5.2 (c) illustrate the relationship between moisture content and resilient modulus for this soil type under each confining pressure level. From these figures, it can be seen that the relationship between the moisture content and the resilient

Table 5.5 Summary of Resilient Modulus Test Results for A-6 Soil Group

Test No	Sample I.D	Dry Unit Weight (pcf)	Moisture Content Lab. (%)	Values of k	Values of n	Values of R^2	Resilient Modulus (psi) @ σ_d (psi) of: 2	4	6
52	390111	116.7	12.7	11,036.7	-0.382	0.788	8,472	6,503	5,570
53		106.0	19.7	22,568.0	-0.598	0.883	14,906	9,845	7,724
54		110.8	16.6	32,763.0	-0.836	0.907	18,350	10,277	7,322
33	390202	115.0	12.5	13,741.0	-0.278	0.837	11,332	9,345	8,349
34		115.7	18.3	9,192.1	-0.714	0.874	5,604	3,417	2,558
35		119.1	16.1	30,417.0	-0.607	0.869	19,975	13,117	10,257
36		120.2	10.0	35,346.9	-0.744	0.645	21,109	12,607	9,325
51	390205	113.6	16.1	42,035.7	-0.803	0.934	24,094	13,810	9,972
55		110.3	15.5	15,507.3	-0.503	0.889	10,940	7,717	6,292
56		110.1	15.1	52,553.0	-0.836	0.911	29,434	16,485	11,744
23	390207	111.4	17.6	26,958.0	-0.987	0.933	13,597	6,858	4,596
24		113.4	16.0	9,743.8	-0.630	0.964	6,295	4,067	3,150
25		117.3	14.1	16,448.7	-0.530	0.938	11,392	7,889	6,364
28		120.5	12.5	37,480.0	-0.653	0.909	23,841	15,165	11,639
1	390211	109.2	7.6	15,584.5	-0.445	0.946	11,450	8,412	7,024
9		112.7	11.5	18,659.7	-0.175	0.569	16,531	14,646	13,644
17		112.4	11.9	16,116.4	-0.304	0.760	13,057	10,578	9,353
19		112.3	18.4	7,541.9	-0.827	0.966	4,251	2,396	1,714
21		109.3	17.7	10,984.7	-0.696	0.972	6,779	4,183	3,154
22		111.3	10.4	19,115.7	-0.333	0.920	15,174	12,045	10,523
41	390262	114.1	15.0	33,228.0	-0.658	0.207	21,059	13,347	10,222
42		100.8	19.9	12,602.0	-0.991	0.633	6,340	3,190	2,134
43		123.6	6.3	37,180.7	-0.631	0.628	24,002	15,494	11,994
44		112.4	18.9	4,739.1	-0.502	0.513	3,346	2,362	1,927
Maximum		123.6	19.9				29,434	16,485	13,644
Average		113.3	14.6				14,222	9,323	7,356
Minimum		100.8	6.3				3,346	2,362	1,714
Std. Deviation		4.9	3.7				7,253	4,501	3,618

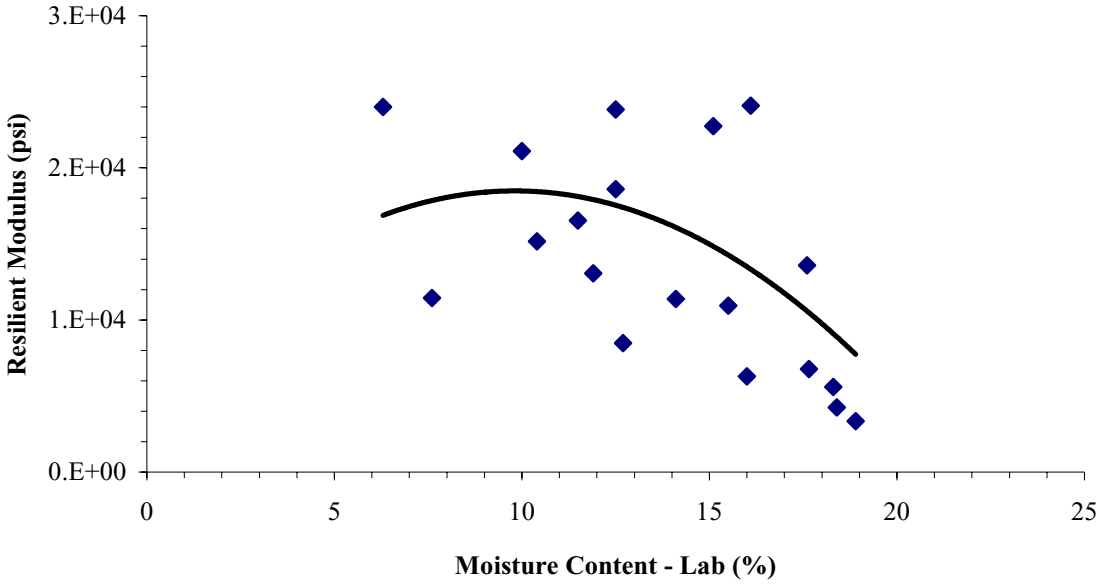


Figure 5.2 (a) Resilient Modulus Vs. Moisture Content for A-6 Soil Group at Confining Pressure of 2 psi

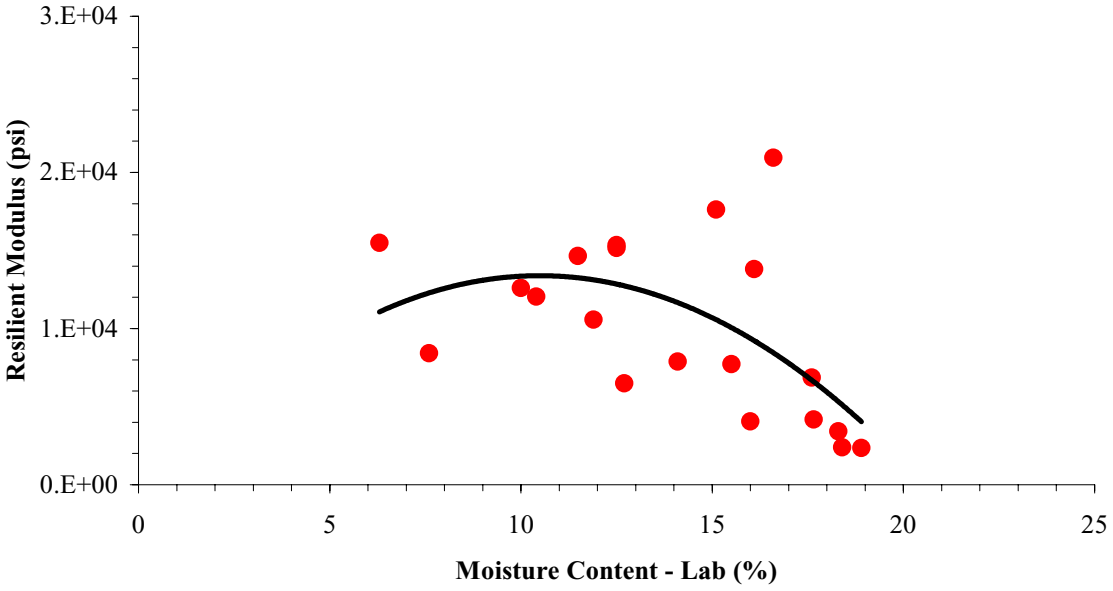


Figure 5.2 (b) Resilient Modulus Vs. Moisture Content for A-6 Soil Group at Confining Pressure of 4 psi

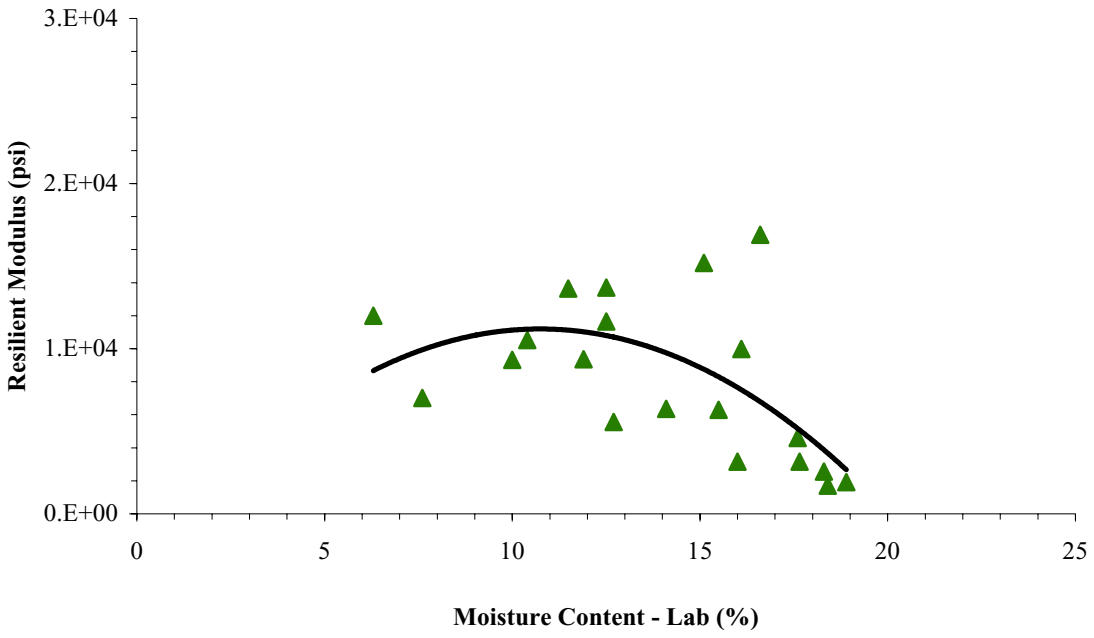


Figure 5.2 (c) Resilient Modulus Vs. Moisture Content for A-6 Soil Group at Confining Pressure of 6 psi

modulus is nonlinear and shaped like one side of a bell curve. The bell shape appeared since the resilient modulus generally decreased with the moisture content on the wet side of the optimum moisture content (OMC), and lower resilient modulus were measured at very low moisture content. This observation applies to the results obtained for the A-7-6 soil group. The bell-shape curve had a tendency to become flatter and move to a lower position as the deviator stress level increased.

Table 5.6 provides a summary of the resilient modulus test results for the A-4 soil group. The moisture content of the A-4 soil group varied between 10.45 and 21.5% in the laboratory, while the dry unit weights remained mostly within 112.8 pcf (17.71 kN/m^3). The relationship between the moisture content and resilient modulus for this soil group

was similar to those exhibited by the other soil groups, as seen in Figures 5.3 (a) to 5.3 (c).

Table 5.6 Summary Results of Resilient Modulus Test for A-4 Soil Group

Test No	Sample I.D	Dry Unit Weight (pcf)	Moisture Content Lab. (%)	Values of k	n	R^2	Resilient Modulus (psi) @ σ_d (psi) of: 2	4	6
57	390110	114.1	15.7	14,362.6	-0.4215	0.898	10,724	8,007	6,750
58		109.01	20.1	5,784.6	-0.4611	0.648	4,202	3,053	2,532
59		113.16	14.8	24,645.0	-0.7516	0.617	14,638	8,694	6,410
37	390160	112.98	13.8	68,914.4	-0.9488	0.872	35,703	18,496	12,590
38		114.92	18.05	5,742.2	-0.5442	0.723	3,938	2,700	2,166
39		121.28	11	52,428.0	-0.6157	0.735	34,215	22,329	17,396
40		118.6	13.9	10,177.9	-0.1933	0.406	8,902	7,785	7,199
10	390809	106.3	17.84	15,502.0	-0.2201	0.737	13,309	11,425	10,450
11		106.6	17.5	25,851.5	-0.3057	0.946	20,915	16,921	14,948
12		104.3	20	26,403.8	-0.5886	0.971	17,558	11,676	9,197
13		105.94	20.7	23,585.0	-0.4681	0.930	17,050	12,326	10,196
3	390810	105	14.5	23,635.9	-0.3301	0.973	18,802	14,957	13,084
4		104.9	14.8	23,478.0	-0.3448	0.941	18,488	14,558	12,659
5		110.41	17.25	30,784.0	-0.4341	0.995	22,785	16,865	14,143
6		106.8	21.3	26,830.9	-0.6961	0.897	16,562	10,223	7,709
7		106.8	21.2	18,494.1	-0.4141	0.976	13,880	10,417	8,807
8		106.6	21.5	11,178.7	-0.1963	0.625	9,757	8,516	7,864
26	390902	119.2	15	16,506.0	-0.2300	0.621	14,074	12,000	10,931
27		119.8	15.4	11,814.8	-0.5437	0.613	8,105	5,560	4,460
29		118.29	19.4	7,501.5	-0.6384	0.861	4,819	3,096	2,390
30		119.7	12.9	12,103.6	-0.2437	0.183	10,222	8,634	7,821
31		125.9	14.4	19,343.8	-0.2211	0.481	16,595	14,237	13,016
32		123.3	10.45	15,156.0	-0.1925	0.270	13,263	11,606	10,735
Maximum		125.9	21.5				35,703	22,329	17,396
Average		112.8	16.6				14,521	10,587	8,894
Minimum		104.3	10.5				3,938	2,700	2,166
Std. Deviation		6.8	3.3				8,494	5,417	4,446

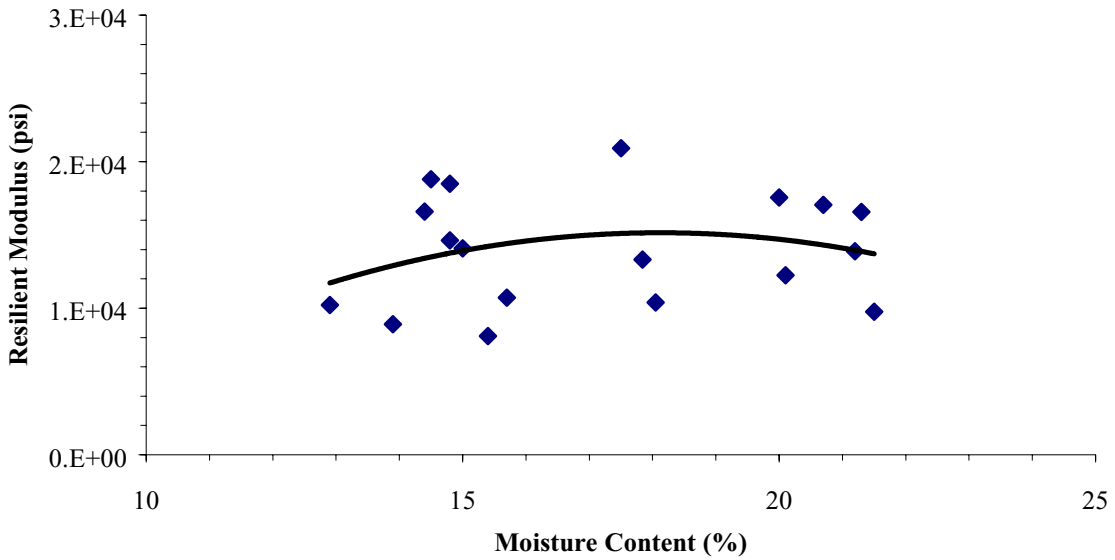


Figure 5.3 (a) Resilient Modulus Vs. Moisture Content for A-4 Soil at Confining Pressure of 2 psi

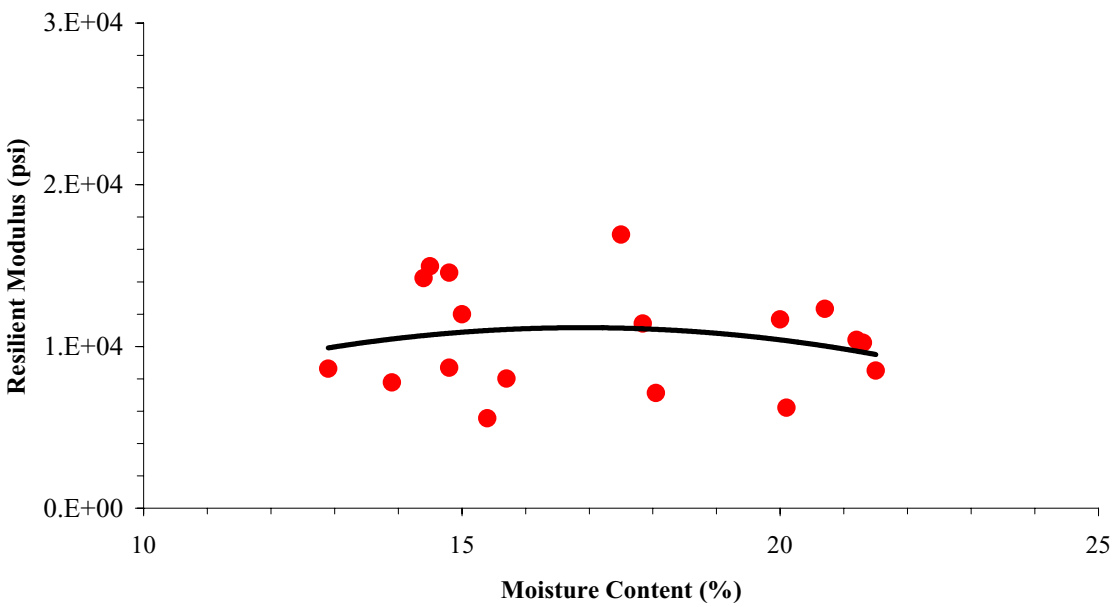


Figure 5.3 (b) Resilient Modulus Vs. Moisture Content for A-4 Soil at Confining Pressure of 4 psi

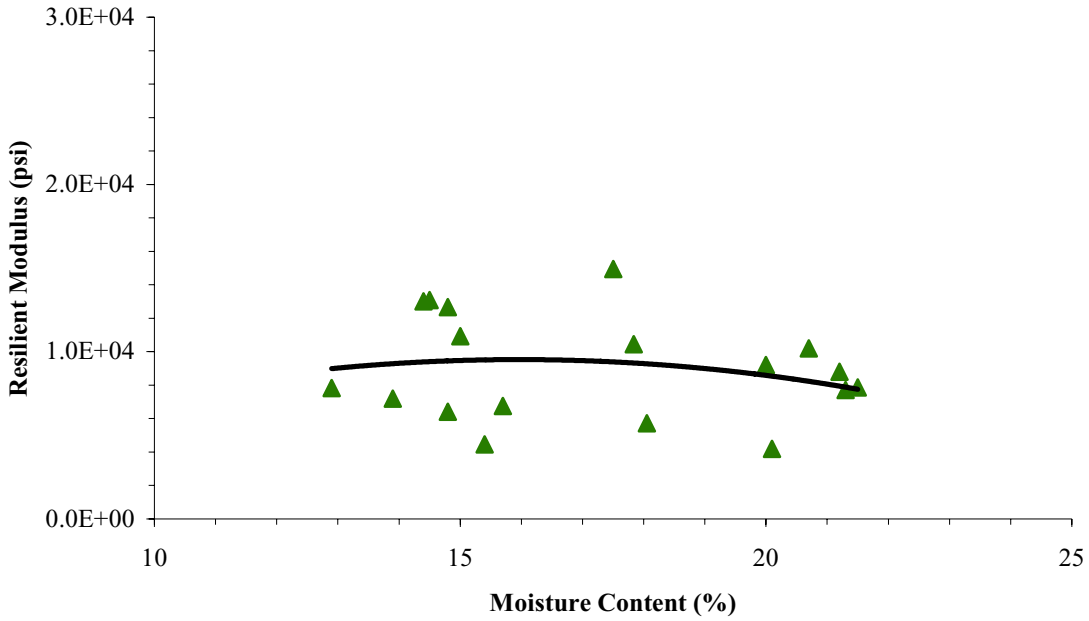


Figure 5.3. (c) Resilient Modulus Vs. Moisture Content for A-4 Soil at Confining Pressure of 6 psi

Overall, the laboratory test results for the three soil groups indicated that the resilient properties for soils containing higher clay contents were more sensitive to changes in the moisture content. The resilient behavior can be analyzed in detail by examining the data presented in Tables 5.4 through 5.6. The data supplied by Test Nos. 15 and 16 in Table 5.4 illustrate that the resilient modulus of the A-7-6 soil group declined as the moisture content increased, while the dry unit weight remained constant. The same trend was observed for Test Nos. 1 and 21; 9 and 44; 17 and 4; and 22 and 23 listed in Table 5.5. Additional support for this trend was also provided by comparing the results between Test Nos. 3 and 4; Test Nos. 6 and 11; Test Nos. 7 and 11; Test Nos. 8 and 11; Test Nos. 26 and 58; and Test Nos. 37 and 59 (shown in Table 5.6).

The A-6 soil group also shows a consistent trend, that an increased dry unit weight leads to a higher resilient modulus at low moisture content. Test Nos. 21 and 23; 22 and 36; 28 and 33; and 24 and 35 presented in Table 5.5 support this trend. The A-4 soil type did not exhibit the same trend as that of the other two soil types.

Falling Weight Deflectometer (FWD) test data were used to obtain surface deflection data in the field (Wasniak, 1999). The tests were conducted at the completion of the subgrade preparation. The maximum, minimum, and mean modulus of subgrade soils in the field for each northbound and southbound section was analyzed by using the Falling Weight Deflectometer (FWD) data, summarized in Tables 5.7 and 5.8, respectively. The formula used to backcalculate the modulus of subgrade soil was based on the Boussinesq solution for a circularly loaded rigid area;

$$E = \left(\frac{\pi \mu r^2 (1 - \ ^2) P}{2\ D} \right) \tag{5.2}$$

where E = average elastic modulus of subgrade; r = radius of loaded area; μ = Poisson's ratio of subgrade soil; P = applied pressure; and D = deflection measured at the surface.

The mean modulus, standard deviations, and coefficient of variation for each lane and for the entire lane are tabulated in Table 5.9.

Table 5.7 Summary of Statistical Analysis on FWD Test Data on Subgrade Soil for Each Northbound Section (SPS-2) (Wasniak, 1999)

SHRP Section No.	Moisture Content (%)	Dry Unit Weight (pcf)	AASHTO Soil Type	Elastic Modulus (psi) of Subgrade Soil:				
				Minimum	Maximum	Mean	Standard Deviation	Coeff. of Variation
390201	11.1	119.6		2,698	16,971	9,051	4,149	0.459
390202	10.5	124.2	A-6	3,438	36,989	17,900	10,154	0.567
390203	8.4	120.4	A-6	8,529	24,935	14,941	4,091	0.274
390204	9.2	125.9	A-6	10,052	59,356	29,780	13,838	0.465
390205	10.7	118.2	A-6	2,451	21,990	9,327	5,381	0.576
390206	10.1	120.0	A-6	2,538	24,122	12,736	6,687	0.524
390207	8.0	120.7	A-6	9,762	26,675	17,087	5,251	0.307
390208	9.3	115.2	A-6	9,951	32,231	16,348	5,657	0.346
390209	10.5	118.8		2,509	25,587	10,386	7,847	0.756
390210	9.6	116.1		3,148	18,930	10,313	4,555	0.441
390211	9.2	118.7	A-6	8,935	21,062	15,854	3,075	0.194
390212	9.2	126.0		9,153	36,437	20,438	7,108	0.348
390259	8.7	115.0		2,974	19,655	11,459	4,917	0.429
390260	11.6	121.4		3,525	28,518	14,723	6,034	0.409
390261	9.0	120.7	A-6	3,351	33,159	18,001	6,368	0.353
390262	8.8	120.0	A-6	6,005	32,260	15,637	6,179	0.396
390263	11.3	119.4	A-6	3,829	25,645	13,592	6,194	0.455
390264	12.4	115.5		2,495	12,649	4,975	2,292	0.459
390265	8.6	121.9		9,559	16,275	12,866	2,654	0.207

Table 5.8 Summary of Statistical Analysis on FWD Test Data on Subgrade Soil for Each Southbound Section (SPS-1 and SPS-9) (Wasniak, 1999)

SHRP Section No.	Moisture Content (%)	Dry Unit Weight (pcf)	AASHTO Soil Type	Elastic Modulus (psi) of Subgrade Soil:				
				Minimum	Maximum	Mean	Standard Deviation	Coeff. of Variation
390101	8.9	116.8	A-7-6	4,091	22,570	11,691	5,817	0.498
390102	8.3	124.6		7,905	34,059	20,380	8,457	0.415
390103	7.7	119.8	A-6	5,875	24,964	15,695	4,381	0.279
390104	9.2	119.7	A-6	8,138	33,188	16,855	7,064	0.419
390105	9.7	117.6	A-6	10,806	22,701	15,550	3,307	0.213
390106	9.8	124.4		6,629	27,647	17,885	5,933	0.332
390107	7.3	120.6	A-7-6	7,949	28,329	16,768	5,715	0.341
390108	8.1	117.6		11,880	29,780	18,959	6,382	0.336
390109	9.7	119.7	A-4	3,916	27,038	11,517	5,686	0.493
390110	9.5	118.6	A-4	4,874	23,107	12,953	5,440	0.420
390111	9.4	122.5	A-6	3,989	36,916	18,088	8,993	0.497
390112	8.7	121.9	A-6	2,988	28,431	13,824	6,295	0.455
390159	11.3	118.9		2,074	12,228	5,773	3,191	0.554
390160	8.3	123.4	A-4	10,516	30,519	18,639	5,599	0.300
390901	10.8	124.1	A-4	9,037	61,387	26,980	14,447	0.536
390902	10.8	122.3		4,845	32,231	15,506	6,934	0.447
390903	8.8	126.1		7,064	31,259	14,331	5,962	0.416

The subgrade modulus computed for the 19 test sections on the northbound side of the project site (SPS-2) ranged from 5 to 29.8 ksi (34.3 to 205.3 MPa), with an average of 14.6 ksi (100.8 MPa) as shown in Tables 5.7 and 5.9.

Table 5.9 Mean Modulus, Standard Deviation, and Coefficient of Variation of Modulus Computed from the FWD Data (Wasniak, 1999)

Location	Minimum Modulus (psi)	Maximum Modulus (psi)	Mean Modulus (psi)	Standard Deviation (psi)	Coefficient of Variation
Northbound (SPS-2)	2,451	59,356	14,621	8,297	0.567
Southbound (SPS-1 and SPS-9)	2,074	61,387	15,796	8,138	0.515
Entire Project	2,074	61,387	15,173	8,239	0.543

From Tables 5.8 and 5.9, it can be seen that the average subgrade modulus calculated for the 17 test sections of the southbound side had a range of 5.8 to 27 ksi (39.8 to 186.0 MPa), with an overall average of 15.8 ksi (109.1 MPa). The sections having the lowest modulus on each side of the pavement were constructed one year later than the other sections. It was noted that these sections had much higher moisture contents at the time Falling Weight Deflectometer tests were conducted.

Although the moisture and dry unit weight conditions of the A-7-6 soil group did not match well between Tables 5.4 and 5.8, the ranges of resilient modulus were similar between the laboratory results (3.9 to 25.2 ksi) and the field test results (4.1 to 28.3 ksi). For the A-6 soil group, the ranges of resilient modulus were 1.7 to 29.4 ksi in the laboratory (Table 5.5) and 2.5 to 59.4 ksi in the field (Tables 5.7 & 5.8). Similarly, for the A-4 soil group, the ranges of resilient modulus were 2.2 to 35.7 ksi in the laboratory (Table 5.5) and 3.9 to 61.4 ksi in the field (Table 5.8). Overall, a good correlation can be seen between the range of resilient moduli measured in the laboratory and the range of resilient moduli backcalculated from the in-situ Falling Weight Deflectometer (FWD) test data. The slightly wider range measured in the field is believed to be due to larger

fluctuations in the in-situ moisture content/dry unit weight conditions, the fact that the soils were compacted to denser states by the sheepsfoot roller (promoting kneading action), and the fact that a larger volume was tested in the FWD test method. Point by point comparisons between the laboratory and field test results are not possible due to mismatches in the dry unit weight and compaction method and the fact that the confining pressure was not measured in the field.

The condition of the subgrade at the time of the FWD tests was relatively dry, and the stiffness calculated afterward was expected to be higher than the average design values assumed for the entire year, except for the two sections with uncommonly low moduli. The in-situ moduli were expected to drop significantly, as suggested by the laboratory test results as the base and pavement layers were added, and as moisture migrated into the subgrade. These moduli may decrease below the assumed value of 7.2 ksi (49.6 MPa), as per ODOT Design Manual, which could result in a reduced service life for these pavement sections.

5.3 Laboratory Test Results on Unbound Base Materials or Dense Graded Aggregate Base (DGAB)

Several bags of unbound base material or DGAB used in the Ohio-SHRP project site were brought to the ORITE laboratory to determine engineering properties. All of the unbound materials were taken from the SPS-1 experiment sections (390101, 390102).

Mechanical sieve analysis tests were performed prior to the resilient modulus testing of the unbound base materials. Results of the grain size analysis can be found in

the attached database. As mentioned previously in Chapter 4, the procedures employed during the resilient modulus testing of granular material was similar to that used during the resilient modulus testing of the subgrade soils. The material was compacted in 6 lifts in a large split mold with dimensions of 6 inches in diameter by 12 inches in height (15.2 cm diameter by 30.5 cm height), using an impact hammer. The resilient modulus test results are summarized in Table 5.10.

Table 5.10 Summary of Resilient Modulus Test Results on Unbound Base Material

Test #	Moist Unit Wt. (pcf)	Dry Unit Wt. (pcf)	w (%)	Permanent Strain (%)	K (psi)	n	R^2	Resilient Modulus (psi) at Confining Pressure of:				
								3 psi	5 psi	10 psi	15 psi	20 psi
1	116.6	114.7	1.7	0.72	6,294	0.27	0.557	8,463	9,713	11,708	13,061	14,114
2	116.4	114.0	2.1	0.56	5,619	0.31	0.342	7,856	9,181	11,344	12,837	14,015
3	121.0	115.2	5.1	0.49	3,402	0.43	0.788	5,451	6,787	9,137	10.874	12,303
4	121.6	113.9	6.8	0.80	4,320	0.34	0.658	6,272	7,460	9,439	10,832	11,944
Ave.	118.9	114.5	3.9	0.64	4,909	0.34	0.586	7,011	8,285	10,407	11,901	13,094

[Note] w = Moisture content.

The resilient modulus of the unbound base material was calculated using Equation 4.1. Resilient moduli increased with increased deviatoric stress. The resilient modulus increased by about 15.4, 20.4, 12.6, 9.1% for deviator stress rising from 3 to 5, 5 to 10, 10 to 15, and 15 to 20 psi (20.68 to 34.5, 34.5 to 69, 69 to 103.5 and 103.5 to 137.9 kPa), respectively.

The resilient modulus of unbound base materials also increased as the confining pressure increased. Moisture content varied between 1.7 and 6.8%, with an average of

3.9%. The unbound base material was basically free draining and could not retain high moisture contents. It was noted that the moisture content affected the cumulative permanent strain recorded in the axial direction under the repeated loading. Moisture content was also discovered to affect the resilient modulus. The resilient modulus of the unbound granular materials increased as the permanent strain increased, and the granular material became more compacted. Figure 5.4 (a) depicts the relationship between the resilient modulus and the bulk stress obtained by applying a power regression equation. The bulk stress is defined as the sum of all principal stresses, which is expressed mathematically as:

$$\theta = \sigma_d + 3\sigma_3 \tag{5.3}$$

where θ = bulk stress or first stress invariant (psi); σ_d = deviator stress (psi); σ_3 = confining pressure (psi).

A correlation was established between the resilient modulus and the bulk stress instead of the resilient modulus and deviatoric stress; since the resilient modulus of granular base material is known to depend more on the sum of principal stresses than on the deviator stress (Lekarp, 2000). From Figures 5.4 (a) and (b), it can be seen that the relationship between the resilient modulus and the bulk stress is somewhat strong with a correlation coefficient (R) of 0.82, while the relationship between the resilient modulus and the deviator stress is slightly weaker with a correlation coefficient (R) of 0.71.

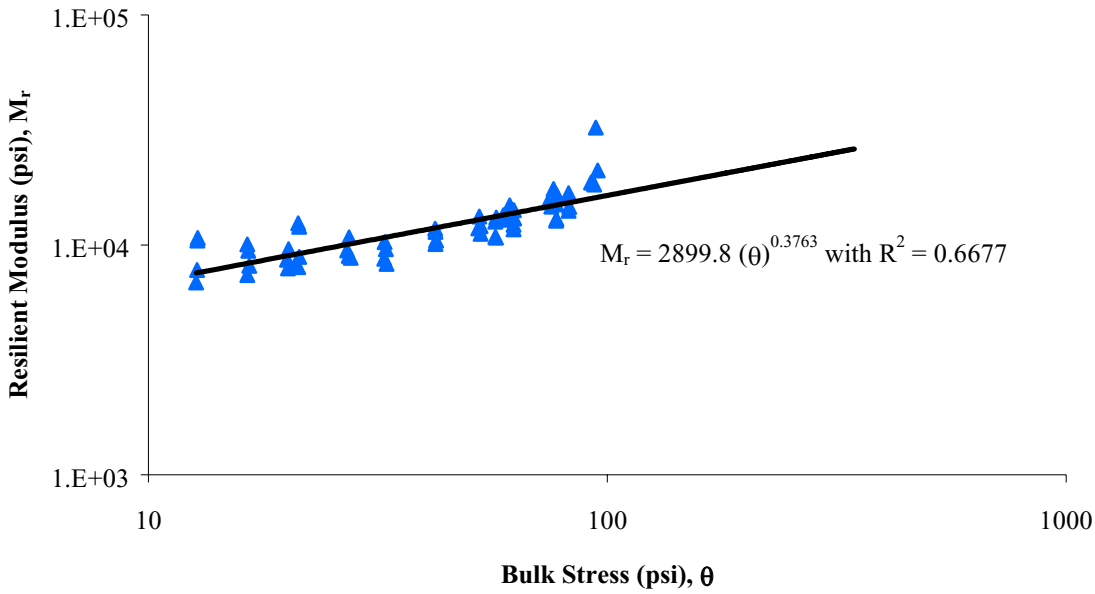


Figure 5.4 (a) Resilient Modulus Vs. Bulk Stress for Unbound Base

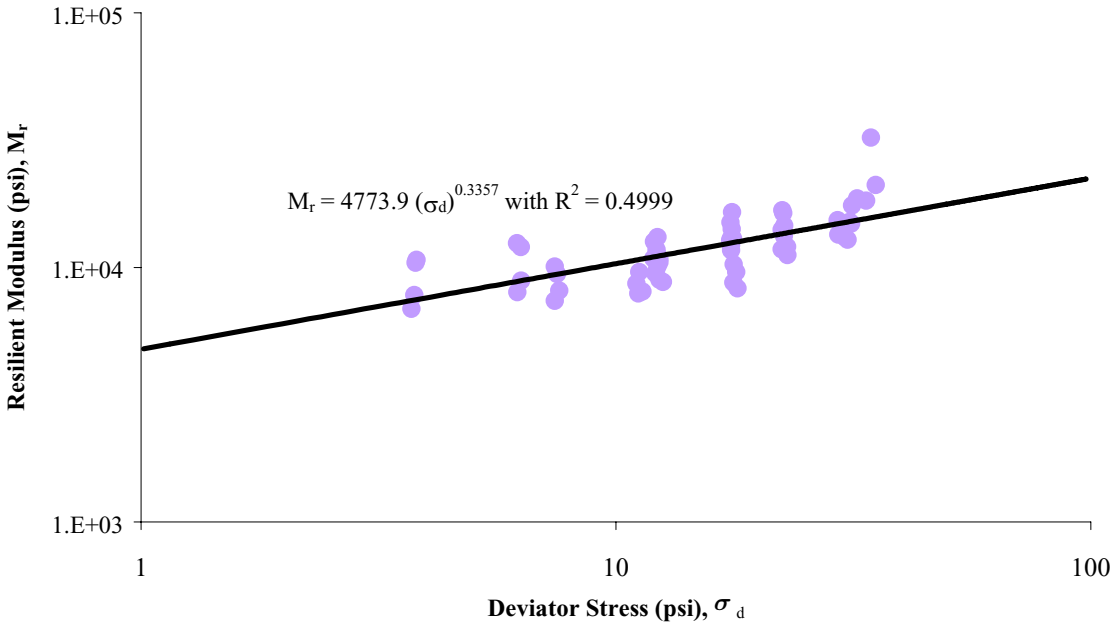


Figure 5.4 (b) Resilient Modulus Vs. Deviator Stress for Unbound Base

5.4 Laboratory Test Results on Stabilized Base Materials

U.S. Rt. 23 was constructed with four different types of stabilized base materials: asphalt-treated base (ATB), permeable cement-treated base (PCTB), lean concrete base (LCB), and permeable asphalt-treated base (PATB). The following sections describe the laboratory test results obtained for each of these materials in the ORITE laboratory.

5.4.1 Asphalt-Treated Base (ATB)

Twenty core specimens of ATB taken from nine different locations within the Ohio-SHRP Test Road, were tested in the ORITE laboratory. Resilient modulus and indirect tensile strength tests were performed on eighteen of the core specimens, per the SHRP Protocol P07, while the other two were tested per the dynamic modulus test (ASTM D-3497). The specimens used for the resilient modulus and indirect tensile strength tests were carefully cut with a large circular saw to fulfill the dimensional requirement in the SHRP protocol, which was 4 inches (101.6 mm) in diameter by 2 inches (50.8 mm) in thickness. These specimens were taken from the SPS-1 (390105, 390106, 390112, 390161, 390162, and 390163) and the SPS-9 (390901, 390902, and 390903) experiments at a sampling frequency of two per section.

Prior to the resilient modulus and indirect tensile strength tests, the bulk specific gravity tests were conducted on the specimens according to the AASHTO T166-93 Method A, as mentioned in Chapter 4. The bulk specific gravity of the specimens was calculated using Equation 4.6. The results of the bulk specific gravity of asphalt-treated base are presented in Table 5.11. Within the SPS-1 experiment, the bulk specific gravity

ranged between 2.232 and 2.317, with an average of 2.287. Within the SPS-9 experiment, the bulk specific gravity ranged between 2.269 and 2.299, with an average of 2.286.

Table 5.11 Bulk Specific Gravity of Asphalt-Treated Base Core Specimens

SHRP ID	BSG	SHRP ID	BSG
390161	2.302	390112	2.275
390162	2.317	390901	2.290
390163	2.293	390902	2.299
390105	2.303	390903	2.269
390106	2.232	390112	2.267
		390902	2.258

The procedure for performing the resilient modulus test for the ATB specimens can be found in the SHRP P07 (1993). Each specimen was subjected to a repeated haversine pulse shape loading over about 50 cycles. The Poisson's ratio and resilient modulus were calculated using Equations 4.7 and 4.8 respectively, by averaging the sensor output readings collected over the last five cycles. Tables 5.12 and 5.13 present a summary of the original and adjusted resilient modulus results. The resilient modulus values having a Poisson's ratio either less than 0.1 or greater than 0.5 (referred to as the original values) were modified, and can be seen in bold in Table 5.13. The relationship between the temperature and resilient modulus for ATB is shown in Figures 5.5 and 5.6.

Table 5.12 Average Original Resilient Modulus Results on Asphalt-Treated Base Core Specimens

SHRP ID	Ave. Original Resilient Modulus (psi) @			Ave. Original Poisson's Ratio (μ) @		
	5°C (41°F)	25°C (77°F)	40°C(104°F)	5°C (41°F)	25°C (77°F)	40°C(104°F)
390161	**6.418E+05**	4.096E+05	**2.396E+05**	0.03	0.31	0.76
390162	8.009E+05	5.112E+05	**2.723E+05**	0.10	0.37	0.60
390163	8.715E+05	5.346E+05	**2.457E+05**	0.11	0.42	0.75
390105	**6.519E+05**	6.022E+05	3.136E+05	-0.03	0.26	0.48
390106	**8.477E+05**	5.805E+05	**2.900E+05**	0.03	0.19	0.51
390112	**7.647E+05**	7.887E+05	3.214E+05	-0.04	0.28	0.44
390901	**6.047E+05**	5.692E+05	2.970E+05	-0.11	0.11	0.33
390902	**8.000E+05**	6.092E+05	2.911E+05	0.01	0.21	0.44
390903	**6.366E+05**	6.330E+05	3.074E+05	-0.03	0.20	0.48

Table 5.13 Average Adjusted Resilient Modulus Results on Asphalt-Treated Base Core Specimens

SHRP ID	Ave. Adjusted Resilient Modulus (psi) @			Ave. Adjusted Poisson's Ratio (μ)		
	5°C (41°F)	25°C (77°F)	40°C(104°F)	5°C (41°F)	25°C (77°F)	40°C(104°F)
390161	**8.143E+05**	4.096E+05	**1.801E+05**	0.10	0.31	0.50
390162	8.273E+05	5.112E+05	**4.590E+05**	0.11	0.37	0.50
390163	8.832E+05	5.346E+05	**1.890E+05**	0.12	0.42	0.50
390105	**1.018E+06**	6.022E+05	2.862E+05	0.10	0.26	0.41
390106	**1.064E+06**	5.805E+05	**2.880E+05**	0.10	0.19	0.50
390112	**1.207E+06**	7.887E+05	3.214E+05	0.10	0.28	0.44
390901	**1.400E+06**	6.053E+05	2.970E+05	0.10	0.13	0.33
390902	**1.072E+06**	6.092E+05	2.895E+05	0.10	0.21	0.43
390903	**9.570E+05**	6.330E+05	3.074E+05	0.10	0.20	0.48

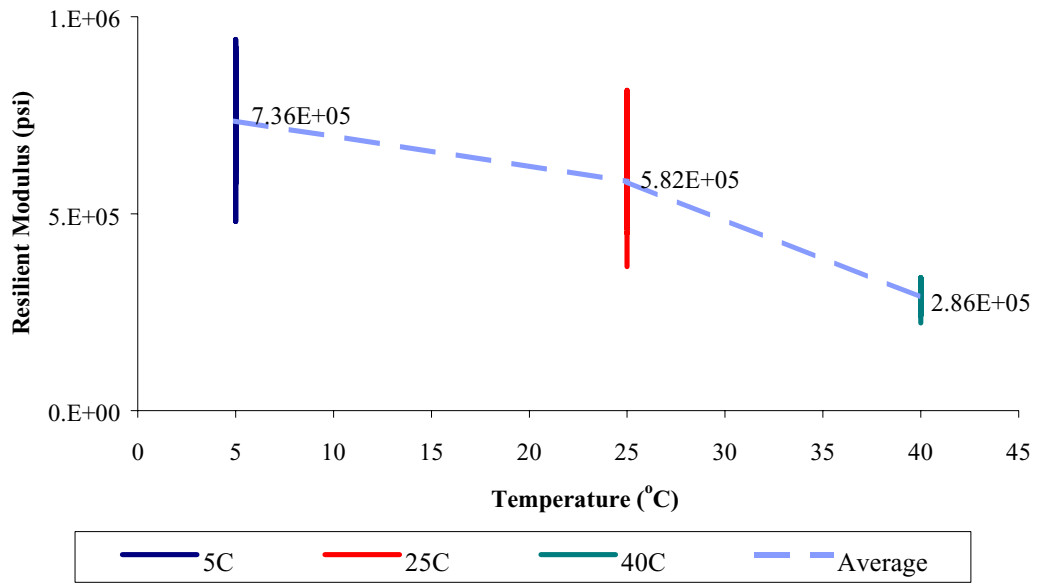


Figure 5.5 Resilient Modulus Vs. Temperature of Asphalt-Treated Base Core Specimens (Original)

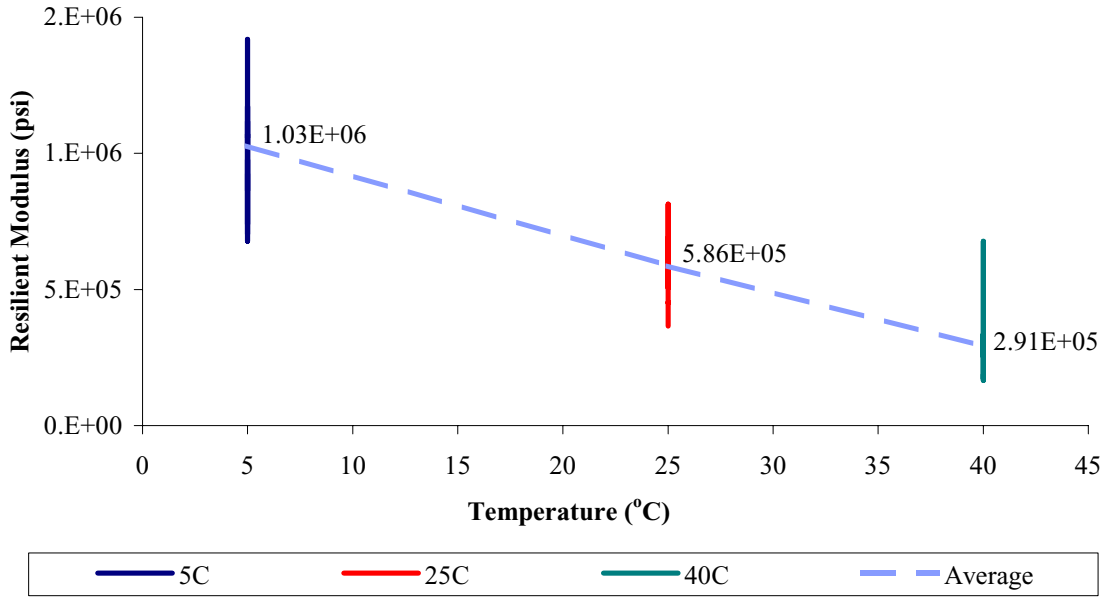


Figure 5.6 Resilient Modulus Vs. Temperature of Asphalt-Treated Base Core Specimens (Adjusted)

Figures 5.5 and 5.6 clearly indicate that temperature plays a significant role in the magnitude of the resilient modulus, which decreases as temperature increases. The average original resilient modulus was reduced by approximately 21% as temperature changed from 41 to 77 °F (5 to 25 °C) in the overall range of 0.37 to 0.94 million psi (2.52 to 6.5 GPa). The resilient modulus decreased further by approximately 51% from 0.81 to 0.22 million psi (5.6 to 1.5 GPa) as the temperature rose from 77 to 104 °F (25 to 40 °C).

In terms of the modified resilient modulus values, the reduction in resilient modulus was 43 and 50% for the temperature changes from 41 to 77 °F (5 to 25 °C) and from 77 to 104 °F (25 to 40 °C), respectively. It can be seen that the temperature effect on the magnitude of resilient modulus was greater over the original resilient modulus values than over the adjusted resilient modulus values. This was due to the fact that only two of the original resilient modulus did not need to be modified and that the resilient modulus typically increased after the Poisson's ratio adjustment.

As seen in Table 5.12, only two of the Poisson's ratios at 41 °F (5 °C) were greater than 0.1. This occurred since the mixture was very stiff at the lower temperature and the ratio between the recoverable horizontal and vertical deformation was very low. Similarly, some of the Poisson's ratios measured at 104 °F (40 °C) were greater than 0.5, since the mixture became softer and caused the ratio between the recoverable horizontal and vertical deformation to be high.

After the resilient modulus tests were concluded, all of the specimens were restored in the environmental chamber for about 24 hours at 25 ± 1 °C (77 ± 2 °F). The

specimens were then subjected to the indirect tension strength test. Each specimen was subjected to a compressive load along the diametrical axis at a fixed deformation rate until the specimen failed. The indirect tensile strength was then calculated using Equation 4.9 (Table 5.14).

From Table 5.14, it is apparent that the specimen from Section 390901 had the highest indirect tensile strength, while Section 390161 had the lowest. Three sections, 390161, 390162, and 390163 were found to have relatively low indirect tensile values. The overall average indirect tension strength was 136.9 psi (943.9 kPa) among the nine sections having the asphalt-treated base (ATB).

Table 5.14 Indirect Tensile Strength Test Results on Asphalt-Treated Base Core Specimens

SHRP ID	Indirect Tensile Strength (psi)
390105	148.882
390106	134.516
390112	166.406
390161	81.041
390162	98.763
390163	90.800
390901	198.726
390902	144.338
390903	168.464

The relationship between the indirect tensile strength and the resilient modulus for asphalt-treated base (ATB), Figure 5.7, shows a considerable degree of data scattering.

Some of the specimens did not meet the size requirement of the SHRP P07 Protocol in terms of thickness or diameter. This imperfect dimension may have caused high stress values, which might have also caused high resilient modulus. This was the primary justification for eliminating several data points. After the outlying data points were eliminated, a better relationship was observed between the two variables (Figure 5.8). Figure 5.8 illustrates that the relationship between the indirect tensile strength and the resilient modulus was quite strong with a correlation coefficient (R) of 0.86. This value shows that the resilient modulus of asphalt-treated base (ATB) materials estimated from the indirect tensile strength was relatively accurate.

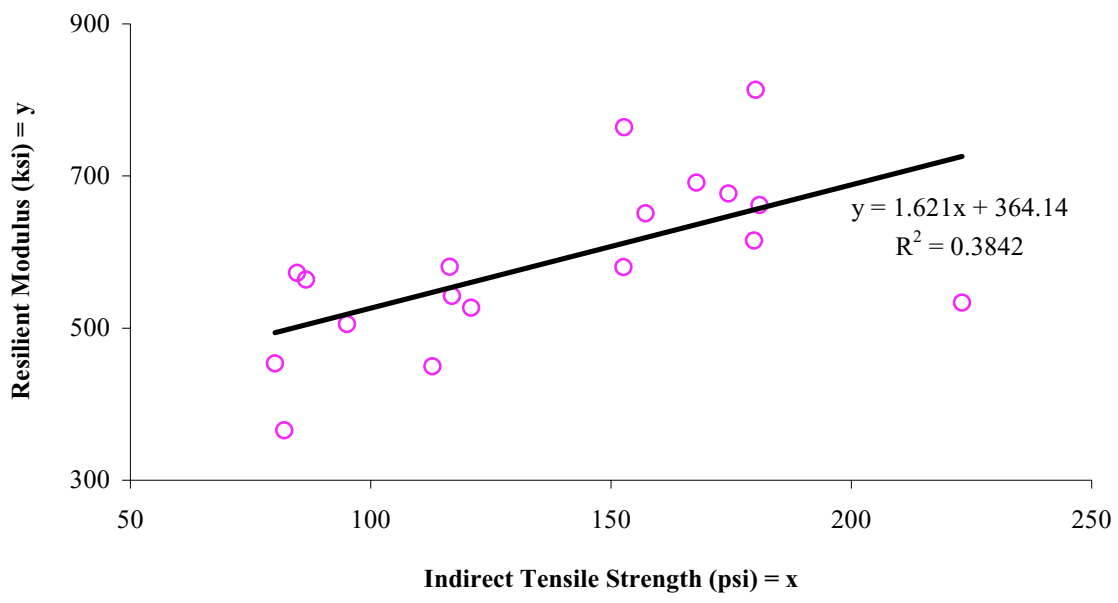


Figure 5.7 Resilient Modulus Vs. Indirect Tensile Strength for Asphalt-Treated Base Core Specimens (Before Adjustment)

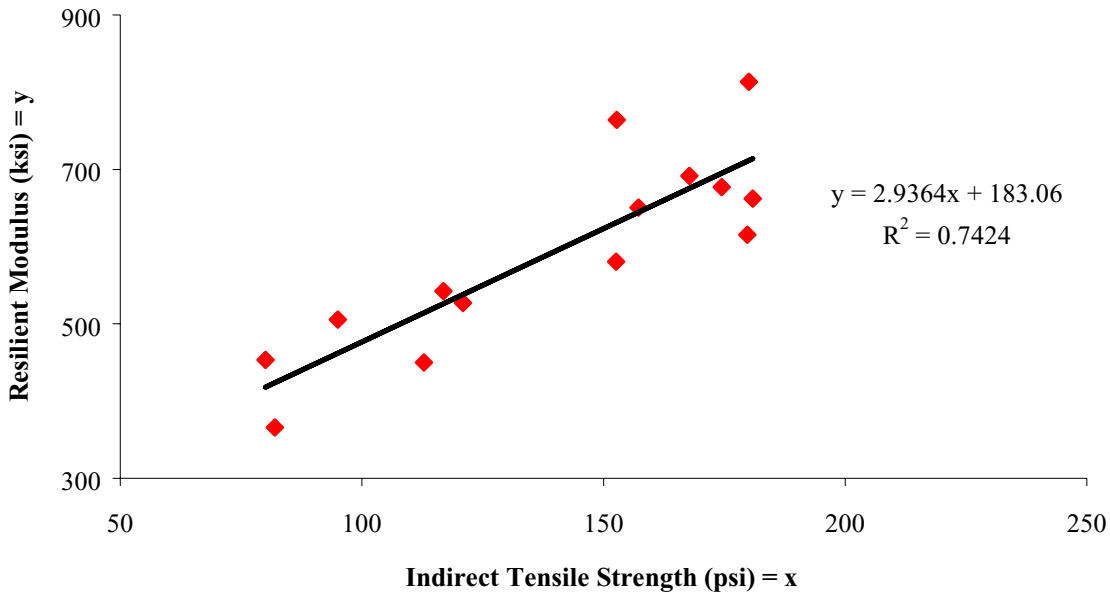


Figure 5.8 Resilient Modulus Vs. Indirect Tensile Strength for Asphalt-Treated Base Core Specimens (After Adjustment)

The variability of the resilient modulus of asphalt-treated base (ATB) within each section was analyzed. The mean resilient modulus, standard deviation, and coefficient of variation at each temperature are all listed in Table 5.15. These values were computed from the best-fit moduli at each temperature using the following equations (Lindeburg, 1995). The data presented in Table 5.15 indicates that the variations of the resilient modulus values of the ATB within each section were not great.

$$\overline{E} = \frac{1}{n}\sum_{i=1}^{n} E_i \qquad (5.4)$$

$$\sigma_{\underline{E}} = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left(\underline{E}_i - \overline{E}\right)^2} \tag{5.5}$$

$$V_E = \frac{\sigma_{\underline{E}}}{\overline{E}} \tag{5.6}$$

where $\overline{E}$ = Average; V_E = Coefficient of Variation; and σ_E = Standard Deviation.

Table 5.15 Mean Modulus, Standard Deviation, and Coefficient of Variation at Each Section of Asphalt-Treated Base Core Specimens

SHRP I.D.	Mean Modulus (psi) @			Standard Deviation (psi) @			Coeff. of Variation @		
	5 °C (41°F)	25 °C (77°F)	40°C (104°F)	5 °C (41°F)	25 °C (77°F)	40°C (104°F)	5 °C (41°F)	25 °C (77°F)	40°C (104°F)
390161	8.14E5	4.10E5	1.80E5	1.57E5	6.20E4	1.17E4	0.193	0.151	0.065
390162	8.27E5	5.11E5	4.59E5	2.12E5	8.68E4	3.09E5	0.256	0.170	0.673
390163	8.83E5	5.35E5	1.89E5	2.77E4	4.11E4	3.30E4	0.031	0.077	0.175
390105	1.03E6	6.02E5	2.86E5	2.19E5	8.46E4	5.82E3	0.215	0.140	0.020
390106	1.06E6	5.81E5	2.88E5	1.03E4	2.22E2	2.24E4	0.010	0.0004	0.078
390112	1.21E6	7.89E5	3.21E5	7.49E4	3.49E4	2.30E4	0.062	0.044	0.071
390901	1.40E6	6.05E5	2.97E5	2.77E4	1.01E5	2.81E4	0.020	0.168	0.095
390902	1.07E6	6.09E5	2.89E5	6.55E4	1.16E5	5.29E4	0.061	0.191	0.183
390903	9.57E5	6.32E5	3.07E5	3.07E5	2.51E4	3.73E4	0.321	0.040	0.121

Figure 5.9 shows the relationship between the resilient modulus at 77 °F (25 °C) and bulk specific gravity. It was shown that the relationship between those values was very weak with a correlation coefficient (R) of 0.36. This indicates that the resilient

modulus of asphalt-treated base can be estimated more accurately using indirect tensile strength values, rather than using bulk specific gravity values.

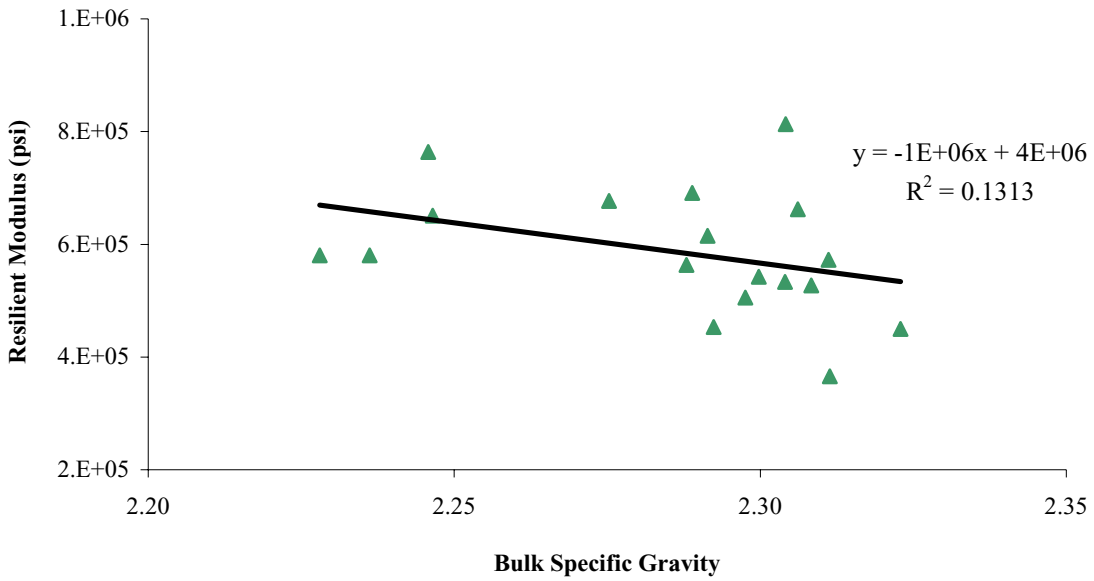


Figure 5.9 Resilient Modulus Vs. Bulk Specific Gravity for Asphalt-Treated Base Core Specimens

The last two ATB specimens listed in Table 5.11 (390112 and 390902) were tested by an alternative testing method, the modified dynamic modulus test, to compare the results between the indirect tensile mode and the unconfined compression mode. Four uniaxial strain gages were installed on each specimen for determining the resilient modulus and Poisson's ratio. Each test specimen was subjected to a repeated approximate square-wave shape pulse load at normal room temperature. The resilient modulus and Poisson's ratio values were computed using Equations 4.16 and 4.17, respectively. The test results are summarized in Table 5.16.

Table 5.16 Comparison of Test Results on Asphalt-Treated Base Between Indirect Tensile Strength and Alternative Test Methods

Unconfined Compression Loading Method				Indirect Tensile Strength Method		
SHRP ID	Resilient Modulus M_r (psi)	Poisson's Ratio μ		SHRP ID	Resilient Modulus M_r (psi)	Poisson's Ratio μ
390112	4.192E+05	0.338		390112	7.887E+05	0.275
390902	5.270E+05	0.375		390902	6.092E+05	0.210

The average resilient modulus at 77 °F (25 °C) from the indirect tension mode was compared to that from the unconfined dynamic loading method. The resilient modulus and Poisson's ratio of the asphalt-treated base using the unconfined compressive loading were about 0.47 million psi (3.26 GPa) and 0.36 million psi (3.26 GPa), respectively. The corresponding values based on the indirect tensile mode were 0.70 million psi (4.82 GPa) and 0.24 million psi (4.82 GPa), respectively. The resilient modulus at 77 °F (25 °C) obtained in the indirect tensile strength method were higher by 33% than those resulting from the unconfined compression loading method. Conversely, Poisson's ratio obtained from the unconfined compression loading method were about 32% higher than that obtained from the indirect method. The reason for this trend may due to the fact that in the indirect tension mode only the solid part of each layer was tested; while in the unconfined dynamic loading test mode, the solid part of each layer and the interface region between them, were tested all together.

5.4.2 Permeable Cement-Treated Base (PCTB)

Two permeable cement-treated base core specimens were subjected to the unconfined compression loading using the same procedure applied to the two ATB specimens. Figure 5.10 depicts a typical permeable cement-treated base core specimen. Only two permeable cement-treated base core specimens were tested, primarily due to the low success in recovering intact specimens in the field. These specimens with dimensions of 4 inches (102 mm) in diameter by 4 inches (102 mm) in height, were subjected to the unconfined compression loading using the same procedure applied to the two ATB specimens. The raw data obtained from these tests were utilized to compute the resilient modulus and Poisson's ratio of the PCTB material using Equations 4.16 and 4.17. The results are summarized in Table 5.17. This table indicates that the resilient modulus of the PCTB was, on average, approximately 1 million psi. The effect of the unit weight is visible on the magnitude of resilient modulus. One of the Poisson's ratio values was close to what was recommended (0.2) for cement-treated and permeable cement-treated bases in the NCHRP 1-26 report (University of Illinois at Urbana-Champaign, 1992).

Figure 5.10 Permeable Cement-Treated Base Core Specimen

Table 5.17 Laboratory Results on PCTB

Type of Base	Section	Unit Weight (pcf)	Resilient Modulus Mr (psi)	Poisson's Ratio μ
PCTB	3	121.08	6.181E+05	0.0600
	4	135.11	1.661E+06	0.2170

5.4.3 Lean Concrete Base (LCB)

Two lean concrete base core specimens, each having dimensions of 4 inches (102 mm) in diameter by 6 inches (152 mm) in height, were tested using the alternative (dynamic modulus type) test method. Figure 5.11 depicts a typical lean concrete base core specimen. Table 5.18 presents these test results. It was found that the resilient modulus of lean concrete base was about 2.5 million psi (17.4 GPa), with an 11% difference between the two results. The effect of the unit weight is clearly seen on the magnitude of resilient modulus. Conversely, the difference between the two Poisson's

ratio values was about 27%, with an average value of 0.22. Unconfined compression test results yielded the average 7-day, 28-day, and 1-year strengths of 0.82, 1.45, and 1.71 ksi, respectively.

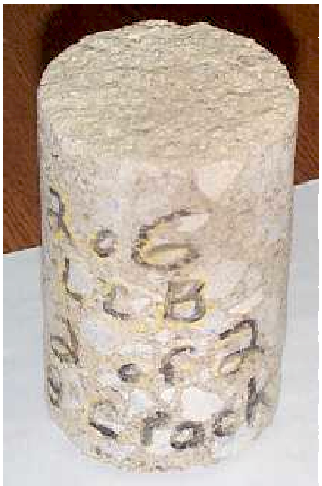


Figure 5.11 Lean Concrete Base Core Specimen

Table 5.18 Laboratory Results on LCB

Type of Base	Section	Unit Weight (pcf)	Resilient Modulus M_r (psi)	Poisson'r Ratio μ
LCB	390207 (1)	143.63	2.380E+06	0.1856
	390208 (1)	146.90	2.678E+06	0.2544

5.4.4 Permeable Asphalt-Treated Base (PATB)

Only bulk specific gravity was measured on the permeable asphalt-treated base (PATB) specimens. This was due to the generally poor conditions of the PATB

specimens cored in the field, which either had imperfect shapes (i.e. edges broken off, etc) or were too thin to be tested by the SHRP P07 Protocol. Figure 5.12 depicts a typical permeable asphalt-treated base core specimen. Bulk specific gravity obtained for these specimens is presented in Table 5.19. The average bulk specific gravity from these sections was about 2.324. The bulk specific gravity of PATB was found to be slightly higher than that of ATB, which might be due to the aggregate gradation used in the base materials.

Figure 5.12 Permeable Asphalt-Treated Base Core Specimen

Table 5.19 Bulk Specific Gravity of Permeable Asphalt-Treated Base (PATB)

Type of Base	Section	Bulk Specific Gravity
PATB	390109	2.3211
	390260	2.3263

5.5 Laboratory Test Results on Concrete

The PCC specimens recovered from the Ohio-SHRP Test Road site were tested in the ORITE laboratory according to the standard test methods previously described. Unit weight, static modulus, Poisson's ratio, and split tensile strength were determined for these specimens. A total of 136 PCC specimens were tested, which included 36 cylinder-molded specimens and 100 specimens directly cored from the rigid pavement slabs. This large number of specimens was needed to evaluate the time-dependent properties of three different mixture designs (low strength, regular strength, and high strength) used in the test road project. Details of the three mixtures were already described in Chapter 3. The following sections present the results obtained from each PCC test method. Some additional test results (ex. Compressive strength) on the PCC materials were provided by the ODOT, and were integrated into the discussions whenever appropriate.

5.5.1 Unit Weight

Concurrently with the placement of the rigid pavement slabs at the project site, some of the concrete was also formed into plastic molds 6 inches (152.4 mm) in diameter by 12 inches (304.8 mm) in height. This standard practice produced thirty-six 6 inch diameter cylindrical specimens of PCC. Additionally, about one hundred 4 inch diameter core specimens were obtained directly from the actual concrete slabs at varying ages. Both the cylindrical mold and cored specimens were utilized in determining the average unit weight of each concrete mixture type. The unit weight of each specimen was calculated using Equation 4.2. Tables 5.20 (a) through (c) present detailed test results for

each mix type. A summary information is listed in Table 5.21. According to these tables, the average unit weight was mostly between 139 and 146 pcf. There was a slight increase in the unit weight as the strength designation shifted from low (138 to 141 pcf) to regular (139 to 143 pcf) to high (140 to 146 pcf). No clear trend can be seen when the unit weights of the molded specimens are compared to the unit weights of the corresponding core specimens.

Table 5.20: Summary of Test Results on Three Different Mix Types of PCC Core Specimens

(a) Low Strength Concrete Specimens

SHRP Section No.	Age	Unit Weight (pcf)	Compressive Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi)	Poisson's Ratio
390809	28 days	NA	NA	NA	NA	NA
	1 year	137.7		422.8		
	1 year	138.0		448.3		
	1 year	138.2			3.83 E+6	0.213
390810	28 days	NA	NA	NA	NA	NA
	1 year	138.0		448.3		
	1 year	135.9		331.2		
	1 year	138.2			2.90E+6	0.144
	1 year	137.9			3.42E+6	0.205
* Average	28 days	NA	NA	NA	NA	NA
	1 year	137.7	NA	412.7	3.38E+6	0.187

(b) Regular Strength Concrete Specimens

SHRP Section No.	Age	Unit Weight (pcf)	Compressive Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi)	Poisson's Ratio
390201	14 days	142.2	5,980	378.5	NA	NA
	28 days	142.4	5,840	548.9	NA	NA
	1 year	143.2	8,580	691.6	NA	NA
390203	14 days	145.2	5,805	388.6	NA	NA
	28 days	141.8 142.8	6,340	580.2	2.98E+6	0.165
	1 year	142.6 141.8	8,465	522.9	4.01E+6	0.243
390205	14 days	NA	5,265	NA	NA	NA
	28 days	144.8 142.3	5,930	543.9	4.38E+6	0.201
	1 year	144.7	7,915	752.1	NA	NA
390207	14 days	141.6	4,110	407.0	NA	NA
	28 days	140.5 140.2	5,100	382.0	2.90E+6	0.179
	1 year	142.2 144.5	7,190	775.0	4.97E+6	0.196
390209	14 days	142.5	6,255	404.9	NA	NA
	28 days	141.9 144.0	5,940	427.2	4.27E+6	0.180
	1 year	143.2	8,710	627.8	NA	NA
390211	14 days	142.0	3,940	352.7	NA	NA
	28 days	141.9 143.9	4,265	490.1	2.50E+6	0.181
	1 year	136.9 138.4	6,520	636.9	4.32E+6	0.275
390260	14 days	142.5	5,660	528.1	NA	NA
	28 days	141.5 142.4	5,440; 5095	467.2	3.46E+6	0.232
	1 year	142.2 142.9	6,920	393.4	5.19E+6	0.29
390261	14 days	141.8	4,990	525.1	NA	NA
	28 days	143.0 141.9	5,440	461.6	2.63E+6	0.181
	1 year	143.5 143.7	8,915	759.7	5.15E+6	0.340
390262	14 days	141.2	5,030	463.6	NA	NA
	28 days	140.3 141.6	5,790	475.4	2.61E+6	0.180
	1 year	141.1 142.4	6,845	458.7	5.62E+6	0.250

(b) Regular Strength Concrete Specimens (cont'd)

SHRP Section No.	Age	Unit Weight (pcf)	Compressive Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi)	Poisson's Ratio
390263	14 days	141.8	5,520	437.6	NA	NA
	28 days	141.5 140.7	6,340	487.8	2.59E+6	0.133
	1 year	142.8	8,160	601.9	NA	NA
390264	14 days	142.6	4,370	570.8	NA	NA
	28 days	141.2 142.4	5,330	480.6	3.89E+6	0.168
	1 year	142.1 143.0	NA	559.5	3.91E+6	0.223
390265	14 days	143.4	5,985	391.4	NA	NA
	28 days	143.5 144.7	6,255	448.9	4.16E+6	0.329
	1 year	142.8 142.9	7,965	604.9	5.39E+6	0.25
* Average	14 days	142.4	5,243	440.8	NA	NA
	28 days	142.2	5,624	482.8	3.31E+6	0.194
	1 year	142.3	7,835	615.4	4.82E+6	0.258

(c) High Strength Concrete Specimens

SHRP Section No.	Age	Unit Weight (pcf)	Compressive Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi)	Poisson's Ratio
390202	14 days	144.5	7,690	553.2	NA	NA
	28 days	144.9 143.3	8,165	704.8	3.65E+6	0.235
	1 year	132.3 130.7	9,465	676.4	6.04E+6	NA
390204	14 days	NA	6,505	686.1	NA	NA
	28 days	145.0 141.7	6,110	578.5	3.91E+6	0.205
	1 year	144.6 145.6	7,380	517.4	5.88E+6	0.275
390206	14 days	144.3	7,105	500.7	NA	NA
	28 days	142.7 142.1	8,165	425.5	4.74E+6	0.212
	1 year	129.4 129.1	8,120	618.7	5.41E+6	NA
390208	14 days	145.5	6,495	551.4	NA	NA
	28 days	144.2 142.4	6,020	377.3	2.87E+6	0.166
	1 year	145.5 NA	9,430	746.9	5.42E+6	0.300
390210	14 days	144.9	7,855	387.0	NA	NA
	28 days	144.9 145.0 143.0	4,810 (removed)	412.9	NA	NA
	1 year	147.1 146.1	11,350	794.0	5.24E+6	NA
390212	14 days	145.1	6,520	572.5	NA	NA
	28 days	144.7 142.0	6,910	655.9	4.34E+6	0.293
	1 year	143.4 145.4	8,150	668.8	5.27E+6	0.21
390259	14 days	NA	7,180	806.7	NA	NA
	28 days	139.9 140.7	6,760	459.0	3.95E+6	0.175
	1 year	143.2 142.3	7,500	739.1	5.21E+6	0.189
* Average	14 days	144.9	7,050	579.9	NA	NA
	28 days	143.1	7,022	516.3	3.91E+6	0.214
	1 year	140.4	8,771	680.2	5.50E+6	0.244

Table 5.21 Overall Summary of Test Results on PCC Specimens

Ave. Material Properties @ Age:		Concrete Mix Trype:		
		Low Strength	Regular Strength	High Strength
Unit Weight (pcf) – Molded Specimens 6" Diameter	14 days	141.0	139.3	143.2
	28 days	141.0	141.2	143.1
	1 year	140.4	143.0	145.5
Unit Weight (pcf) – Cored Specimens 4" Diameter	14 days	NA	142.4	144.9
	28 days	NA	142.2	143.1
	1 year	137.7	142.3	140.4
Compressive Strength (psi) – Cored Specimens	14 days	NA	5,243	7,050
	28 days	NA	5,624	7,022
	1 year	NA	7,835	8,771
Split Tensile Strength (psi) – Molded Specimens	14 days	370.0	416.8	490.9
	28 days	405.0	415.6	573.4
	1 year	361.4	600.5	649.9
Split Tensile Strength (psi) – Cored Specimens	14 days	467.5	440.8	579.9
	28 days	498.3	482.8	516.3
	1 year	412.7	615.4	680.2
Young's Modulus (psi) – Cored Specimens	28 days	1.14E+6	3.31E+6	3.91E+6
	1 year	3.38E+6	4.82E+6	5.50E+6
Poisson's Ratio – Cored Specimens	28 days	NA	0.19	0.21
	1 year	0.19	0.26	0.24

5.5.2 Compressive and Split Tensile Strengths

All of the PCC specimens (both molded and cored) were utilized in the compressive and split tensile strength tests at the ages of 14 days, 28 days, and 1 year. The compressive strength was measured by loading each specimen axially according to the SHRP Protocol PC01. Depending on the specimen end surface quality, the ends were capped with a capping compound in some cases. The average compressive strength of each mixture type, as a function of age, is reported in Tables 5.20 (a) through (c). No compressive strength test data were available for the low strength mix. The test results were as expected between the regular and high strength mixes. The compressive strength

of the regular mix grew from 5.24 ksi (14 days) to 7.84 ksi (1 year), while that of the high strength increased from 7.05 ksi (14 days) to 8.77 ksi (1 year). The results show that the regular strength and high strength mixes gained 39% and 25% of the strength between the age of 28 days and 1 year.

The specimen preparation for performing the split tensile strength test was already described in Chapter 4. Specimen failure was typically characterized by a hairline crack that ran vertically through the center of the specimen. The maximum load sustained before failure was used to compute the split tensile strength using Equation 4.5. Tables 5.20 (a) through (c) summarize the strength test results on each mix type, and an overall summary of the test results can be found in Table 5.21.

The molded specimens primarily exhibited a trend of tensile strength increasing with age. Only the tensile strength of the low strength mix did not support this trend. Up to the age of 28 days, the tensile strengths of the low strength and regular strength PCC were nearly identical. As expected, tensile strength of the high strength PCC was the highest at all ages. The strength of the low strength PCC was the lowest at all ages. At the age of 14 days, the strengths of the low strength and regular strength specimens were about 75 and 85% of that of the high strength concrete. These percent values changed to 71 and 72% at 28 days and to 56 and 92% at 1 year. As indicated here, the regular strength concrete exhibited the largest strength gain of 44% between the ages of 28 days and 1 year.

Test results among the cored specimens exhibited somewhat similar trends. The tensile strength of the high strength PCC was higher than those of the other two mixes at

any age. The early strengths of the low strength and regular strength PCC were almost identical. The strength of the low strength concrete did not increase with age. The observed percent strength gains from the ages of 28 days to 1 year were similar between the regular strength (27%) and the high strength PCC (32%).

Comparing the split tensile strength results between the molded and cored specimens at the same age, it was noted that the tensile strength of cored specimens was generally 14 to 38% higher than that of the equivalent molded specimen. This strength difference between the molded and cored specimens may be attributed to the differences in curing conditions. The molded specimens were transported to the ORITE laboratory and stored in a relatively dry environment (to simulate less than an ideal curing condition and therefore to yield conservative strength properties), while the cored specimens, which were part of the actual rigid pavement slabs in the field, were thermally somewhat insulated all around by the adjacent concrete, and exposed to moisture periodically through precipitation events and subsurface drainage.

5.5.3 Young's (or Static) Modulus and Poisson's Ratio

For the cored specimens 28 days and 1 year old, the static modulus and Poisson's ratio were determined according to the ASTM C469 test procedure. This test began by attaching a special ring fixture, equipped with a compressometer and extensometer, onto the PCC test specimen. A few thousand pound preload was applied to condition the specimen and to verify that both the compressometer and extensometer were functioning properly. Each specimen was then subjected to two cycles of mid-range compressive

loading, during which the readings were recorded simultaneously from the load cell, compressometer, and extensometer gages at predetermined axial strain values. The data were then used to compute both the static modulus and Poisson's ratio. Tables 5.20 (a) through (c) summarize the average test results for each of the three mixture types. An overall summary is given in Table 5.21.

As expected, the measured static modulus reflected the orders of compressive strength among the mixes. At the age of 28 days, the high strength concrete possessed an average modulus of 3.9 million psi (27 GPa), while the average moduli of the low and regular strength mixtures were 1.1 million psi (7.6 GPa) and 3.3 million psi (22.8 GPa), respectively. These values changed to 3.4 million psi (23.4 GPa) for the low strength concrete, 4.8 million psi (33.1 GPa) for the regular strength, and 5.5 million psi (37.9 GPa) for the high strength concrete at the age of 1 year. These results reflect the statement made by Mindess and Young (1981) that the dominant factor in the concrete modulus is porosity, where modulus decreases markedly as the w/c ratio is increased.

Although, the modulus of the low strength concrete was the lowest among the three mixes at any age, its modulus gain (196%) between the ages of 28 days to 1 year was the largest. The regular strength and high strength specimens gained on the average 40 and 46% between the ages of 28 days and 1 year, respectively. The measured Poisson's ratio did not show a significant correlation to the concrete strength and varied mostly between 0.19 and 0.24.

5.5.4 Evaluation of Empirical Relationships

According to Derucher et al. (1998), the typical stress-strain relationship of concrete during unconfined compression tests exhibits linear-elastic characteristics, up to about 45% of its ultimate compressive strength. At higher stress levels, the relationship becomes increasingly non-linear due to a larger number of micro-cracks forming in the body. For this reason, Young's modulus of concrete was empirically estimated using the strength properties obtained from the compressive strength and split tensile strength laboratory tests. The formulas for estimating the static modulus of concrete from the compressive strength have been established by the American Concrete Institute (1989) and by the NCHRP study (University of Illinois at Urbana-Champaign, 1992):

ACI

$$E_c = 33\,(\gamma_c)^{1.5}\,(f'_c)^{0.5} \qquad (5.7)$$

where E_c = Young's modulus of concrete (psi); γ_c = unit weight (pcf); f'_c = compressive strength (psi).

NCHRP Study

$$E_c = 57{,}000\,(f'_c)^{0.5} \qquad (5.8)$$

The American Concrete Institute (ACI) came up with the following formula, which relates the split tensile strength (f'_s) to the compressive strength:

$$f'_s = 6.7\,(f'_c)^{0.5} \tag{5.9}$$

By combining the above three equations, Young's modulus can be also estimated from the split tensile strength using either of the following formulas:

$$E_c = 4.93\,(\gamma_c)^{1.5}\,(f'_s) \tag{5.10}$$

$$E_c = 8{,}507\,(f'_s) \tag{5.11}$$

In this empirical analysis, Young's moduli actually measured during the laboratory tests were compared to those estimated by the empirical methods using Equations 5.7, 5.8, 5.10, and 5.11. These estimations were calculated for all core specimens. A summary of the results is presented for each mixture type in Table 5.22 (a) through (c). In the tables, the values inside parentheses represent the ratio of (predicted/actual) in percentage.

Absence of the compressive strength data resulted in no application of Eqs. 5.7 and 5.8 to estimate the Young's modulus of the low strength mix. Table 5.22 (a) shows that Eqs. 5.11 was on the average slightly more accurate than Eq. 5.10 in predicting the long-term modulus of the low strength PCC. For the regular strength PCC, the empirical formulas all had a tendency to overestimate the 28-day modulus on the average by 26% to 35% (see Table 5.22 (b)). Among the four formulas, Eq. 5.11 overestimated the experimental modulus the least. The static modulus at 1 year was predicted equally well by Eqs. 5.7 and 5.8. Table 5.22 (c) summarizes the results for the high strength PCC.

Table 5.22 Comparison Between Actual and Predicted Young's Modulus Values

(a) Low Strength Concrete Specimens

SHRP No.	Age	Unit Wt . (pcf)	Compr. Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi): Actual	Prediction by Eq. 5.10	Prediction by Eq. 5.11
390809	28 days	NA	NA	NA	1.14E+6	NA	NA
	1 year	137.7	NA	422.8		3.37E+6 (88%)	3.60E+6 (94%)
	1 year	138.0		448.3		3.58E+6 (93%)	3.81E+6 (99%)
	1 year	138.2			3.83E+6		
390810	1 year	138.0	NA	448.3		3.38E+6 (99%)	3.81E+6 (111%)
	1 year	135.9		331.2		2.59E+6 (89%)	2.82E+6 (97%)
	1 year	138.2			2.90E+6		
	1 year	137.9			3.42E+6		
**	28 days					NA	NA
	1 year					92% (ave.) 5.0% (std. dev.)	100% (ave.) 7.5% (std. dev.)

(b) Regular Strength Concrete Specimens

SHRP No.	Age	Unit Wt. (pcf)	Compr. Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi): Actual	Eq. 5.7 ACI	Eq. 5.8 NCHRP	Eq. 5.10	Eq. 5.11
390201	28 days	142.4 NA	5,840	548.9	NA	4.29E6 (NA)	4.36E6 (NA)	4.60E6 (NA)	4.67E6 (NA)
	1 year	143.2 NA	8,580	691.6	NA	5.24E6 (NA)	5.28E6 (NA)	5.84E6 (NA)	5.88E6 (NA)
390203	28 days	141.8 142.8	6,340	580.2	2.98E6	4.44E6 (149%)	4.54E6 (152%)	4.83E6 (162%)	4.94E6 (166%)
	1 year	142.6 141.8	8,465	522.9	4.01E6	5.17E6 (129%)	5.24E6 (131%)	4.39E6 (109%)	4.45E6 (111%)
390205	28 days	144.8 142.3	5,930	543.9	4.38E6	4.43E6 (101%)	4.39E6 (100%)	4.67E6 (107%)	4.63E6 (106%)
	1 year	144.7 NA	7,915	752.1	NA	5.11E6 (NA)	5.07E6 (NA)	6.45E6 (NA)	6.40E6 (NA)
390207	28 days	140.5 140.2	5,100	382.0	2.90E6	3.92E6 (135%)	4.07E6 (140%)	3.14E6 (108%)	3.25E6 (112%)
	1 year	142.2 144.5	7,190	775.0	4.97E6	4.74E6 (95%)	4.83E6 (97%)	6.48E6 (130%)	6.59E6 (133%)
390209	28 days	141.9 144.0	5,940	427.2	4.27E6	4.30E6 (101%)	4.39E6 (103%)	3.56E6 (83%)	3.63E6 (85%)
	1 year	143.2 NA	8,710	627.8	NA	5.28E6 (NA)	5.32E6 (NA)	5.30E6 (NA)	5.34E6 (NA)
390211	28 days	141.9 143.9	4,265	490.1	2.50E6	3.64E6 (146%)	3.72E6 (149%)	4.08E6 (163%)	4.17E6 (167%)
	1 year	136.9 138.4	6,520	636.9	4.32E6	4.27E6 (99%)	4.60E6 (106%)	5.03E6 (116%)	5.42E6 (125%)
390260	28 days	141.5 142.4	5,440	467.2	3.46E6	4.10E6 (118%)	4.20E6 (121%)	3.88E6 (112%)	3.97E6 (115%)
	1 year	142.2 142.9	6,920	393.4	5.19E6	4.65E6 (90%)	4.74E6 (91%)	3.29E6 (63%)	3.35E6 (65%)

(b) Regular Strength Concrete Specimens (cont'd)

SHRP No.	Age	Unit Wt. (pcf)	Compr. Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi):				
					Actual	Eq. 5.7 ACI	Eq. 5.8 NCHRP	Eq. 5.10	Eq. 5.11
390261	28 days	143.0 141.9	5,440	461.6	2.63E6	4.16E6 (158%)	4.20E6 (160%)	3.89E6 (148%)	3.93E6 (149%)
	1 year	143.5 143.7	8,915	759.7	5.15E6	5.36E6 (104%)	5.38E6 (104%)	6.44E6 (125%)	6.46E6 (125%)
390262	28 days	140.3 141.6	5,790	475.4	2.61E6	4.17E6 (160%)	4.34E6 (166%)	3.89E6 (149%)	4.04E6 (155%)
	1 year	141.1 142.4	6,845	458.7	5.62E6	4.58E6 (81%)	4.72E6 (84%)	3.79E6 (67%)	3.90E6 (69%)
390263	28 days	141.5 140.7	6,340	487.8	2.59E6	4.42E6 (171%)	4.54E6 (175%)	4.05E6 (156%)	4.15E6 (160%)
	1 year	142.8 NA	8,160	601.9	NA	5.09E6 (NA)	5.15E6 (NA)	5.06E6 (NA)	5.12E6 (NA)
390264	28 days	141.2 142.4	5,330	480.6	3.89E6	4.04E6 (104%)	4.16E6 (107%)	3.98E6 (102%)	4.09E6 (105%)
	1 year	142.1 143.0	NA	559.5	3.91E6	NA (NA)	NA (NA)	4.67E6 (119%)	4.76E6 (122%)
390265	28 days	143.5 144.7	6,255	448.9	4.16E6	4.49E6 (108%)	4.51E6 (108%)	3.80E6 (91%)	3.82E6 (92%)
	1 year	142.8 142.9	7,965	604.9	5.39E6	5.03E6 (93%)	5.09E6 (94%)	5.09E6 (94%)	5.15E6 (96%)
Ave.	28 days					132%	135%	126%	128%
	1 year					99%	101%	103%	106%

[Note] All test results were obtained on the 4" diameter core specimens.

(c) High Strength Concrete Specimens

SHRP No.	Age	Unit Wt. (pcf)	Compr. Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi):				
					Actual	Eq. 5.7 ACI	Eq. 5.8 NCHRP	Eq. 5.10	Eq. 5.11
390202	28 days	144.9 143.3	8,165	704.8	3.65E6	5.20E6 (142%)	5.15E6 (141%)	6.06E6 (166%)	6.00E6 (164%)
	1 year	132.3 130.7	9,465	676.4	6.04E6	4.89E6 (81%)	5.55E6 (92%)	5.07E6 (84%)	5.75E6 (95%)
390204	28 days	145.0 141.7	6,110	578.5	3.91E6	4.50E6 (115%)	4.46E6 (114%)	4.98E6 (127%)	4.92E6 (126%)
	1 year	144.6 145.6	7,380	517.4	5.88E6	4.93E6 (84%)	4.90E6 (83%)	4.44E6 (76%)	4.40E6 (75%)
390206	28 days	142.7 142.1	8,165	425.5	4.74E6	5.08E6 (107%)	5.15E6 (109%)	3.58E6 (76%)	3.62E6 (76%)
	1 year	129.4 129.1	8,120	618.7	5.41E6	4.38E6 (81%)	5.14E6 (95%)	4.49E6 (83%)	5.26E6 (97%)
390208	28 days	144.2 142.4	6,020	377.3	2.87E6	4.43E6 (154%)	4.42E6 (154%)	3.22E6 (112%)	3.21E6 (119%)
	1 year	145.5 NA	9,430	746.9	5.42E6	5.62E6 (104%)	5.54E6 (102%)	6.46E6 (119%)	6.35E6 (117%)

(c) High Strength Concrete Specimens (cont'd)

SHRP No.	Age	Unit Wt. (pcf)	Compr. Strength (psi)	Split Tensile Strength (psi)	Static Modulus (psi):				
					Actual	Eq. 5.7 ACI	Eq. 5.8 NCHRP	Eq. 5.10	Eq. 5.11
390210	28 days	144.9 143.0	4,810	412.9	NA	3.99E6 (NA)	3.95E6 (NA)	3.55E6 (NA)	3.51E6 (NA)
	1 year	147.1 146.1	11,350	794.0	5.24E6	6.27E6 (120%)	6.07E6 (116%)	6.98E6 (133%)	6.75E6 (129%)
390212	28 days	144.7 142.0	6,910	655.9	4.34E6	4.77E6 (110%)	4.74E6 (109%)	5.63E6 (130%)	5.58E6 (129%)
	1 year	143.4 145.4	8,150	668.8	5.27E6	5.16E6 (98%)	5.15E6 (98%)	5.66E6 (107%)	5.69E6 (108%)
390259	28 days	139.9 140.7	6,760	459.0	3.95E6	4.49E6 (114%)	4.69E6 (119%)	3.74E6 (95%)	3.90E6 (99%)
	1 year	142.3 143.2	7,500	739.1	5.21E6	4.85E6 (93%)	4.94E6 (95%)	6.19E6 (119%)	6.29E6 (121%)
* Ave.	28 days					124%	124%	118%	118%
	1 year					94%	97%	103%	106%

[Note] All test results were obtained on the 4" diameter core specimens.

All four formulas overpredicted the 28-day static modulus by about 20%. The modulus at the age of 1 year was estimated slightly better by Eqs. 5.8 and 5.10.

5.5.5 Thermal Coefficient of Expansion

Thermal coefficient of expansion was measured on a few selected core specimens at the Federal Highway Administration laboratory. The results were listed in Table 5.23.

Table 5.23 Thermal Coefficient of Expansion of PCC Specimens

SHRP Section No.	Design Mix Type	Thermal Coeff. of Expansion
390204	High Strength	11.6E(- 6) per ºC
390209	Regular Strength	11.3E(- 6) per ºC
390264	Regular strength	11.3E(- 6) per ºC

5.6 Laboratory Test Results on Asphalt Concrete

Three types of tests were conducted at the ORITE laboratory in order to determine mechanistic properties of the asphalt concrete material utilized in the Ohio-SHRP Test Road. The ATB test program described earlier, and the AC test program were similar because these materials shared similar physical characteristics. A few additional tests were conducted on the AC, however, simply because the AC constituted the more important top paving layer. The AC test program consisted of bulk specific gravity tests, resilient modulus test (per SHRP P07), indirect tension strength test (per SHRP P07), and creep compliance test (per SHRP P06). The following sections present the results procured from these three test methods.

5.6.1 Resilient Modulus Test

A total of sixty-one specimens were cored from the flexible pavements at the Ohio-SHRP Test Road. These included fifty specimens from the SPS-1 experiment sections (390101, 390104, 390105, 390106, 390107, 390108, 390111, and 390112) and eleven specimens from the SPS-9 experiment sections (390901, 390902, and 390903). The original core specimens were carefully trimmed by a circular saw to separate them into the surface and intermediate layer specimens. All the specimens had a diameter of 4 inches, but their thickness was less than 2 inches for most surface layer specimens. Prior to the resilient modulus test, the bulk specific gravity was determined for each specimen according to the AASHTO T166-93 Method A. The bulk specific gravity test results are presented in Table 5.24.

Table 5.24 Bulk Specific Gravity (BSG) of Asphalt Concrete Specimens

SHRP Sec. No.	BSG - Surface	BSG - Intermediate
390101	2.211	2.441
390104	2.257	2.236
390105	2.241	2.226
390106	2.195	2.252
390107	2.230	2.192
390108	2.228	2.274
390111	2.199	2.348
390112	2.245	2.285
390901	2.289	2.273
390902	2.295	2.227
390903	2.210	2.227

Within the SPS-1 experiment, the bulk specific gravity for the surface layer ranged between 2.20 and 2.26, with an overall average of 2.23, while the intermediate layer ranged between 2.19 and 2.44, with an average of 2.28. Within the SPS-9 experiment, the bulk specific gravity for the surface layer ranged between 2.21 and 2.30, with an average of 2.26. The intermediate layer ranged from 2.23 to 2.27, with an average of 2.24. These values indicate that the average bulk specific gravity was about the same between the two layers regardless of the experiment sections. It was noted that the average bulk specific gravity of the asphalt concrete specimens were slightly lower than those measured for the ATB specimens.

The resilient modulus test was performed following the bulk specific gravity test, and was carried out according to the same procedure previously applied for asphalt-treated base specimens, SHRP Protocol P07. A summary of the resilient modulus (adjusted), Poisson's ratio, and indirect tensile strength for surface and intermediate layers for each section is listed in Tables 5.25 and 5.26, respectively. Figures 5.13 and

Table 5.25 Summary of Resilient Modulus, Poisson's Ratio, and Indirect Tensile Strength of Asphalt Concrete Specimens (Surface Layer)

SHRP ID	Mean Modulus (psi) @			Mean Poisson's Ratio (μ) @			ITS (psi)
	5°C (41°F)	25°C (77°F)	40°C(104°F)	5°C (41°F)	25°C (77°F)	40°C(104°F)	@ 25°C
390101	7.05E+05	3.31E+05	1.41E+05	0.16	0.45	0.50	85.96
390104	7.02E+05	3.31E+05	1.72E+05	0.14	0.41	0.50	65.97
390105	1.05E+06	5.43E+05	3.14E+05	0.15	0.29	0.48	139.94
390106	7.82E+05	4.28E+05	2.32E+05	0.13	0.32	0.48	97.13
390107	8.49E+05	4.47E+05	1.86E+05	0.14	0.38	0.49	104.99
390108	9.72E+05	4.92E+05	2.71E+05	0.12	0.21	0.46	116.24
390111	7.51E+05	3.65E+05	1.89E+05	0.17	0.49	0.50	63.32
390112	7.61E+05	4.39E+05	2.02E+05	0.10	0.33	0.49	105.65
390901	7.31E+05	4.23E+05	2.51E+05	0.10	0.17	0.41	134.03
390902	6.94E+05	4.44E+05	1.86E+05	0.24	0.49	0.50	112.73
390903	7.07E+05	4.17E+05	1.90E+05	0.13	0.35	0.48	93.86

[Note] "ITS" = Indirect Tensile Strength

Table 5.26 Summary of Resilient Modulus, Poisson's Ratio and Indirect Tensile Strength of Asphalt Concrete Specimens (Intermediate Layer)

SHRP ID	Mean Modulus (psi) @			Mean Poisson's Ratio (ν) @			ITS (psi)
	5°C(41°F)	25°C(77°F)	40°C(104°F)	5°C(41°F)	25°C(77°F)	40°C(104°F)	@ 25°C
390101	7.05E+05	3.31E+05	1.41E+05	0.16	0.45	0.50	85.96
390104	7.02E+05	3.31E+05	1.72E+05	0.14	0.41	0.50	65.97
390105	1.05E+06	5.43E+05	3.14E+05	0.15	0.29	0.48	139.94
390106	7.82E+05	4.28E+05	2.32E+05	0.13	0.32	0.48	97.13
390107	8.49E+05	4.47E+05	1.86E+05	0.14	0.38	0.49	104.99
390108	9.72E+05	4.92E+05	2.71E+05	0.12	0.21	0.46	116.24
390111	7.51E+05	3.65E+05	1.89E+05	0.17	0.49	0.50	63.32
390112	7.61E+05	4.39E+05	2.02E+05	0.10	0.33	0.49	105.65
390901	7.31E+05	4.23E+05	2.51E+05	0.10	0.17	0.41	134.03
390902	6.94E+05	4.44E+05	1.86E+05	0.24	0.49	0.50	112.73
390903	7.07E+05	4.17E+05	1.90E+05	0.13	0.35	0.48	93.86

5.15 plot the original measured resilient modulus against the three test temperatures for the surface and intermediate layers, respectively. Figures 5.14 and 5.16 are similar plots constructed after adjusting some of the Poisson's ratio values for the surface and intermediate layers, respectively.

Figures 5.13 through 5.16 both illustrate the temperature dependency of the resilient modulus of the asphalt concrete material for both the surface and intermediate layers. The average original resilient moduli of the surface layer were 0.69, 0.43, and 0.23 million psi (4.8, 3.0, and 1.6 GPa) at the three test temperatures. This indicates that the moduli decreased by 38.1 and 66.6% at 77 and 104 °F (25 and 40 °C), respectively, when compared to those at 41 °F (5 °C). The temperature sensitive nature of the asphalt binder played a significant role in the resilient moduli, as reported by Little et al. (1993).

The average original resilient moduli of the intermediate layer were 0.8, 0.43, and 0.21 million psi (5.5, 3.0, and 1.5 GPa) at the three test temperatures. This indicates that the moduli at 77 and 104 °F (25 and 40 °C) were typically 46.1 and 73.8%, respectively, of that measured at 41°F (5 °C).

The original resilient modulus for the surface layer at 41 °F (5 °C) ranged widely between 0.29 and 0.98 million psi (2 and 6.8 GPa). Some of the resilient moduli at 41 °F (5 °C) had to be adjusted due to a problem with the measured Poisson's ratio values (they were less than 0.1). This modification resulted in an average adjusted resilient modulus of 0.80 million psi (5.5 GPa) at 41 °F (5 °C) as shown in Figure 5.17. The original resilient modulus at 77 °F (25 °C) changed only slightly, since only one of the original Poisson's ratio values needed to be adjusted.

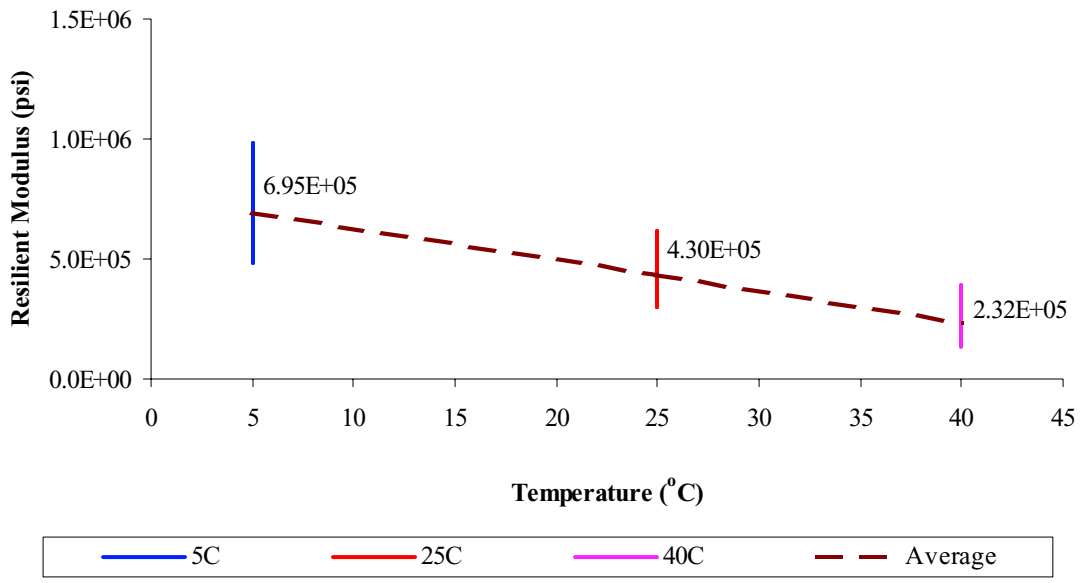


Figure 5.13 Resilient Modulus vs. Temperature Relationship for Asphalt Concrete Core Specimens – Surface Layer (Original)

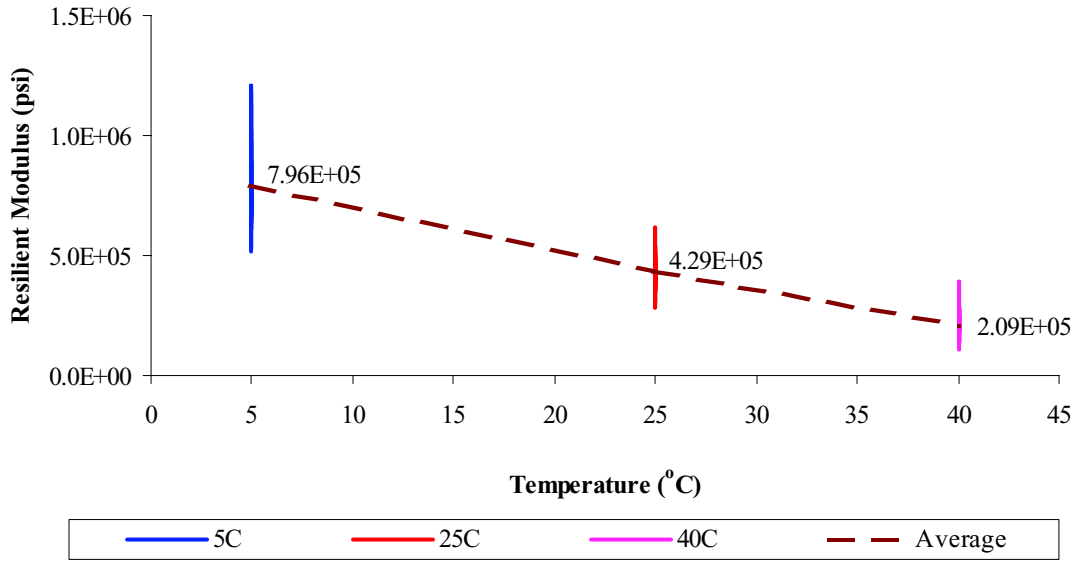


Figure 5.14 Resilient Modulus vs. Temperature Relationship for Asphalt Concrete Core Specimens – Surface Layer (After Adjustment)

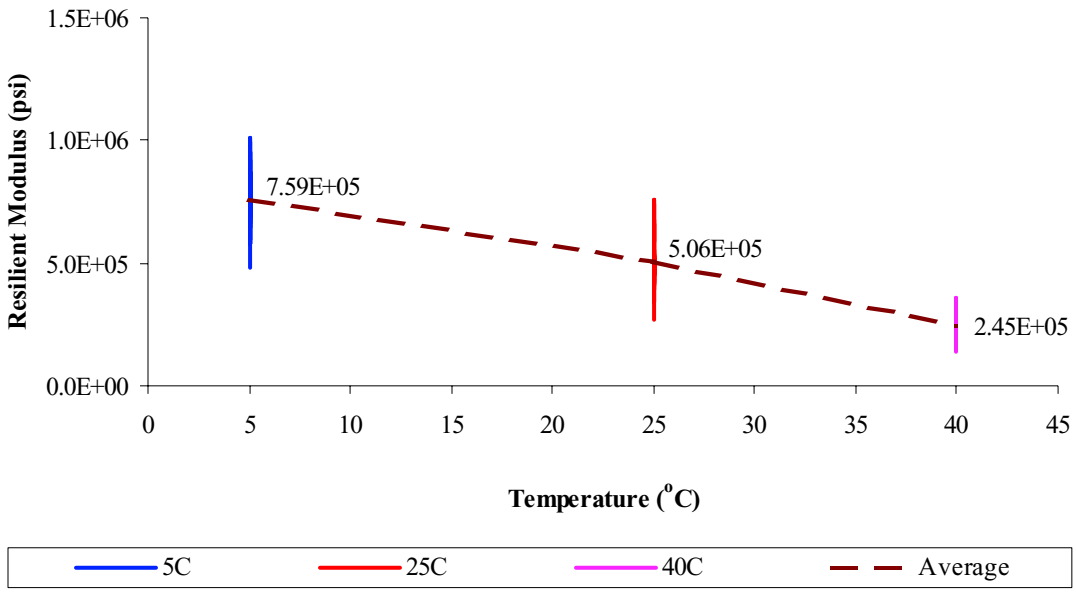


Figure 5.15 Resilient Modulus vs. Temperature Relationship for Asphalt Concrete Core Specimens – Intermediate Layer (Original)

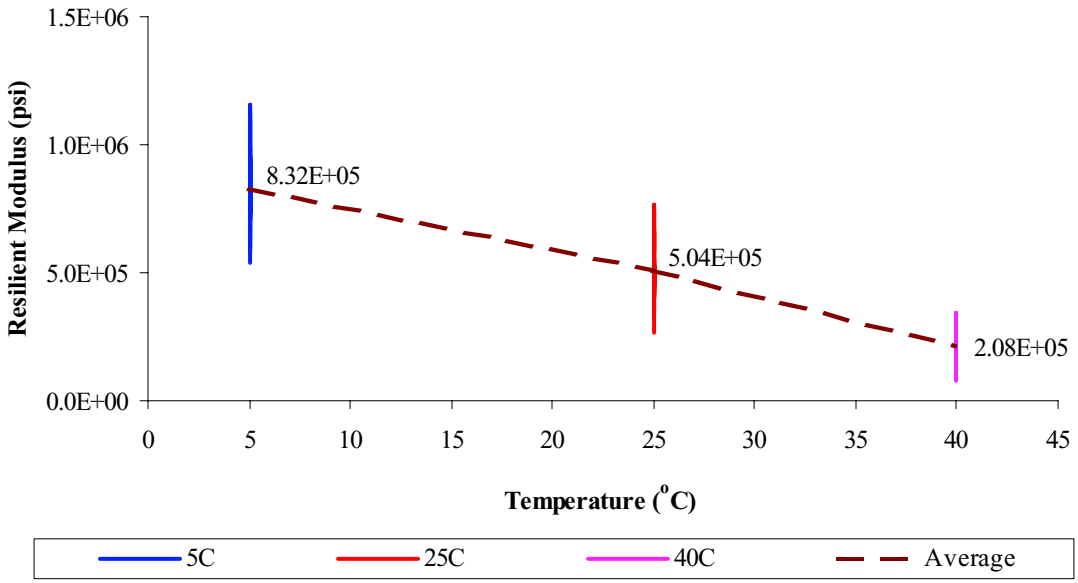


Figure 5.16 Resilient Modulus vs. Temperature Relationship for Asphalt Concrete Core Specimens – Intermediate Layer (After Adjustment)

Several resilient moduli for the surface layer at 104 °F (40 °C) had to be adjusted, since most of the Poisson's ratios were greater than 0.5. It should be noted that the temperature effect extended to the Poisson's ratio, and this observation supports the finding by Little et al (1993). Most of the Poissons' ratio values at higher temperatures were greater than 0.5, which suggests that the aggregate interaction played an important role as the viscosity of asphalt decreased at higher temperatures. The adjustment made to the Poisson's ratio values, resulted in the reduction of the resilient modulus at 104 °F (40 °C) by 9.9%.

For the intermediate layer, the original resilient modulus at 41 °F (5 °C) ranged widely between 0.48 and 1.01 million psi (3.3 and 7.0 GPa). Most of the Poisson's ratios were smaller than 0.1 and the resilient modulus had to be adjusted. This resulted in an average adjusted resilient modulus of 0.83 million psi (5.7 GPa) at 41 °F (5 °C) as shown in Figure 5.14. Almost all of the resilient moduli for the intermediate layer at 104 °F (40 °C) also had to be adjusted for the same reason. The adjustment made to the Poisson's ratio values, resulted in the reduction of the resilient modulus at 104 °F (40 °C) by 15%. At the temperatures of 41 °F (5 °C) and 77 °F (25 °C), resilient moduli of the intermediate layer specimens were higher than those of the surface layer specimens.

Overall, after Poisson's ratio adjustment, the average resilient moduli at 77 and 104 °F (25 and 40 °C) for surface layer became equal to 46.1, and 73.7% of the average modulus at 41 °F (5 °C). Meanwhile, after modification, the average resilient moduli at 77 and 104 °F (25 and 40 °C) for intermediate layer became equal to 39.4 and 75% of the

average modulus at 41 °F (5 °C). These values were comparable to those observed for ATB specimens. For the ATB, the average resilient moduli at 77 and 104 °F (25 and 40 °C) were 43 and 71.8% of the average modulus at 41°F (5 °C) after adjusting the Poisson's ratio values.

Next, variability of the AC resilient modulus for surface and intermediate layer within each section was analyzed. The mean resilient modulus, standard deviation, and coefficient of variation at each temperature are all listed in Tables 5.27 and 5.28. These values were computed using Eqns. 5.4, 5.5, and 5.6. From the data presented, it may be stated that resilient modulus of the asphalt concrete for both layers did not fluctuates widely within each section.

Table 5.27 Statistical Summary of Test Results on Asphalt Concrete Core Specimens at Different Temperatures for Surface Layer

SHRP I.D.	Mean Modulus (psi) @			Standard Deviation (psi) @			Coeff. of Variation @		
	5 °C (41°F)	25 °C (77°F)	40°C (104°F)	5 °C (41°F)	25 °C (77°F)	40°C (104°F)	5 °C (41°F)	25 °C (77°F)	40°C (104°F)
390101	7.05E5	3.31E5	1.41E5	7.22E4	6.09E4	4.71E4	0.102	0.184	0.334
390104	7.02E5	3.31E5	1.72E5	-	-	-	-	-	-
390105	1.05E6	5.43E5	3.14E5	2.23E5	1.04E5	1.04E5	0.212	0.192	0.331
390106	7.82E5	4.28E5	2.32E5	2.86E5	8.05E4	6.34E4	0.366	0.188	0.274
390107	8.49E5	4.47E5	1.86E5	1.76E5	7.17E4	4.59E4	0.207	0.161	0.247
390108	9.72E5	4.92E5	2.71E5	1.66E5	4.24E4	3.02E4	0.171	0.086	0.112
390111	7.51E5	3.65E5	1.89E5	-	-	-	-	-	-
390112	7.61E5	4.39E5	2.02E5	1.04E5	7.05E5	5.38E4	0.137	0.161	0.267
390901	7.31E5	4.23E5	2.51E5	5.84E5	1.87E4	3.11E4	0.080	0.044	0.124
390902	6.94E5	4.44E5	1.86E5	1.25E5	2.10E4	4.95E1	0.181	0.047	0.000
390903	7.07E5	4.17E5	1.90E5	2.64E5	1.02E5	8.07E4	0.374	0.245	0.425

[Note] " - " means that the standard deviation cannot be calculated since only one sample was tested in the section.

Figures 5.17 and 5.18 depict a relationship between the resilient modulus at 77 °F (25 °C) and bulk specific gravity for both the surface and intermediate layers. It was found that the relationships were not strong at all for both layers. This indicates that resilient modulus estimated from bulk specific gravity will not be accurate. This explains why there was no recommendation on the relationship in the NCHRP study.

Table 5.28 Statistical Summary of Test Results on Asphalt Concrete Core Specimens at Different Temperatures for Intermediate Layer

SHRP I.D.	Mean Modulus (psi) @			Standard Deviation (psi) @			Coeff. of Variation @		
	5 °C (41°F)	25 °C (77°F)	40°C (104°F)	5 °C (41°F)	25 °C (77°F)	40°C (104°F)	5 °C (41°F)	25 °C (77°F)	40°C (104°F)
390101	5.39E5	4.23E5	1.56E5	-	-	-	-	-	-
390104	7.75E5	4.71E5	1.68E5	2.50E5	8.67E4	2.62E4	0.322	0.184	0.156
390105	9.34E5	6.18E5	2.40E5	2.77E5	5.66E4	4.69E3	0.297	0.092	0.019
390106	7.93E5	4.94E5	2.08E5	1.94E5	1.08E5	5.71E4	0.244	0.217	0.274
390107	8.37E5	5.37E5	2.96E5	1.70E5	2.81E4	6.72E4	0.203	0.052	0.227
390108	9.30E5	4.81E5	2.36E5	8.63E4	1.51E4	1.33E4	0.093	0.031	0.056
390111	7.58E5	4.26E5	1.40E5	3.08E4	1.77E4	6.54E3	0.041	0.042	0.047
390112	9.39E5	5.78E5	2.46E5	1.46E5	1.51E5	5.48E4	0.156	0.262	0.223
390901	7.52E5	5.23E5	1.66E5	2.94E4	3.50E3	1.92E4	0.039	0.007	0.116
390902	9.54E5	5.17E5	1.95E5	-	-	-	-	-	-
390903	7.15E5	3.20E5	1.28E5	8.16E4	7.16E4	7.19E4	0.114	0.224	0.564

[Note] "-" means that the standard deviation cannot be calculated since only one sample was tested in the section.

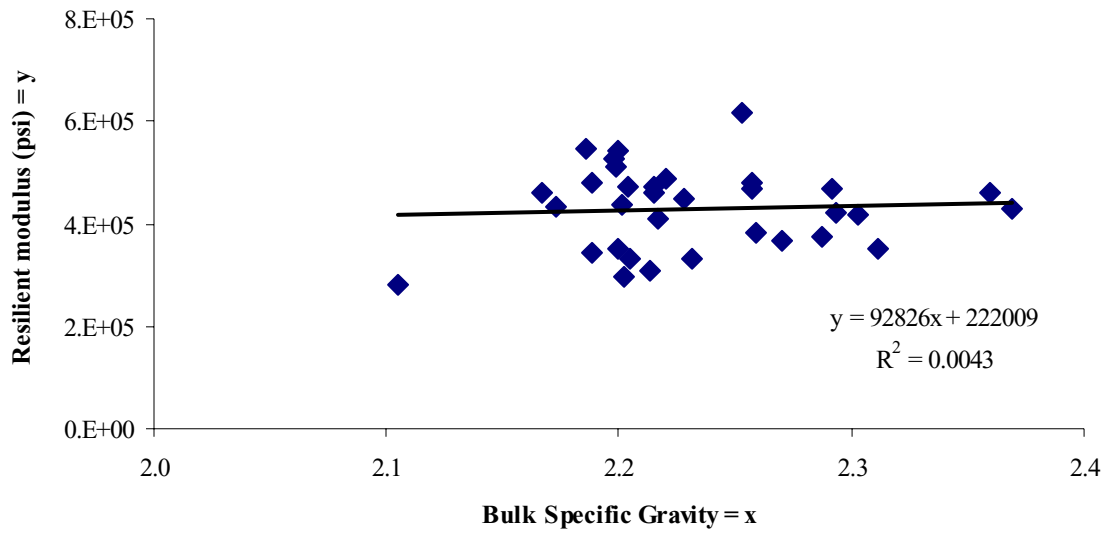


Figure 5.17 Resilient Modulus Vs. Bulk Specific Gravity Relationship for Asphalt Concrete Core Specimens--Surface Layer

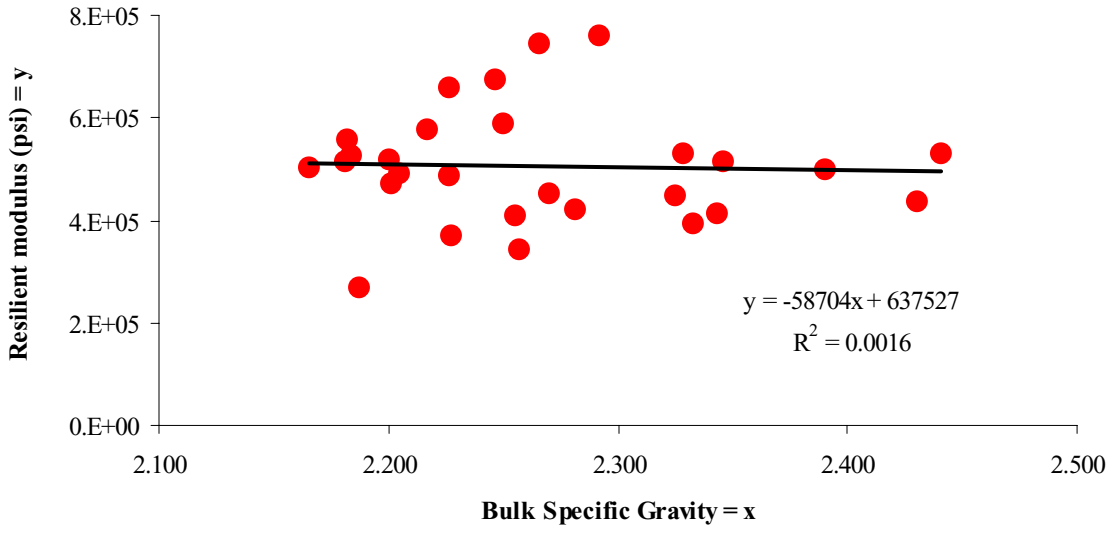


Figure 5.18 Resilient Modulus Vs. Bulk Specific Gravity Relationship for Asphalt Concrete Core Specimens--Intermediate Layer

5.6.2 Indirect Tensile Strength Test

The indirect tensile strength test was performed, following the resilient modulus test at 104 °F (40 °C). The indirect tensile strength test utilized the same procedure previously applied to the asphalt-treated base material. The specimens were first re-stored in the environmental chamber for at least 24 hours at 77 ± 2 °F (25 ± 1 °C). Each test specimen was then loaded diametrically in the indirect tension mode. The test results are summarized in Tables 5.25 and 5.26 for each asphalt concrete section.

Indirect tensile strength values were used to indicate the strength of the asphalt mixture without rupture. Only materials with high indirect tensile strength values can sustain loading without rupture. Tables 5.29 and 5.30 list the indirect tensile strength and coefficient of variation for surface and intermediate layers carried out at room temperature for each asphalt concrete section. It was found that the indirect tensile strength for the surface layer varied between 63.3 and 151.0 psi (0.44 and 1.04 MPa) with an average of 104.1 psi (0.42 MPa). The indirect tensile strength for the intermediate layer varied from 61.98 to 208.51 (0.72 to 1.44 MPa), with an average of 112.1 psi (0.77 MPa). These results indicate that the intermediate layer possessed slightly higher indirect tensile strengths than the surface layer. As shown in Tables 5.29 and 5.30, even though there was some inherent scattering among the test results, the strength did not fluctuate widely within each section for either layer.

A linear regression equation was used to establish a relationship between the resilient modulus and indirect tensile strength, as suggested by the NCHRP 1-26 Report

Table 5.29 Statistical Analysis Results on Indirect Tensile Strength at 25°C for Each Asphalt Concrete Section (Surface Layer)

SHRP ID	Indirect Tensile Strength (psi)		Coefficient of Variation
	Mean	Standard Deviation	
390101	85.96	23.16	0.269
390104	65.97	-	-
390105	139.94	15.63	0.112
390106	97.13	22.17	0.228
390107	104.99	13.95	0.133
390108	116.24	15.87	0.137
390111	63.32	-	-
390112	105.65	18.86	0.179
390901	134.03	6.36	0.047
390902	112.73	6.04	0.054
390903	93.86	16.17	0.172

Table 5.30 Statistical Analysis Results on Indirect Tensile Strength at 25°C for Each Asphalt Concrete Section (Intermediate Layer)

SHRP ID	Indirect Tensile Strength (psi)		Coefficient of Variation
	Mean	Standard Deviation	
390101	90.05	-	-
390104	99.46	20.60	0.207
390105	101.18	41.94	0.415
390106	107.23	37.02	0.345
390107	117.41	2.08	0.018
390108	111.73	4.22	0.038
390111	97.91	13.62	0.139
390112	142.38	50.92	0.358
390901	100.70	9.30	0.092
390902	145.67	-	-
390903	74.12	17.16	0.232

(University of Illinois at Urbana-Champaign, 1992). The general form of the equation was:

$$y = a + bx \tag{5.10}$$

where a = y intercept (ksi); x = indirect tensile strength (psi); and y = resilient modulus of asphalt concrete (ksi).

The relationship between resilient modulus and indirect tensile strength for surface and intermediate layer were shown in Figures 5.19 and 5.21, respectively. The relationship between those values for both layers was fairly strong having a similar correlation coefficient (R) of about 0.80. However, these figures indicate that there was some scattering among the data points. After further examination of the test data, it was decided to remove the small number of data points from the plot, due to the imperfect specimen dimensions or extreme bulk specific gravity values associated with them. After such a modification, the relationship between resilient modulus and indirect tensile strength became stronger especially for intermediate layer, as evident in the revised correlation coefficient value (R) of 0.9 for surface layer and correlation coefficient value (R) of 0.94 for intermediate layer (see Figures 5.20 and 5.22). This implies that it is possible to estimate the room temperature strength with a sufficient degree of confidence.

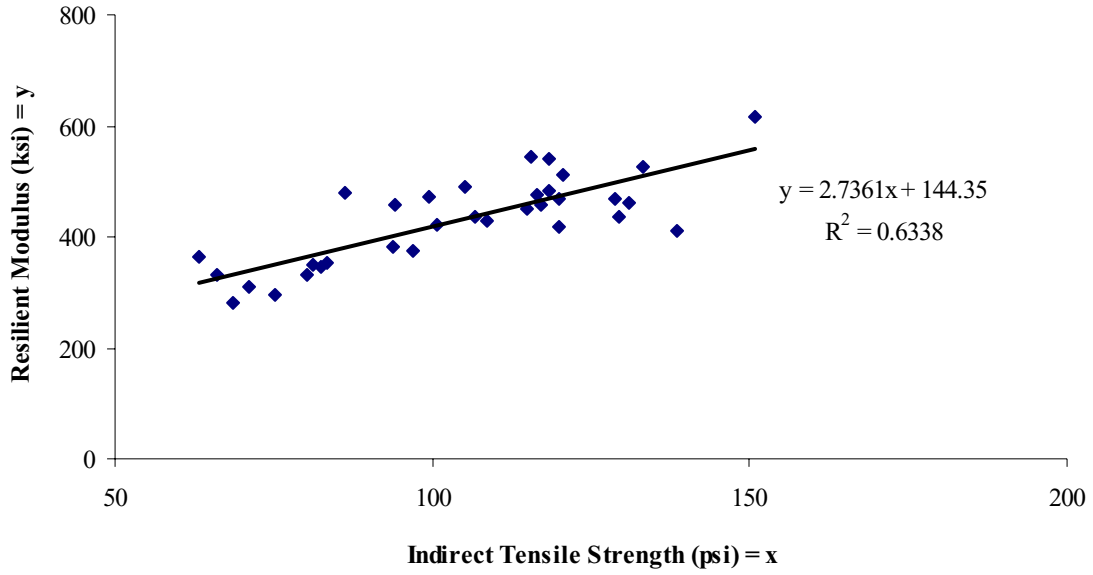


Figure 5.19 Resilient Modulus Vs. Indirect Tensile Strength at 25°C for Asphalt Concrete Material – Surface Layer (Original)

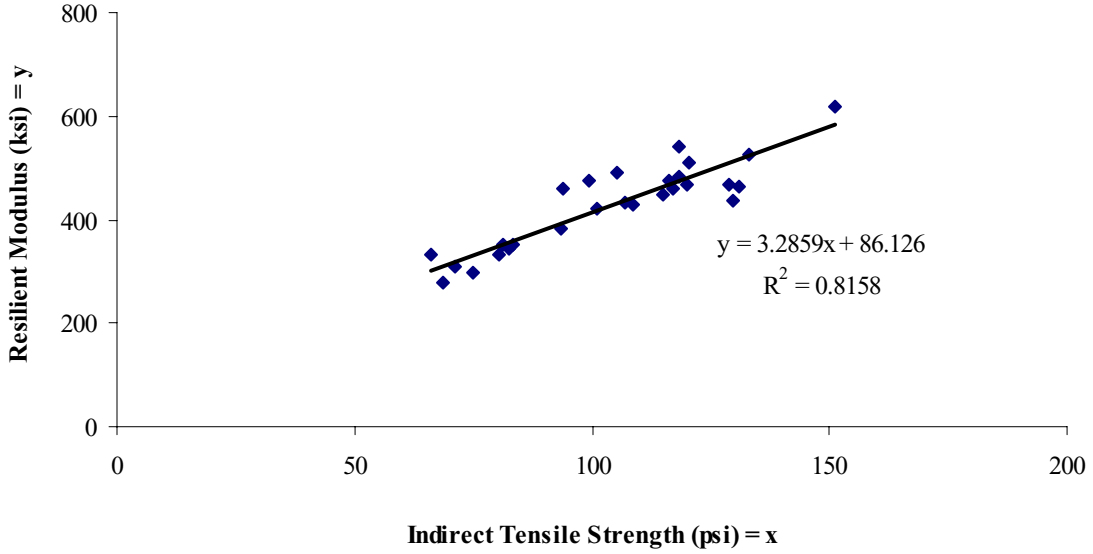


Figure 5.20 Resilient Modulus Vs. Indirect Tensile Strength at 25°C for Asphalt Concrete Material – Surface Layer (After Adjustment)

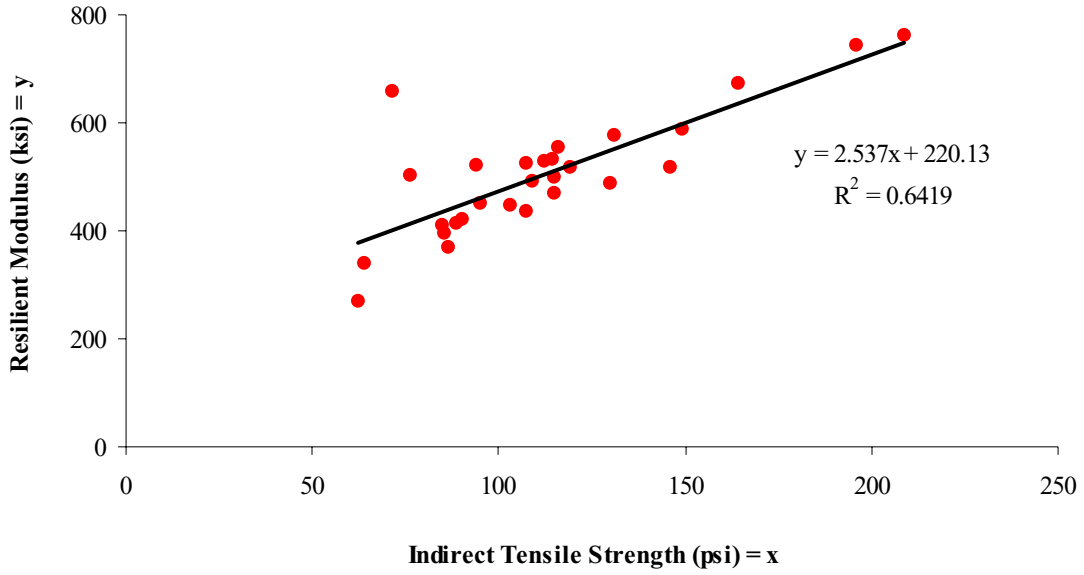


Figure 5.21 Resilient Modulus Vs. Indirect Tensile Strength at 25°C for Asphalt Concrete Material – Intermediate Layer (Original)

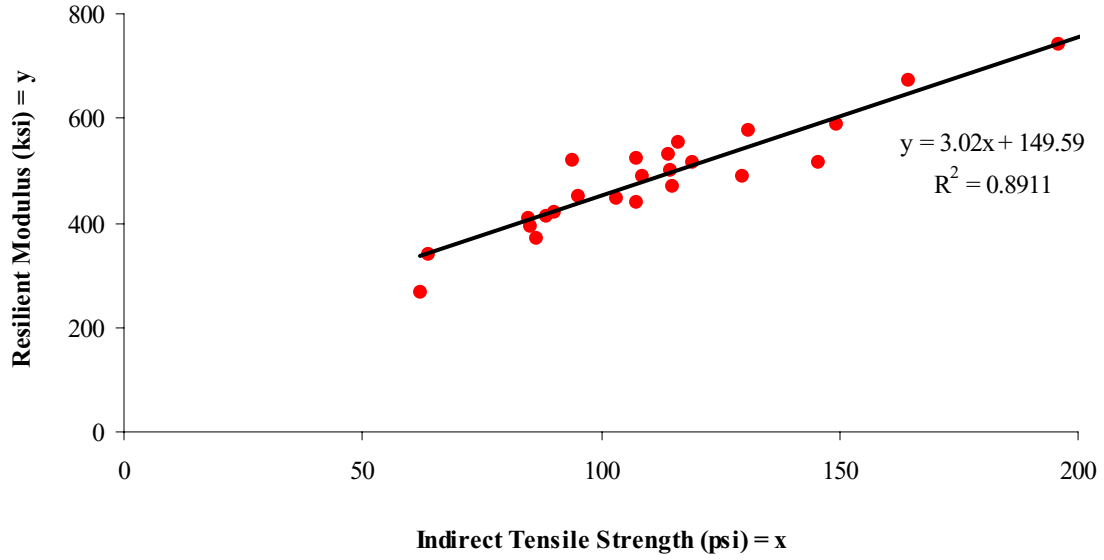


Figure 5.22 Resilient Modulus Vs. Indirect Tensile Strength at 25°C for Asphalt Concrete Material - Intermediate Layer (After Adjustment)

5.6.3 Creep Compliance Test

Twenty-three core specimens were taken from the Ohio-SHRP Test Road to measure the creep modulus of the asphalt concrete. This included nineteen specimens from the SPS-1 experiment sections (390101, 390104, 390106, 390107, 390111, 390112) and four specimens from the SPS-9 experiment sections (390901, 390902, 390903). Eleven of these specimens had to be trimmed in order to meet the standard dimension requirements of the SHRP P-06 Protocol, 4 inches (10.2 cm) in diameter by 4 inches (10.2 cm) in thickness. The remaining twelve core specimens were not trimmed, since their original thickness dimensions were less than 4 inches (10.2 cm). Because of this thickness requirement, each test specimen for the creep compliance test actually consisted of a composite of the surface and intermediate layer materials. This is a major difference between this test and the resilient modulus/indirect tensile strength test. In the latter, surface and intermediate layer specimens were tested separately.

The bulk specific gravity test was performed prior to the creep modulus test. The procedure for this test was the same as that utilized earlier to measure the bulk specific gravity of the ATB and AC core specimens. The bulk specific gravity results are presented in Table 5.31. The bulk specific gravity of the specimens ranged from 2.255 to 2.405, with an average of 2.293. The results are similar to those obtained previously as part of the resilient modulus testing. Specimens with bulk specific gravity values lower than the average were from Sections 390101, 390104, 390106, 390901, 390902, and 390903.

Table 5.31 Bulk Specific Gravity of Asphalt Concrete Specimens Used for Creep Modulus Test

SHRP ID	Bulk Specific Gravity
390101	2.287
390104	2.268
390106	2.286
390107	2.313
390111	2.405
390112	2.295
390901	2.255
390902	2.260
390903	2.266

The AC specimens were kept in the environmental chamber for at least 24 hours prior to testing at 41 and 77 °F (5 and 25 °C) and about 3 to 6 hours prior to testing at 104 and 140 °F (40 and 60 °C). Each of these specimens was subjected to a static load for a period of 60 minutes ± 15 seconds and then released for another 60 minutes, while the axial deformations were measured by the LVDTs.

Testing of the standard 4 inch thick specimens proceeded by the test protocol. The thin (less than 4 inches thick) non-standard specimens were tested with a solid steel block, 4 inches (10.16 cm) in diameter by 2.5 inches (6.35 cm) in height, inserted under the asphalt concrete specimen. This arrangement was necessary to make the vertical setting of the LVDTs possible. A series of calibration tests were conducted with the steel block to measure its deformation behaviors for the same duration of loading/unloading

under each test temperature. The deformation of the steel block was subtracted from the total deformation experienced by the AC/steel composite specimen to compute the creep modulus of the thin non-standard AC specimen. A specific magnitude of load was applied for each test temperature as described in Chapter 4.

A summary of the creep compliance test results is presented in Table 5.32 for the standard thickness specimens, and in Table 5.33 for the thin non-standard specimens. The relationship between the creep modulus values and the test temperatures are shown in Figures 5.23 and 5.24 for standard and non-standard thickness AC specimens, respectively.

Table 5.32 Creep Modulus Test Results on Standard Asphalt Concrete Specimens

Specimen ID	Time (sec)	Creep Modulus (psi) @			
		5°C	25°C	40°C	60°C
390101	1	1.400E+05	7.222E+04	5.210E+04	5.897E+04
	10	1.081E+05	4.480E+04	4.010E+04	4.773E+04
	100	7.226E+04	2.915E+04	3.189E+04	3.985E+04
	1000	3.772E+04	1.615E+04	2.272E+04	3.399E+04
	1800	3.204E+04	1.385E+04	2.021E+04	3.236E+04
	2700	2.890E+04	1.246E+04	1.856E+04	3.190E+04
	3600	2.694E+04	1.165E+04	1.758E+04	3.110E+04
390107	1	9.162E+04	7.540E+04	5.554E+04	5.598E+04
	10	6.857E+04	4.566E+04	4.101E+04	4.388E+04
	100	5.467E+04	3.014E+04	3.112E+04	3.628E+04
	1000	3.339E+04	1.668E+04	1.990E+04	3.112E+04
	1800	2.950E+04	1.424E+04	1.704E+04	2.993E+04
	2700	2.723E+04	1.288E+04	1.526E+04	2.945E+04
	3600	2.576E+04	1.196E+04	1.403E+04	2.870E+04
390112	1	1.333E+05	8.484E+04	5.268E+04	3.817E+04
	10	1.069E+05	5.776E+04	4.190E+04	2.236E+04
	100	7.678E+04	3.181E+04	2.708E+04	8.423E+03
	1000	4.469E+04	1.131E+04	1.092E+04	1.653E+03
	1800	3.886E+04	8.783E+03	8.225E+03	9.109E+02
	2700	3.536E+04	7.448E+03	6.848E+03	6.568E+02
	3600	3.303E+04	6.722E+03	5.890E+03	5.050E+02
390901	1	5.359E+04	7.219E+04	6.338E+04	5.529E+04
	10	5.083E+04	5.046E+04	4.269E+04	3.989E+04
	100	4.276E+04	3.141E+04	2.878E+04	2.703E+04
	1000	3.255E+04	1.459E+04	1.547E+04	1.422E+04
	1800	3.017E+04	1.185E+04	1.296E+04	1.177E+04
	2700	2.868E+04	1.032E+04	1.146E+04	1.034E+04
	3600	2.772E+04	9.370E+03	1.057E+04	9.441E+03

Table 5.32 Creep Modulus Test Results on Standard Asphalt Concrete Specimens (Continued)

Specimen ID	Time (sec)	Creep Modulus (psi) @			
		5°C	25°C	40°C	60°C
390902	1	1.429E+05	8.656E+04	4.473E+04	4.377E+04
	10	1.189E+05	5.927E+04	3.367E+04	3.614E+04
	100	8.638E+04	3.521E+04	2.568E+04	2.997E+04
	1000	5.205E+04	1.582E+04	1.518E+04	2.580E+04
	1800	4.603E+04	1.289E+04	1.292E+04	2.504E+04
	2700	4.251E+04	1.116E+04	1.158E+04	2.453E+04
	3600	4.025E+04	1.017E+04	1.074E+04	2.416E+04
390903	1	4.357E+04	8.046E+04	6.743E+04	6.823E+04
	10	4.169E+04	5.929E+04	4.857E+04	5.821E+04
	100	3.802E+04	4.022E+04	3.614E+04	4.700E+04
	1000	3.117E+04	2.108E+04	2.258E+04	3.933E+04
	1800	2.929E+04	1.756E+04	1.983E+04	3.752E+04
	2700	2.794E+04	1.563E+04	1.800E+04	3.606E+04
	3600	2.702E+04	1.436E+04	1.673E+04	3.450E+04
Average	1	1.008E+05	7.861E+04	5.598E+04	5.340E+04
	10	8.250E+04	5.287E+04	4.133E+04	4.137E+04
	100	6.181E+04	3.299E+04	3.012E+04	3.143E+04
	1000	3.860E+04	1.594E+04	1.780E+04	2.435E+04
	1800	3.432E+04	1.319E+04	1.520E+04	2.292E+04
	2700	3.177E+04	1.165E+04	1.362E+04	2.216E+04
	3600	3.012E+04	1.071E+04	1.259E+04	2.140E+04

Table 5.33 Creep Modulus Test Results on Thin Asphalt Concrete Specimens

Specimen ID	Time (sec)	Creep Modulus (psi) @			
		5°C	25°C	40°C	60°C
390101	1	3.309E+04	3.655E+04	2.681E+04	1.788E+04
	10	2.867E+04	2.743E+04	2.195E+04	1.623E+04
	100	2.363E+04	2.149E+04	1.979E+04	1.504E+04
	1000	1.704E+04	1.564E+04	1.764E+04	1.486E+04
	1800	1.555E+04	1.454E+04	1.711E+04	1.496E+04
	2700	1.465E+04	1.383E+04	1.685E+04	1.499E+04
	3600	1.408E+04	1.349E+04	1.663E+04	1.466E+04
390104	1	8.070E+04	4.242E+04	2.688E+04	1.556E+04
	10	6.196E+04	3.084E+04	2.266E+04	1.432E+04
	100	4.302E+04	2.219E+04	1.945E+04	1.200E+04
	1000	2.368E+04	1.384E+04	1.652E+04	1.197E+04
	1800	2.030E+04	1.230E+04	1.577E+04	1.201E+04
	2700	1.841E+04	1.133E+04	1.514E+04	1.215E+04
	3600	1.722E+04	1.081E+04	1.467E+04	1.215E+04
390106	1	6.715E+04	3.765E+04	2.269E+04	1.883E+04
	10	5.482E+04	2.836E+04	1.910E+04	1.552E+04
	100	4.067E+04	2.175E+04	1.648E+04	1.492E+04
	1000	2.469E+04	1.532E+04	1.415E+04	1.517E+04
	1800	2.160E+04	1.398E+04	1.359E+04	1.705E+04
	2700	1.964E+04	1.321E+04	1.335E+04	1.730E+04
	3600	1.850E+04	1.275E+04	1.307E+04	1.794E+04
390107	1	7.641E+04	3.888E+04	2.761E+04	2.346E+04
	10	6.171E+04	2.900E+04	2.322E+04	2.125E+04
	100	4.585E+04	2.229E+04	2.100E+04	1.897E+04
	1000	2.781E+04	1.576E+04	1.898E+04	1.819E+04
	1800	2.403E+04	1.450E+04	1.859E+04	1.978E+04
	2700	2.211E+04	1.373E+04	1.812E+04	2.107E+04
	3600	2.057E+04	1.341E+04	1.781E+04	2.182E+04

Table 5.33 Creep Modulus Test Results on Thin Asphalt Concrete Specimens (Continued)

Specimen ID	Time (sec)	Creep Modulus (psi) @			
		5°C	25°C	40°C	60°C
390111	1	8.514E+04	3.500E+04	2.307E+04	3.178E+04
	10	6.902E+04	2.668E+04	1.935E+04	2.775E+04
	100	4.796E+04	2.115E+04	1.687E+04	2.389E+04
	1000	2.641E+04	1.515E+04	1.538E+04	2.741E+04
	1800	2.262E+04	1.382E+04	1.482E+04	3.082E+04
	2700	2.056E+04	1.299E+04	1.459E+04	3.006E+04
	3600	1.923E+04	1.252E+04	1.444E+04	3.244E+04
390112	1	5.887E+04	3.511E+04	3.322E+04	1.854E+04
	10	4.950E+04	2.675E+04	1.887E+04	1.622E+04
	100	3.872E+04	2.086E+04	2.109E+04	1.451E+04
	1000	2.511E+04	1.449E+04	1.793E+04	1.513E+04
	1800	2.223E+04	1.313E+04	1.676E+04	1.498E+04
	2700	2.062E+04	1.222E+04	1.609E+04	1.549E+04
	3600	1.948E+04	1.188E+04	1.576E+04	1.517E+04
Average	1	6.689E+04	3.760E+04	2.671E+04	2.101E+04
	10	5.428E+04	2.818E+04	2.086E+04	1.855E+04
	100	3.998E+04	2.162E+04	1.911E+04	1.655E+04
	1000	2.412E+04	1.503E+04	1.677E+04	1.712E+04
	1800	2.105E+04	1.371E+04	1.611E+04	1.826E+04
	2700	1.933E+04	1.288E+04	1.569E+04	1.851E+04
	3600	1.818E+04	1.248E+04	1.540E+04	1.903E+04

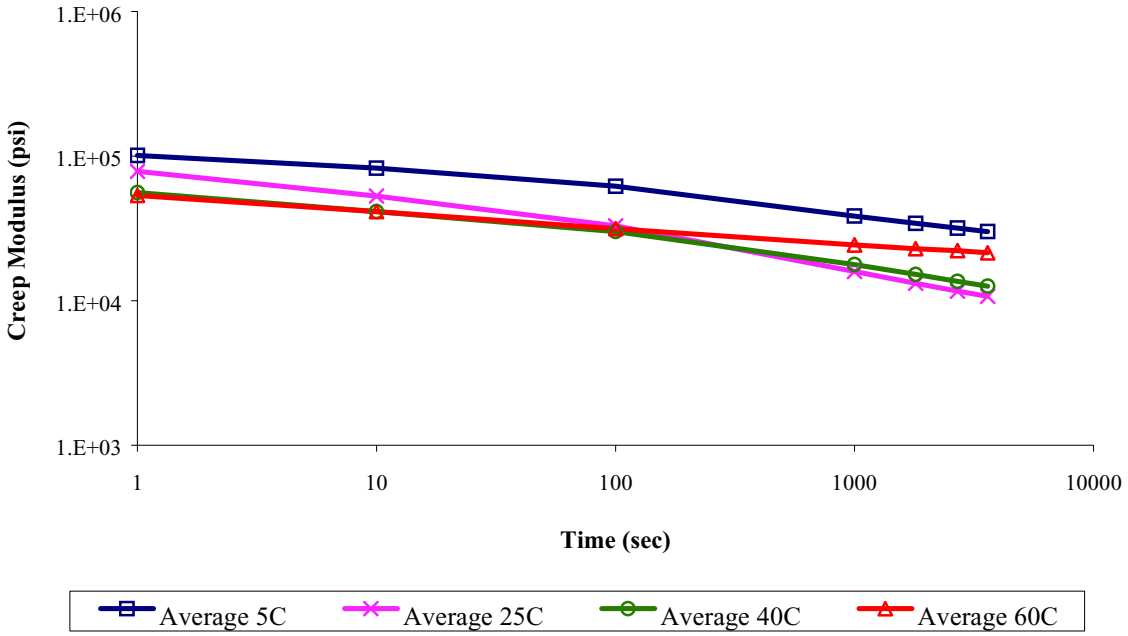


Figure 5.23 Time Vs. Creep Modulus Relationship for Standard Asphalt Concrete Specimens

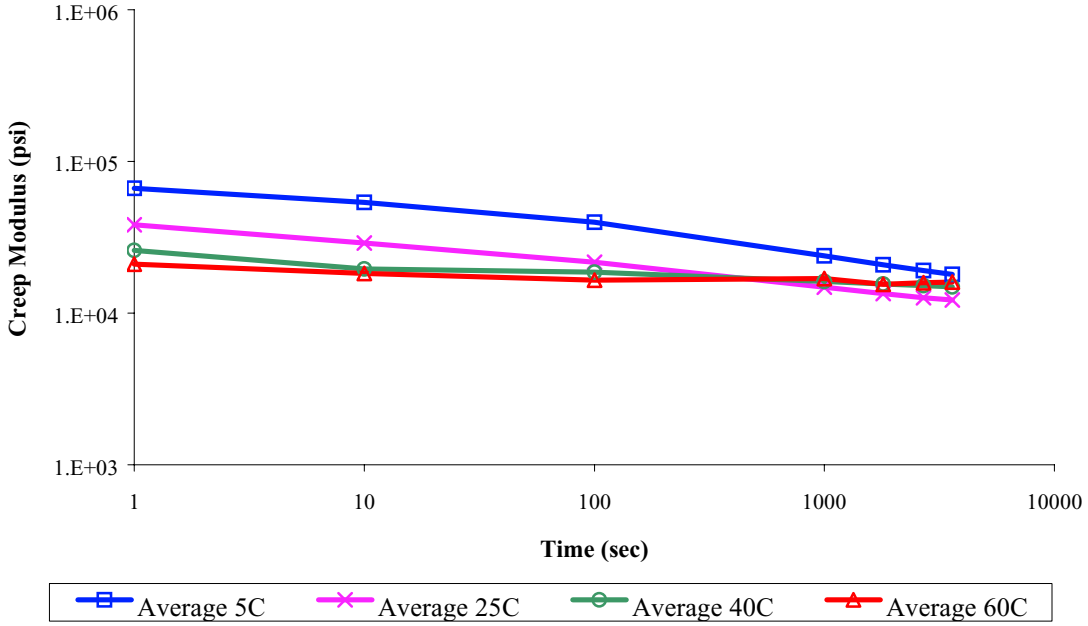


Figure 5.24 Time Vs. Creep Modulus Relationship for Thin Asphalt Concrete Specimens

Creep modulus varied between 0.5 million psi and 178 million psi (3.5 and 12204 MPa) for the standard thickness specimens and between 6.1 and 88 million ksi (42.1 and 606.7 MPa) for the thin non-standard specimens. As seen in Figures 5.26 and 5.27, both test temperature and duration of the loading time had a significant influence on the creep modulus test results. The data presented in the tables and figures indicate an overall trend of lower creep modulus values with higher temperature and longer loading time.

As expected, the creep modulus of the standard thickness asphalt concrete decreased as the loading time increased. The creep modulus measured by the start of loading was reduced on the average by 70 and 86% at 41 and 77 °F (5 and 25 °C), respectively, at the end of the loading period. The average percent reductions in the creep modulus at the other two test temperatures were 78% at 104 °F (40 °C) and 60% at 140 °F (60 °C), respectively. The different degrees of creep observed among these test temperature settings were due to the fact that an increasingly lower axial stress was applied to the test specimen under higher temperature. For the thin non-standard specimens, similar reductions in the creep moduli were observed. The creep modulus decreased by 73 and 68% at 41 and 77 °F (5 and 25 °C), respectively, when compared to the modulus measured at the initial time. Nevertheless, the modulus at 140 °F (60 °C) first decreased by 22% and then increased by 2.7% after 100 seconds of constant loading. Viscous properties of the asphalt binder material, increasingly imperfect shapes of the test specimens, and gradual compaction of the aggregate component might have caused the deviation from the expected trend.

It is possible to perform the creep test on the thin non-standard AC specimens. However, more care is needed when handling these specimens to assure the quality of the test outcome; since the adverse effect of uneven deformations experienced by the test specimen on the final creep modulus value is more pronounced for these specimens. It is therefore understandable why the SHRP Protocol requires a specimen thickness of about 4 inches (10.2 cm).

CHAPTER 6:
DEVELOPMENT OF DATABASE

(Users' Manual)

6.1. Introduction

The Ohio-SHRP test project was a very large project, encompassing four SPS experimental studies. The actual management required the active involvement of research teams from six universities in Ohio, and the original project management plan divided responsibilities among the universities as follows:

Ohio University ---------- Project Team Leader & Coordinator
Load Response Instrumentation & Data
The Ohio State University ---------- Seasonal & Soil Suction Instrumentation
University of Akron ---------- Pressure Cell Calibration and Installation
University of Cincinnati ---------- Joint Movement Instrumentation & Data
Case Western Reserve University --- Weather Station Installation & Data
University of Toledo ----------Seasonal & Soil Suction Instrumentation

A consequence of such a complicated management structure was that data compiled for the Ohio-SHRP project became vast and scattered among the several participants. The ODOT and Ohio University have been realizing a need to integrate and centralize all the data for future research reference. There are efforts going on at Ohio University to

assemble all the data collected on the Ohio-SHRP project would be combined into a computer database. The database attached to this report is still in its very initial stages and contain data mostly collected by the ODOT and Ohio University.

6.2. Database

A database for the Ohio-SHRP project was developed during the summer and fall of 2000 through a joint effort between the Civil Engineering Department and the Electrical Engineering/Computer Science Department at Ohio University. At the initial stage of the database project, the joint team decided to produce the database in the form of a CD-ROM disk. Visual Basic™ was used to set up each frame, interfaces among the frames, and between the frames and data files. Microsoft Access™ was used to construct each data file.

The database contained in the CD-ROM disk runs on any IBM PC compatible with Windows 95/98/NT/2000 and a CD-ROM drive. For optimum viewing, the screen resolution must be set at the standard 1024 by 768 pixels. The following steps must be taken to install the database on any computer:

1) Insert the CD-ROM disk into the CD-ROM disk drive.
2) Wait until a proper name appears for the CD-ROM disk icon. Click on the CD-ROM disk icon.
3) Click on the Setup icon within the folder. Wait for a few seconds.
4) Begin the installation process by pressing on a large square PC icon.

5) Follow instructions on the screen. Choose 'O.K.' to the questions.

6) See that the installation is done successfully.

7) Go back to the window main screen.

8) Scroll up a program list. Select a program 'orite' and click on it.

9) Click on an icon named 'USRT23" to go to the main screen of the database.

10) Click on available buttons on the screen to view various general and material property data for the U.S. Rt. 23 project

The Ohio SHRP Test Road database is divided into three major sections:

1- General project information section

2- Specific pavement selection section

3- Information/data review section

These sections interface smoothly, so that the user can go back and forth among them or move to different locations within each section with no technical difficulty. Once installed, the user can print out any of the frames appearing on the monitor screen by pressing "Print Screen" button on the computer keyboard and then selecting a proper notepad to paste the screen image for printing.

The first section is devoted to the presentation of general project information. This section consists of four frames, and presents to the scope of the project, the project

location, and background information, such as construction, climate, and traffic data. Contents of these four frames in the first section are listed below:

Frame 1 (General Information) – Title of database, aerial photo of the Ohio-SHRP test road, logos of the FHWA, ODOT, and ORITE, as well as a contact information. See Figure 6.1.

Frame 2 (General Information) – Site location information. See Figure 6.2

Frame 3 (General Information) – General layout of the Ohio-SHRP test road. See Figure 6.3.

Frame 4 (General Information) – A folder containing a detailed site layout map (see Figure 3.1) and pavement design information (Tables 3.1 & 3.2).

The second major section has two frames and is designed to let the user select a particular pavement section. The first frame (shown in Figure 6.4) of this section is called the "SHRP Section Selection" frame. Here, the user must narrow down his/her focus to a specific SPS experiment by clicking on either SPS-1, 2, 8, or 9 bullet. Then, a dynamic table can be scrolled down to reveal all available SHRP sections under the selected SHRP experiment. The user can also select a specific section of interest by clicking on one of the section icons shown on the Ohio-SHRP Test Road site layout map, Frame 3 of the first (general information) section. Once a particular SHRP section is chosen, the user will be directed automatically to the second frame (seen in Figure 6.5). This frame is a folder containing three items (basic information, layer and construction

Figure 6.1 Welcome Page of Database for the Ohio-SHRP Test Road

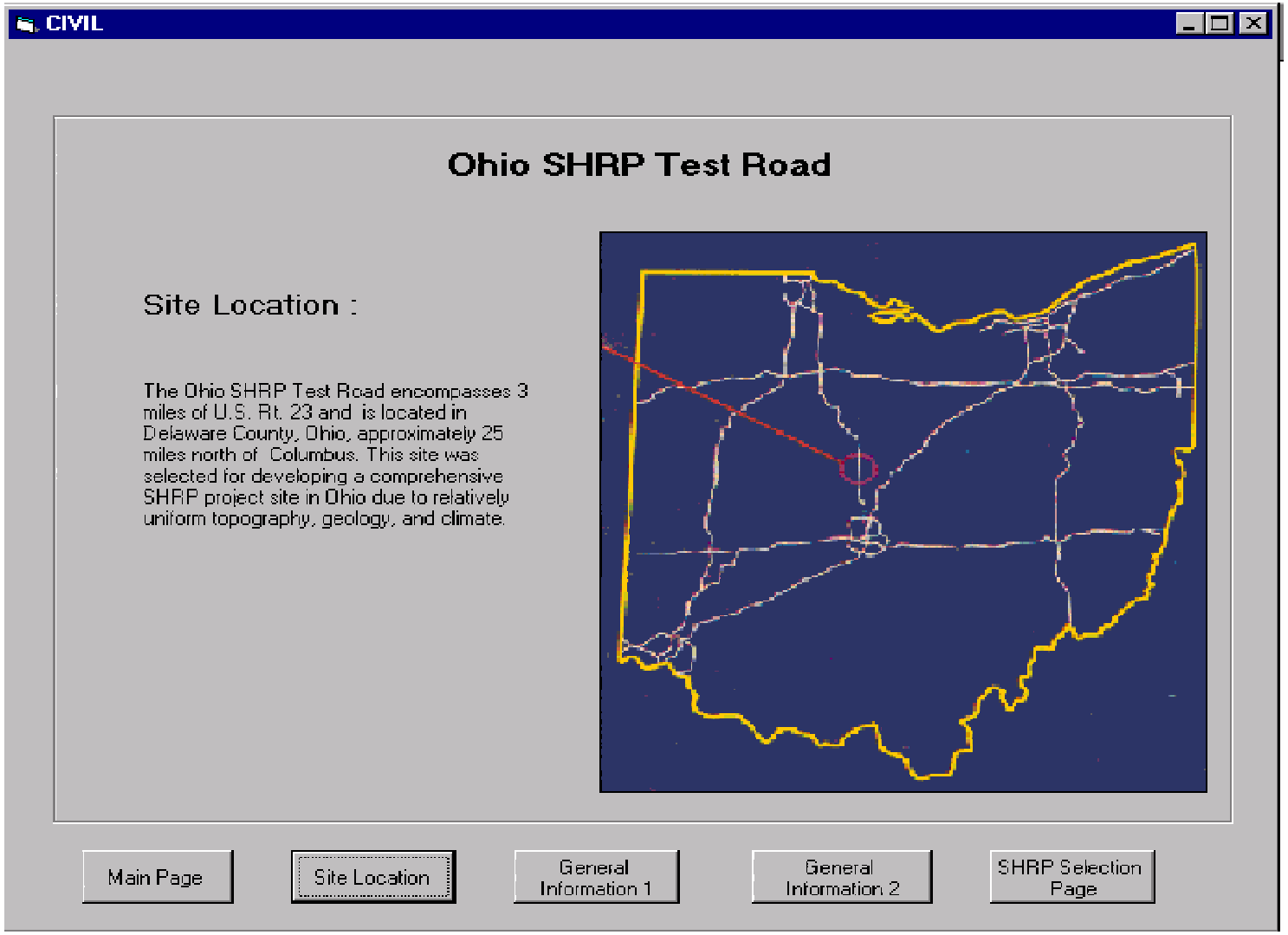


Figure 6.2 Site Location of the Ohio-SHRP Test Road

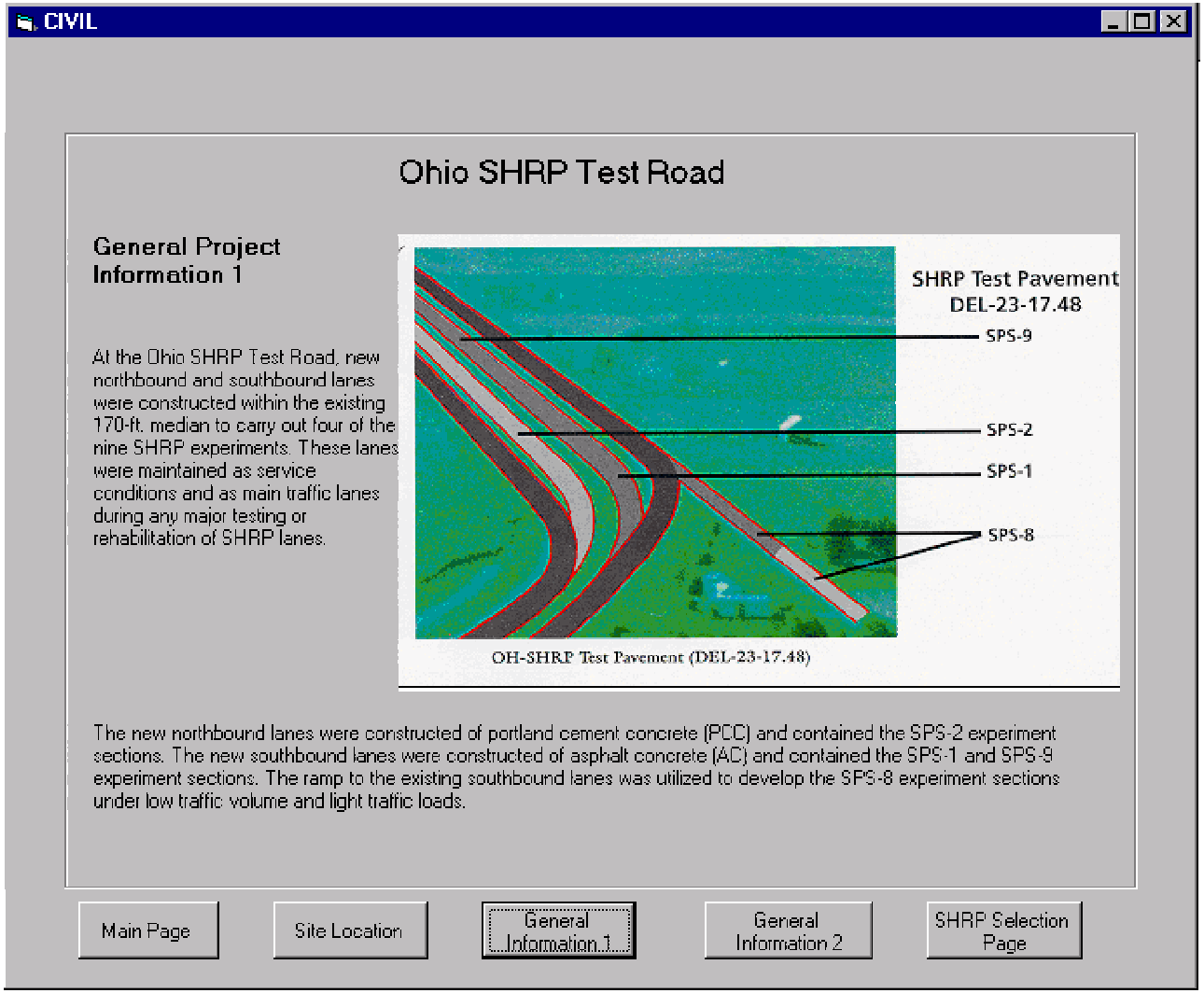


Figure 6.3 General Project Information for the Ohio-SHRP Test Road

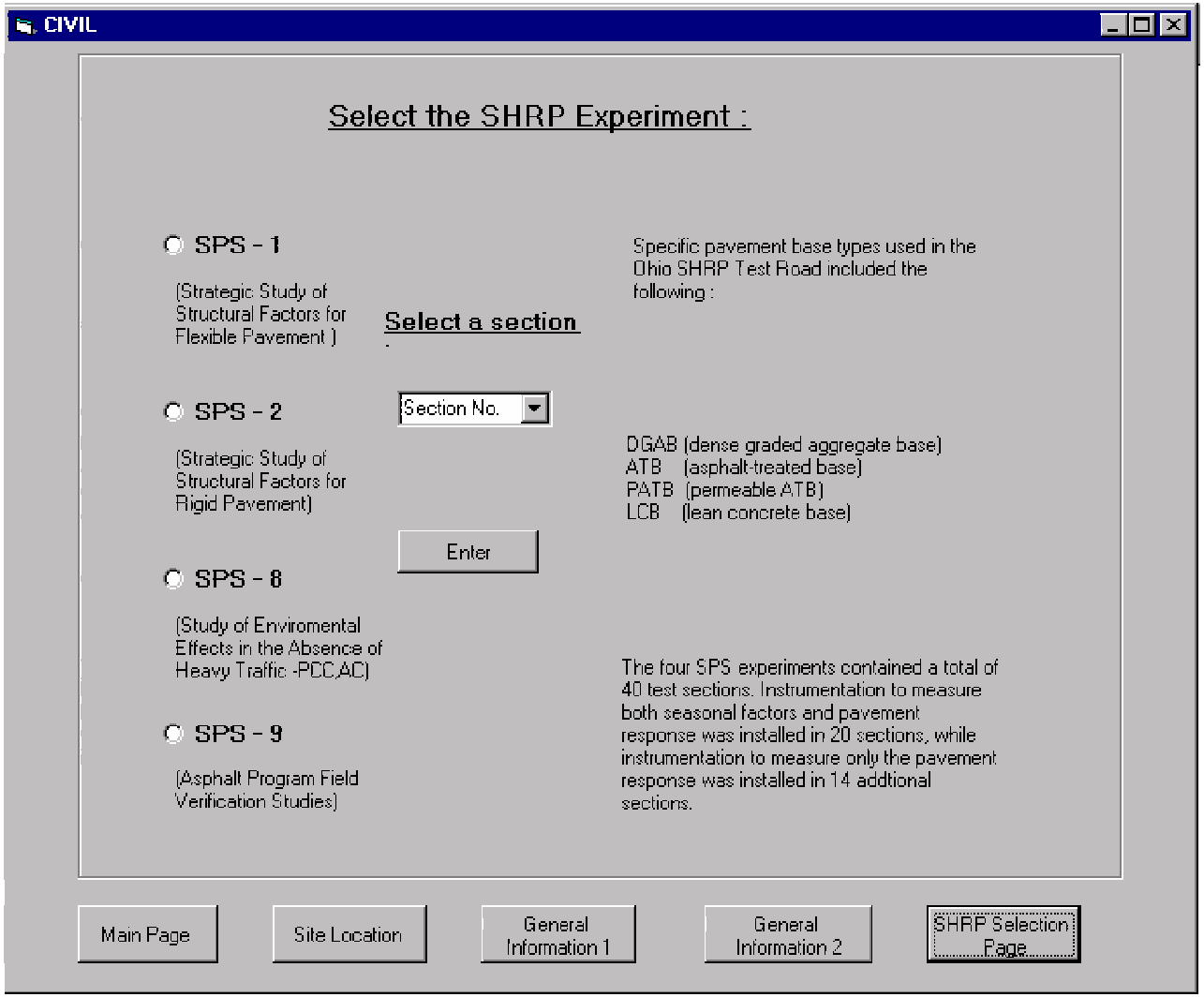


Figure 6.4 SHRP Experiment/Section Selection Page

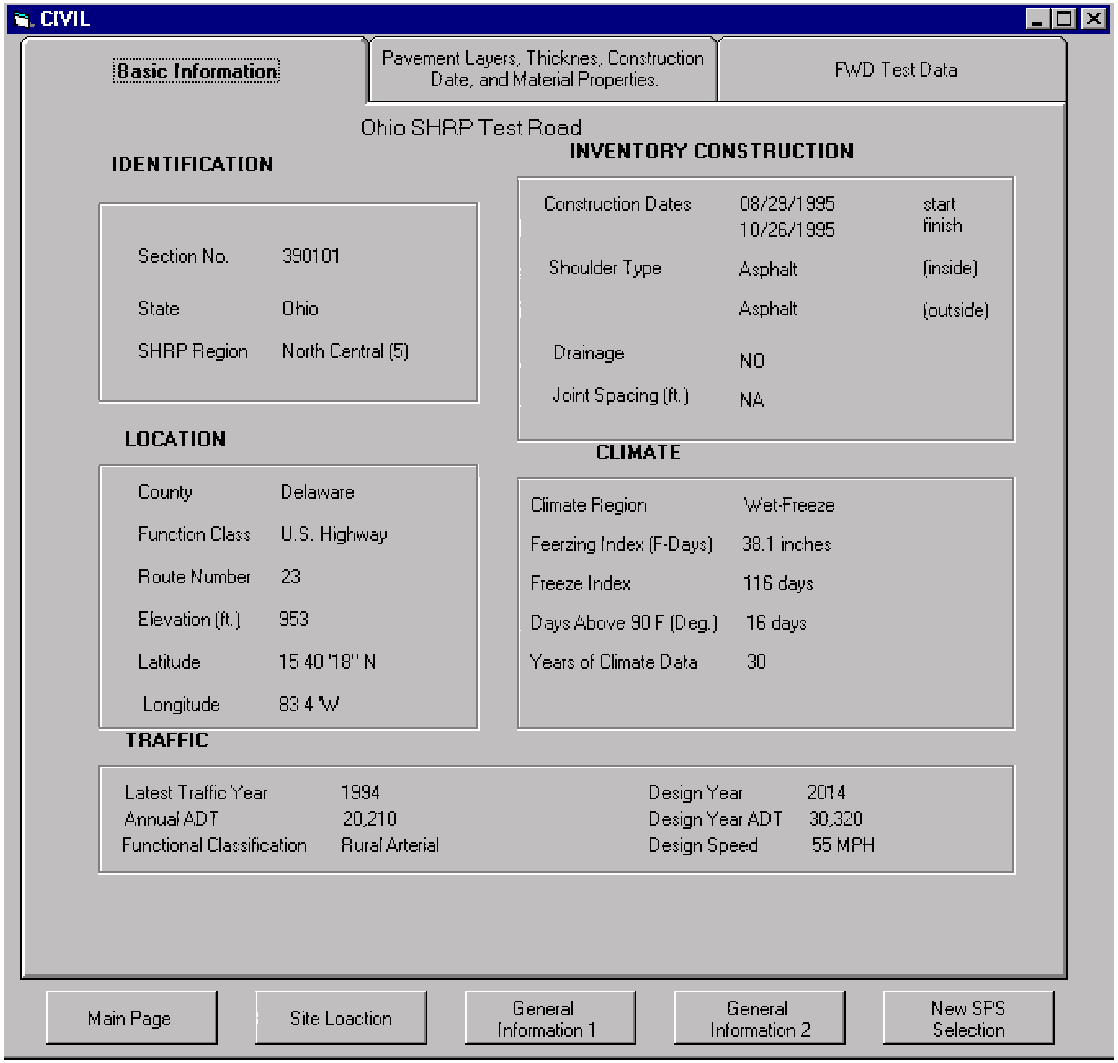


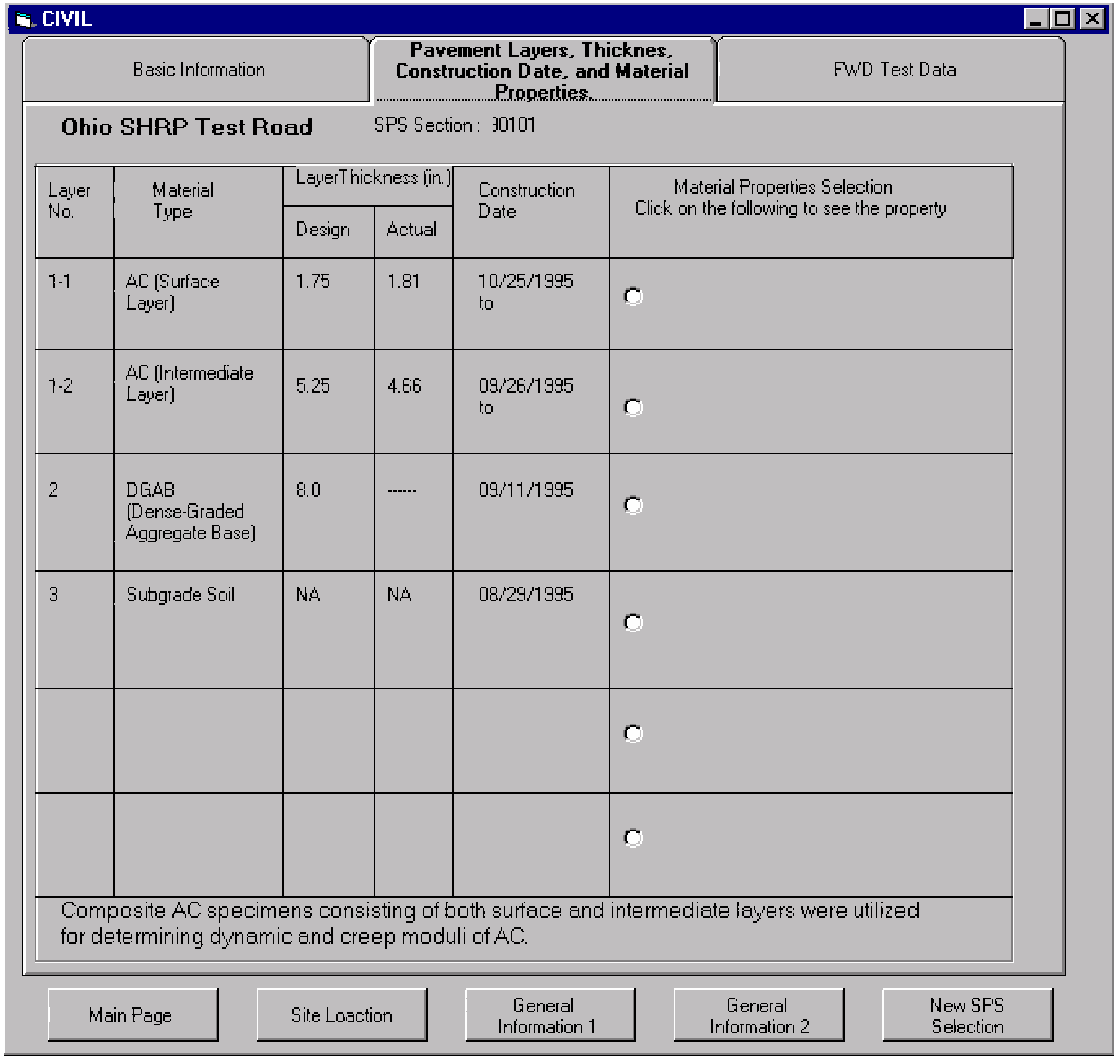
Figure 6.5 Basic Information for Ohio-SHRP Test Road Section

data, and FWD test data). Under the first item (basic information), the design and actual thickness data, as well as the construction dates for each layer can be reviewed for the selected section. This is shown in Figure 6.5. The second item, shown in Figure 6.6, presents a complete vertical profile (top to bottom) of the pavement section selected by the user. This frame also has bullets that lead to specific material properties of each layer shown in the frame. The user is taken to the third section of the database as soon as one of the pavement layers is selected. The third item (FWD test data) has not been constructed (so it does not provide any data) but left here for a possible future expansion.

The third major section constitutes the heart of this computer database and permits a review of the material specifications, construction data, and material properties for the pavement section/material selected in the second stage. The list of material properties available in the database is quite extensive, and it contains all of the test results reported in the current project report and more.

The most detailed material property presentation is available for the asphalt concrete (AC) material, which is grouped into four categories: basic (Figure 6.7), resilient modulus/indirect tension (Figure 6.8), dynamic modulus, and creep modulus. Material properties listed under the "Basic Properties" include:

- Design specifications
- Construction dates
- Specific gravity values (bulk, maximum)
- % air voids



CIVIL

Basic Information | **Pavement Layers, Thicknes, Construction Date, and Material Properties** | FWD Test Data

Ohio SHRP Test Road SPS Section : 30101

Layer No.	Material Type	LayerThickness (in.)		Construction Date	Material Properties Selection Click on the following to see the property
		Design	Actual		
1-1	AC (Surface Layer)	1.75	1.81	10/25/1995 to	
1-2	AC (Intermediate Layer)	5.25	4.66	09/26/1995 to	
2	DGAB (Dense-Graded Aggregate Base)	8.0	------	09/11/1995	
3	Subgrade Soil	NA	NA	08/29/1995	

Composite AC specimens consisting of both surface and intermediate layers were utilized for determining dynamic and creep moduli of AC.

Main Page | Site Loaction | General Information 1 | General Information 2 | New SPS Selection

Figure 6.6 Pavement Layers, Thickness, Construction Date, and Material Properties Information

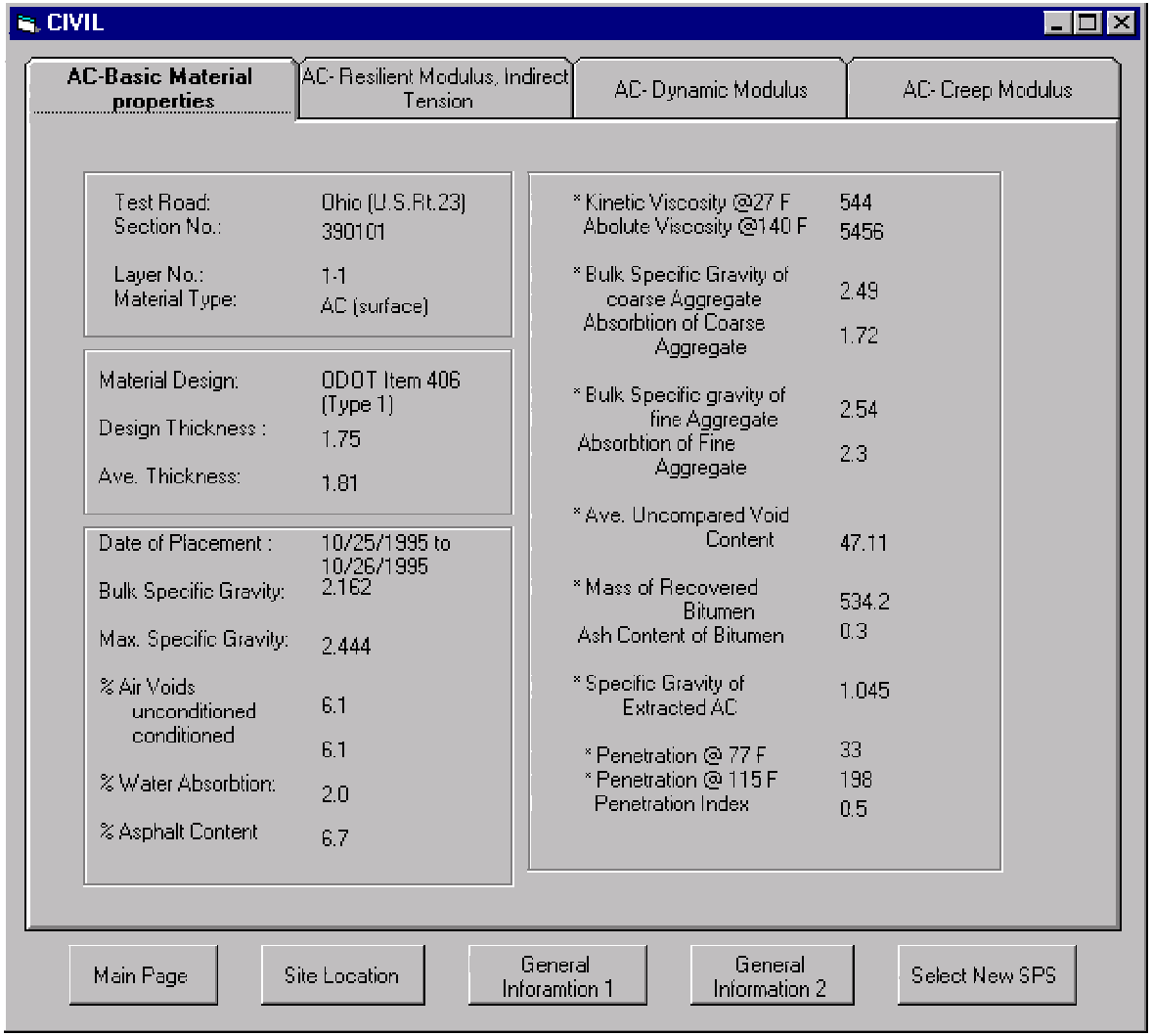
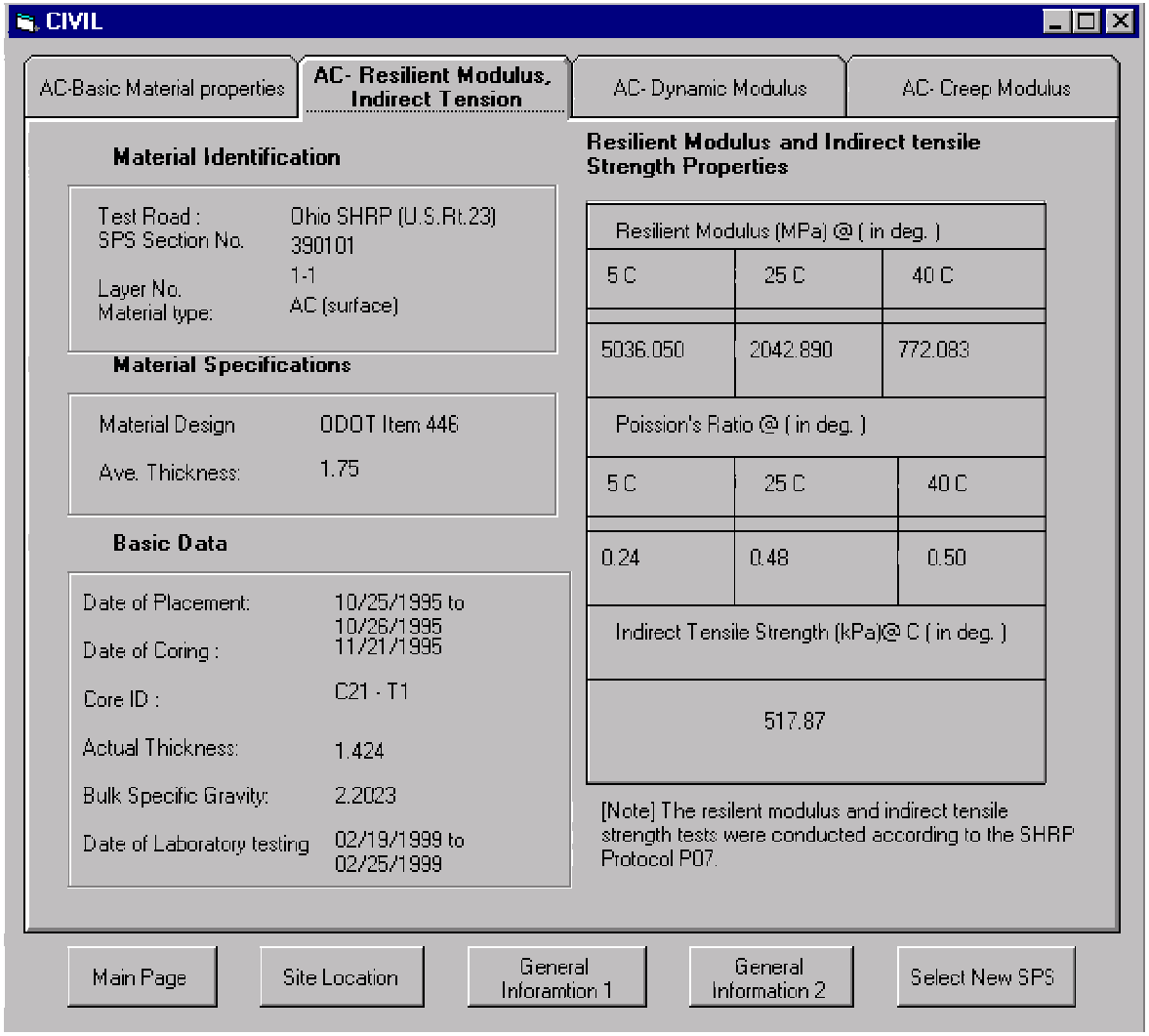


Figure 6.7 Basic Material Properties for Asphalt Concrete (AC)



CIVIL

AC-Basic Material properties | **AC- Resilient Modulus, Indirect Tension** | AC- Dynamic Modulus | AC- Creep Modulus

Material Identification

Test Road : Ohio SHRP (U.S.Rt.23)
SPS Section No. 390101
Layer No. 1-1
Material type: AC (surface)

Material Specifications

Material Design ODOT Item 448
Ave. Thickness: 1.75

Basic Data

Date of Placement: 10/25/1995 to 10/26/1995
Date of Coring : 11/21/1995
Core ID : C21 - T1
Actual Thickness: 1.424
Bulk Specific Gravity: 2.2023
Date of Laboratory testing 02/19/1999 to 02/25/1999

Resilient Modulus and Indirect tensile Strength Properties

Resilient Modulus (MPa) @ (in deg.)		
5 C	25 C	40 C
5036.050	2042.890	772.083
Poission's Ratio @ (in deg.)		
5 C	25 C	40 C
0.24	0.48	0.50
Indirect Tensile Strength (kPa)@ C (in deg.)		
517.87		

[Note] The resilent modulus and indirect tensile strength tests were conducted according to the SHRP Protocol P07.

Main Page | Site Location | General Inforamtion 1 | General Information 2 | Select New SPS

Figure 6.8 Resilient Modulus and Indirect Tension Information for Asphalt Concrete (AC)

- % water absorption
- Asphalt content
- Viscosity of asphalt (kinematic, absolute)
- Bulk specific gravity of coarse aggregate
- Bulk specific gravity of fine aggregate
- Mass of extracted bitumen
- Ash content of bitumen
- Penetration test results

The “Resilient Modulus” page presents:

- Core I.D.
- Date of coring
- Actual core thickness
- Bulk specific gravity of the core
- Date of laboratory testing
- Resilient modulus at 5, 25, and 40 ºC (41, 77, 104 ºF)
- Poisson’s ratio at 5, 25, and 40 ºC (41, 77, 104 ºF)
- Indirect tensile strength at 25 ºC (77 ºF)

The “Dynamic Modulus” page lists:

- Core I.D.
- Date of coring

- Actual core thickness
- Bulk specific gravity of the core
- Date of laboratory testing
- Dynamic modulus and phase angle at 1, 4, and 16 Hz. loading frequencies and at 5, 25, and 40 ºC (41, 77, and 104 ºF)

The last page in the AC folder presents:

- Core I.D.
- Date of coring
- Actual core thickness
- Bulk specific gravity of the core
- Date of laboratory testing
- Creep modulus at 5, 25, 40, and 60 ºC (41, 77, 104, and 140 ºF)

Comprehensive Portland cement concrete (PCC) properties are available within a single frame and span from unit weight to various strength properties. The data available within the frame for the PCC (Figure 6.9) are:

- Mix design specifications
- Design and actual thickness
- Actual slump, air content, and unit weight
- Construction dates
- Compressive strength at 14 days, 28 days, and 1 year

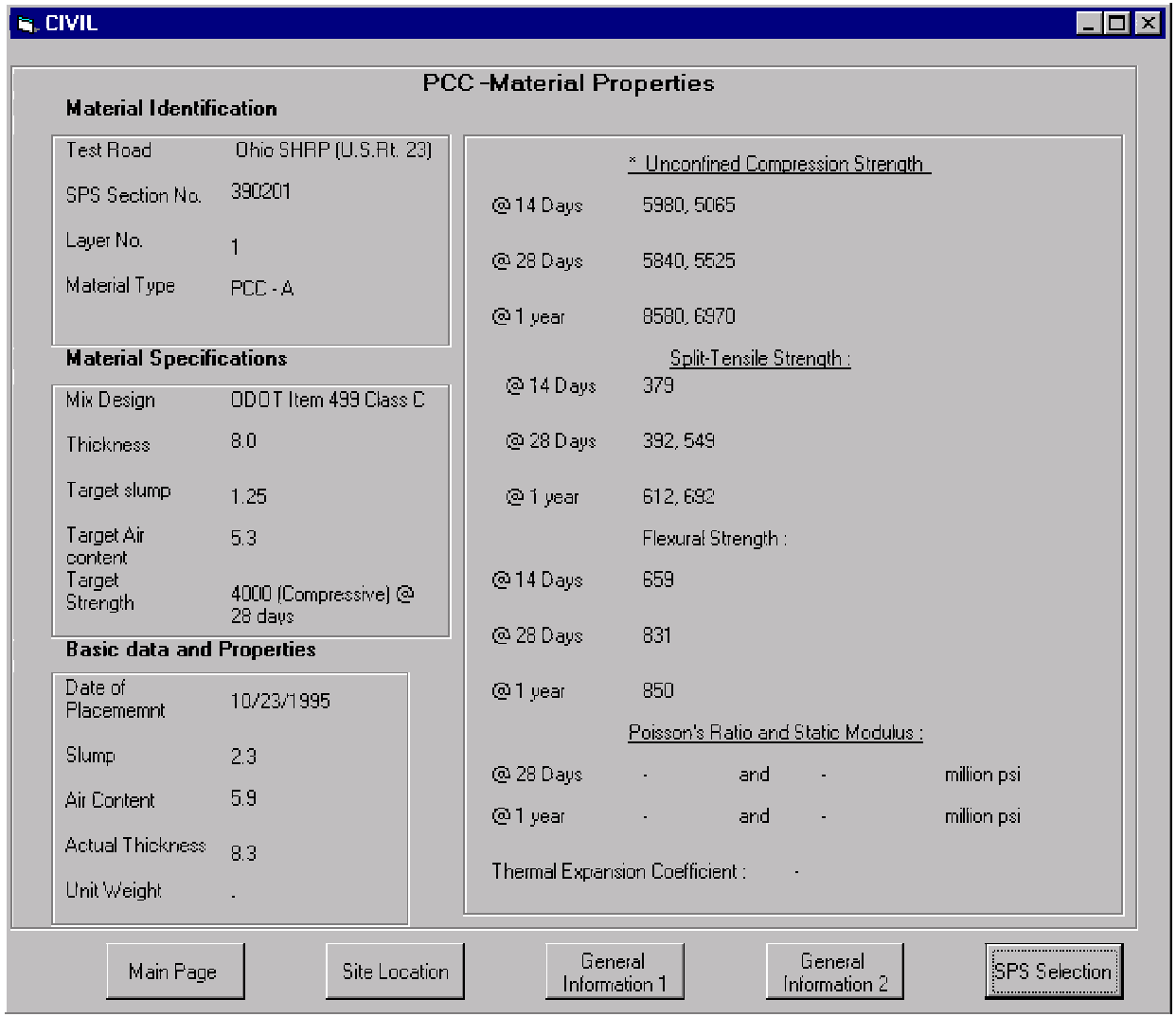


Figure 6.9 Material Properties for Portland Cement Concrete (PCC)

- Split tensile strength at 14 days, 28 days, and 1 year (cores only)
- Modulus of rupture at 14 days, 28 days, and 1 year
- Static modulus and Poisson's ratio at 28 days and 1 year (cores only)
- Thermal expansion coefficient (typical value)

At least one frame is devoted to each of the unbound or stabilized bases utilized in the field. For example, the frame on the PATB (Figure 6.10) lists:

- Mix design information
- Construction dates
- Actual thickness (average)

The following list of material properties can be viewed for the ATB (see Figure 6.11):

- Mix design information
- Construction dates
- Actual thickness (average, range)
- Unit weight
- Bulk specific gravity
- Indirect tensile strength at 25 ºC (77 ºF)
- Resilient modulus and Poisson's ratio at 5, 25, and 40 ºC (41, 77, and 104 ºF)

Two pages developed for the DGAB (see Figures 6.12 and 6.13) provide:

- Design specifications

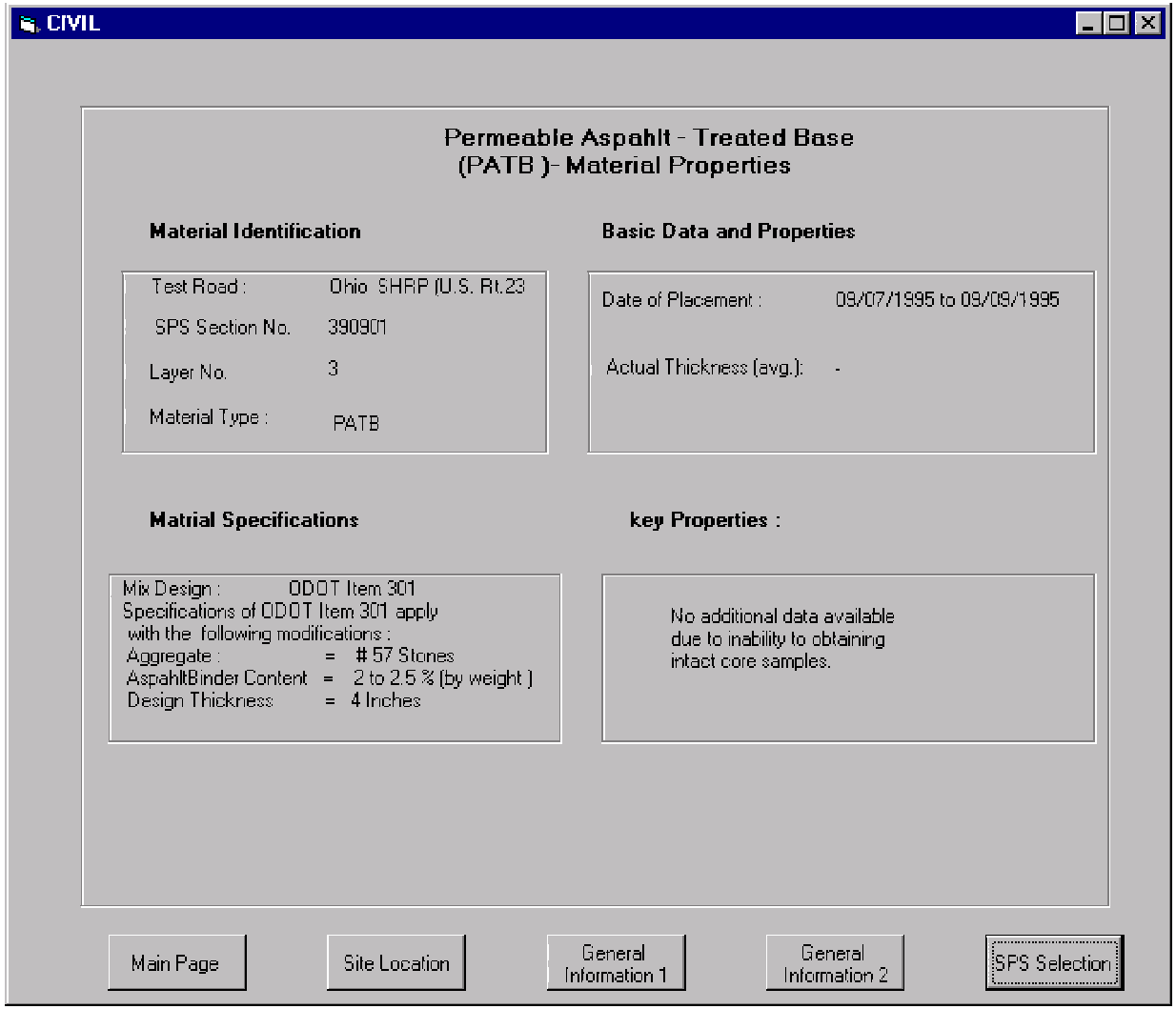


Figure 6.10 Material Properties for Permeable Asphalt-Treated Base (PATB)

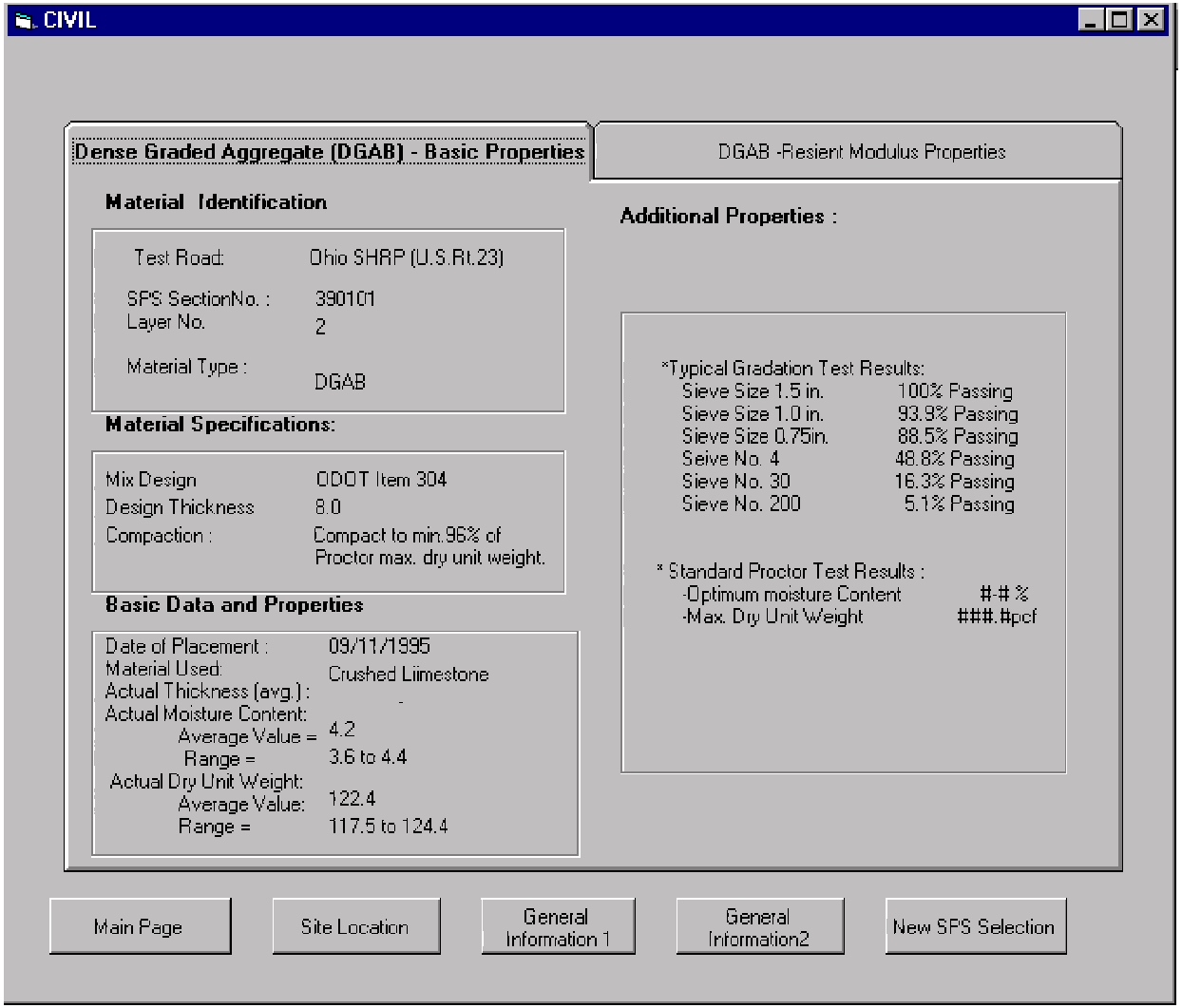


Figure 6.11 Material Properties for Asphalt-Treated Base (ATB)

CIVIL

Dense Graded Aggregate (DGAB) - Basic Properties | DGAB -Resient Modulus Properties

Material Identification :

Test Road: Ohio SHRP (U.S. Rt.23)

SPS Selection No.: 390101

Layer No.: 2

Material Type: DGAB

Material Specifications:

Mix Design : ODOT Item 304

Design Thickness: 8.0

Compaction : Compact to min. 96% of Proctor max. dry unit weight

Typical Resilient Modulus Test Results

A total of four tests were performed in the laboratory according to the SHRP P46 Protocol .

Test No.	Moisture Content (%)	Dry Weight (pcf)	Permanent Strain (%)	Parameters:	
				K	n
1	1.68	114.7	0.72	6293.5	0.270
2	2.14	114.0	0.56	5618.6	0.305
3	5.08	115.2	0.49	3402.0	0.429
4	6.77	113.9	0.80	4319.4	0.340
Ave.	3.92	114.5	0.64	4908.4	0.336

Main Page | Site Location | General Information 1 | General Information2 | New SPS Selection

Figure 6.12 Material Properties for Dense Graded Aggregate Base (DGAB)

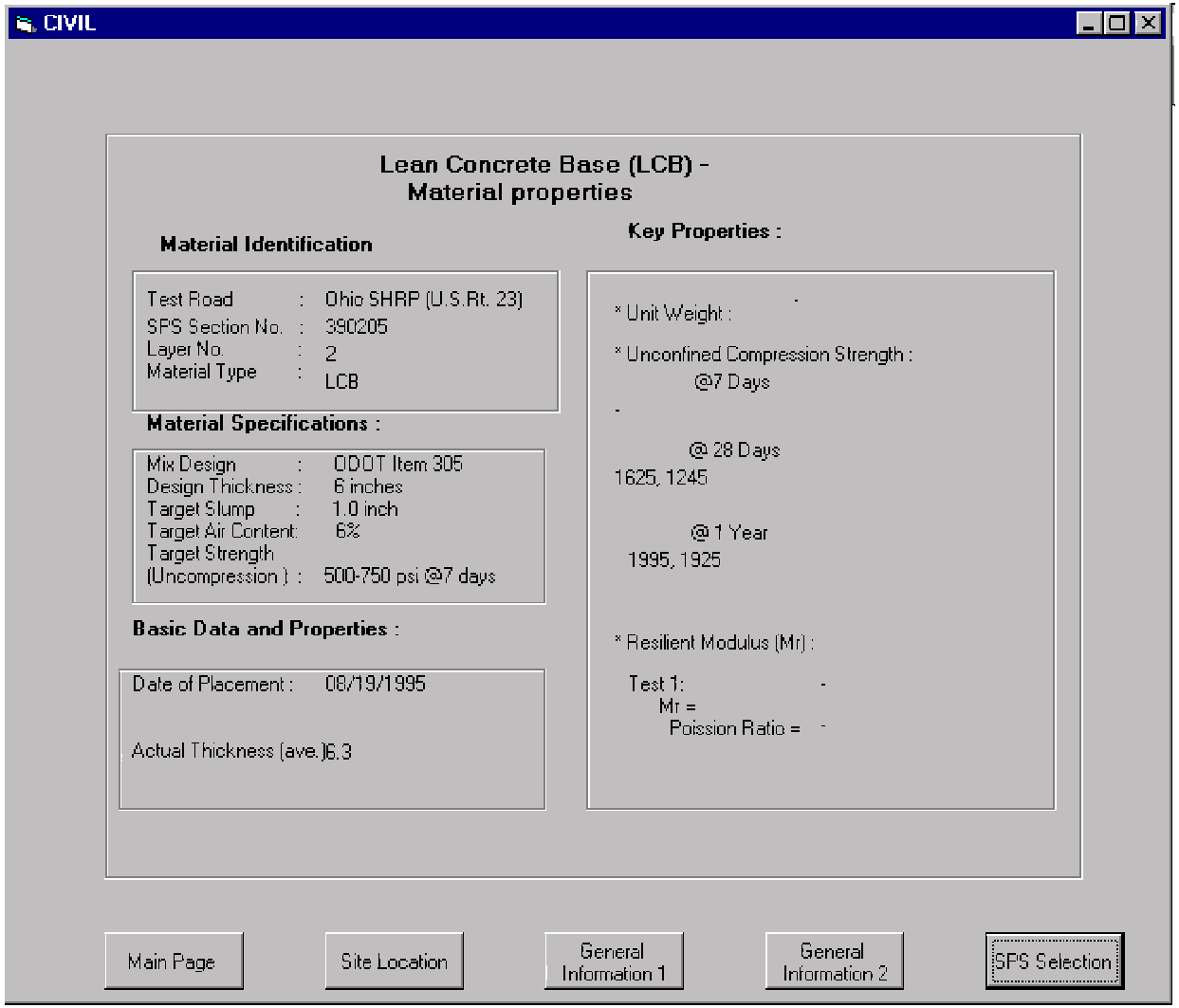


Figure 6.13 Resilient Modulus Information for Dense Graded Aggregate Base (DGAB)

- Construction dates
- Actual thickness
- In-situ moisture and density conditions (accepted values during construction)
- Typical gradation test results
- Laboratory resilient modulus test results

The page (see Figure 6.14) on the lean concrete base (LCB) contains:

- Mix design information
- Construction dates
- Actual thickness (average)
- Unit weight
- Unconfined compression strength at 7 days, 28 days, and 1 year
- Resilient modulus and Poisson's ratio (typical)

The frame (see Figure 6.15) constructed for the PCTB presents:

- Mix design information
- Construction dates
- Actual thickness (average)
- Unit weight
- Resilient modulus and Poisson's ratio (typical)

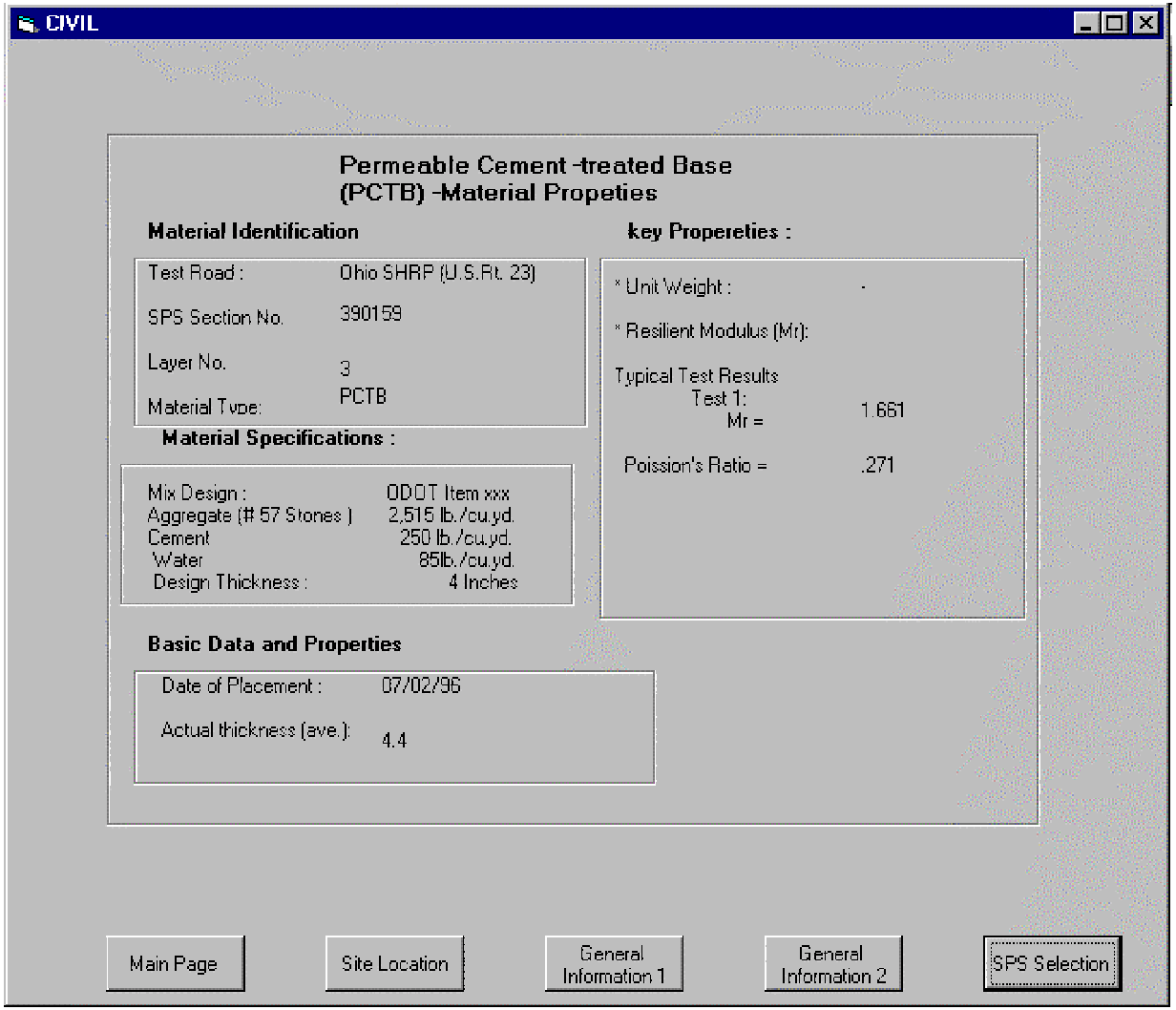


Figure 6.14 Material Properties for Lean Concrete Base (LCB)

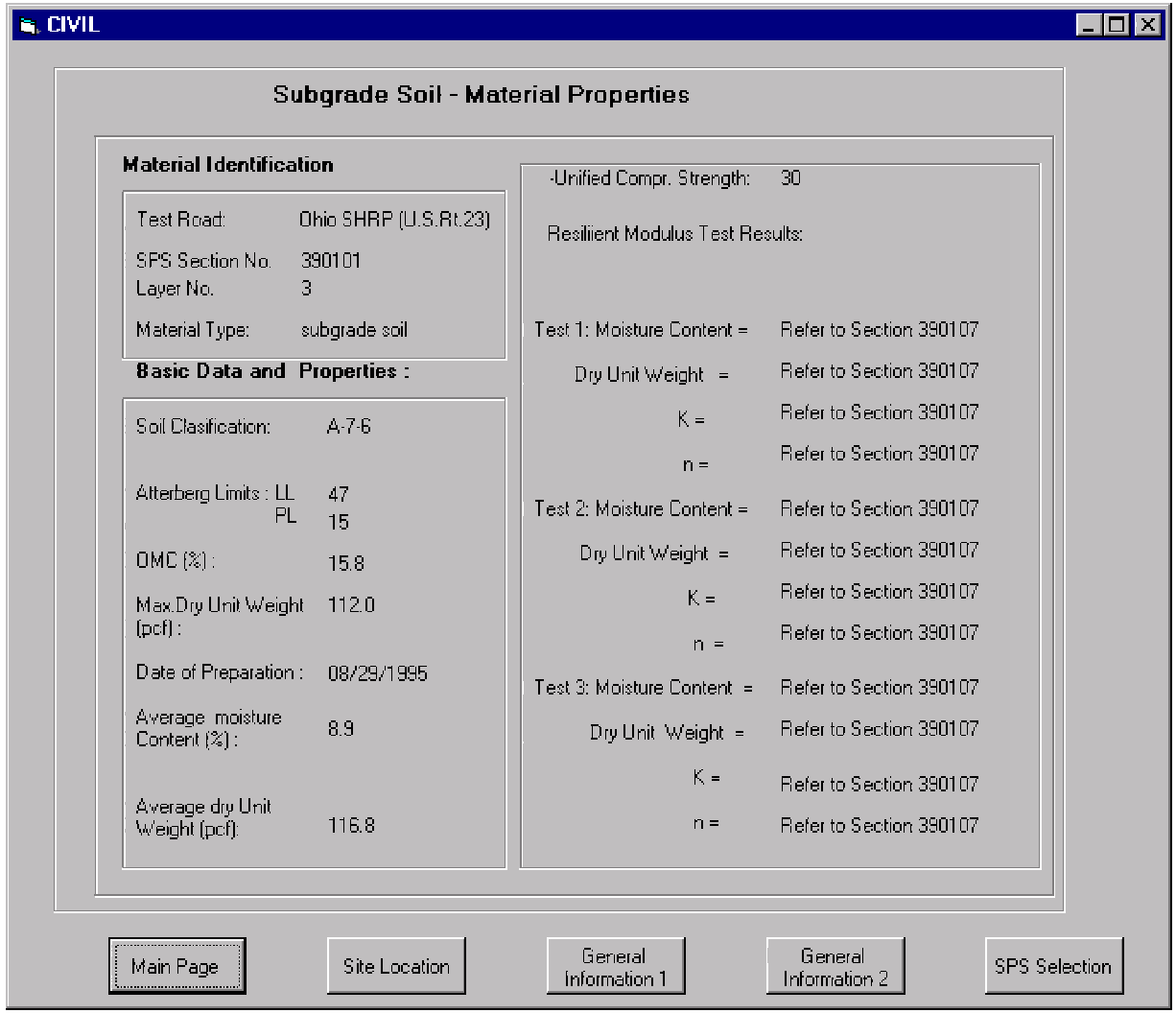


Figure 6.15 Material Properties for Permeable Cement-Treated Base (PCTB)

The following information is available on the subgrade soil under any of the selected sections (see Figure 6.16):

- AASHTO soil classification type
- Atterberg limits
- Standard Proctor test results (typical)
- In-situ moisture and density conditions (accepted values during construction)
- Unconfined compression strength
- Resilient modulus test results

The development of the computer database for the OH-SHRP project is still in an infancy stage. The CD-ROM database described above represents the initial phase of the development of a comprehensive database for the Ohio-SHRP project. Future developments under a separate contract will include the addition of the Falling Weight Deflectometer (FWD) data, load response data, environmental response data, statistical analysis, and report generation capability.

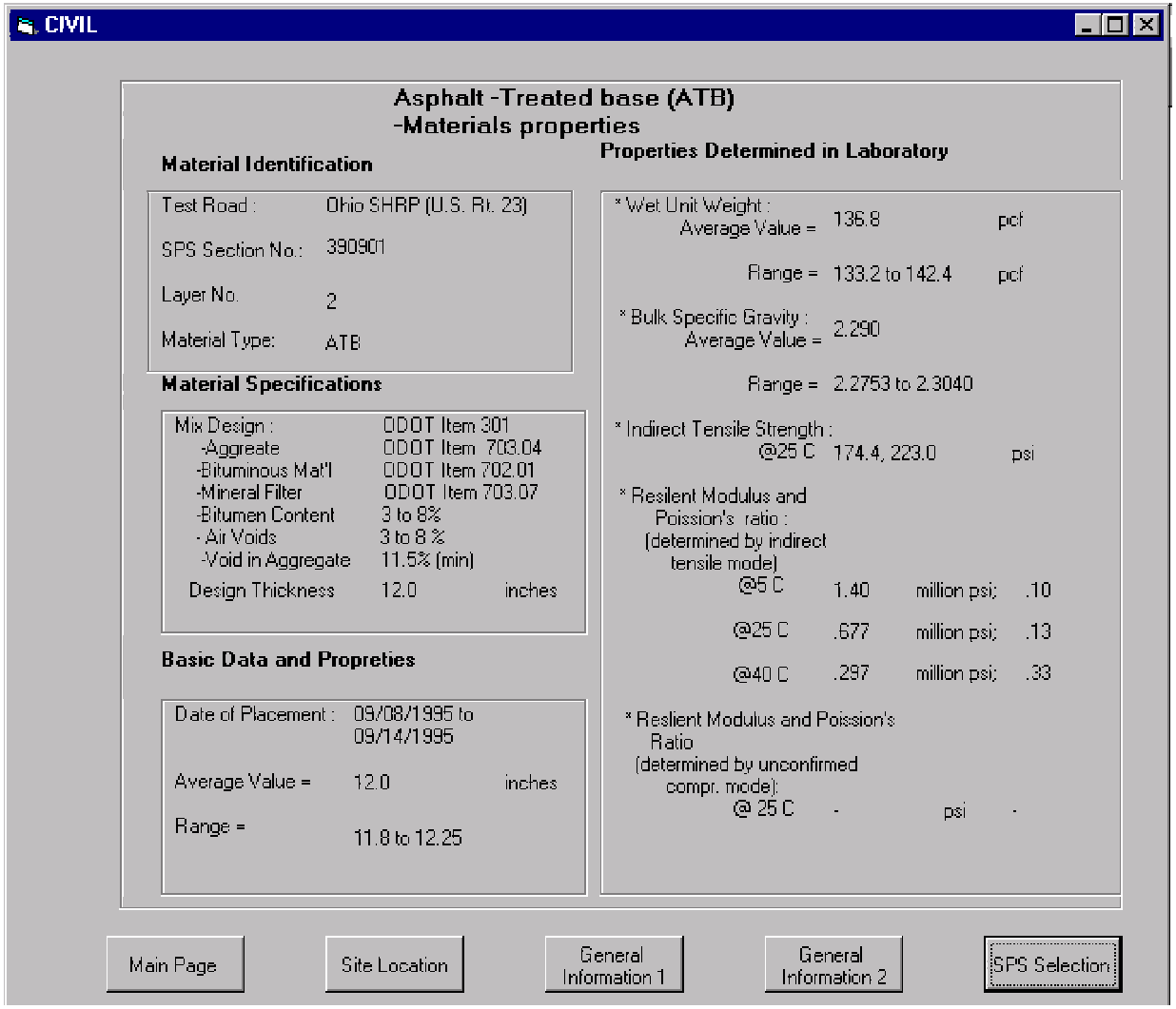


Figure 6.16 Material Properties for Subgrade Soils

CHAPTER 7:
SUMMARY, CONCLUSIONS, AND IMPLEMENTATIONS

7.1 General Summary

The current study was conducted to characterize mechanical properties of the pavement materials utilized in the construction of the Ohio-SHRP Test Road. The main thrust for this study came from the SHRP LTPP study. First, background information about the LTPP study was described. Then, detailed information was presented on various aspects of the Ohio-SHRP Test Road project. After describing each key laboratory test procedure, a series of laboratory experiments were performed on a wide range of pavement component materials that included subgrade soils, unbound granular base, stabilized bases, Portland cement concrete, and asphalt concrete. Laboratory test results were examined carefully to point out some important trends observed. In some cases, previously published empirical correlations, which would be applicable to the test results, were tested in light of the current laboratory test results. This was done to identify any reliable empirical correlations for predicting hard-to-measure pavement material properties from basic properties that are routinely measured. Also, in a few isolated cases, results from the relevant in-situ test method were compared to the laboratory test results. In parallel to the comprehensive laboratory testing, efforts were made to consolidate a large amount of material property data on the Ohio-SHRP Test Road. The result was a computer database packaged into a CD-ROM disk. The

following sections summarize key findings from the pavement material characterization study.

7.2 Conclusions

Based on the results obtained in this investigation, the following was concluded:

Subgrade Soils

- The subgrade at the Ohio-SHRP Test Road consisted of three fine-grained soil types (A-4, A-6, and A-7-6). In the laboratory tests, the resilient modulus of these soil samples ranged widely between 1.6 and 35.7 ksi (11.0 and 246.2 MPa) under varying deviatoric stress and moisture contents.
- Resilient moduli of the fine-grained soils were insensitive to changes in confining pressure. The resilient modulus of each of the three soil types had a tendency to decrease with the increasing deviatoric stress. These moduli were more sensitive to the changes in moisture contents for soils containing more clay.
- The relationship between resilient modulus and moisture content was nonlinear and resembled the right-hand side of a bell-shaped curve. This bell-shaped curve had a tendency to become flatter and lower its position on the coordinate system as the deviatoric stress increased.

- The range of resilient modulus values observed in the laboratory corresponded fairly well to the wide range of resilient modulus values resulting from the in-situ Falling Weight Deflectometer (FWD) test.

Unbound Granular Base

- The resilient moduli results obtained from lab tests on the unbound granular base material ranged between 11.94 to 14.11 ksi (82.3 to 97.3 MPa) under varying deviatoric stress and confining pressure. The resilient property of dense graded aggregate base (DGAB) was affected by the deviatoric stress. Resilient modulus of the granular base material had a tendency to increase with increasing deviatoric stress and confining pressure. Cumulative permanent strain experienced by the four test specimens did not vary significantly, remaining at 0.5 to 0.8%.
- The relationship between the resilient modulus and the bulk stress (θ) was relatively strong, with the correlation coefficient (R) of 0.82. The relationship between the resilient modulus and the deviatoric stress was weaker, represented by the correlation coefficient (R) of 0.5.
- Dry unit weight had a greater influence on the magnitude of the resilient modulus than moisture content, since the granular base material did not contain a large amount of fines and was free draining.

Asphalt-Treated Base (ATB)

- The resilient modulus of the asphalt-treated base ranged widely from 180.1 to 1,400 ksi (1.2 to 9.7 GPa) at the test temperature of 41 and 104 °F (5 and 40 °C).
- Temperature had a major effect on the magnitude of the resilient modulus, as the temperature increased the resilient modulus decreased. Poisson's ratio of the ATB was also found to be dependent on the temperature.
- A much stronger correlation was observed between the resilient modulus and the indirect tensile strength than the resilient modulus and bulk specific gravity.

Permeable Cement-Treated Base (PCTB)

- The average modulus of PCTB at room temperature using the unconfined dynamic loading test mode was about 1.14E+6 psi (7.9 GPa). The Poisson's ratio is 0.2 taken from the recommended ratio in the NCHRP 1-26 project statement.

Lean Concrete Base (LCB)

- The average modulus and Poisson's ratio of LCB at room temperature using the same procedure as that of PCTB, were 2.53E+6 psi (17.4 GPa) and 0.22, respectively.

Permeable Asphalt-Treated Base (PATB)

- Only bulk specific gravity was obtained from PATB specimens due to their generally poor quality. The bulk specific gravity of PATB was 2.324.

Portland Cement Concrete (PCC)

- The three types of mix design (low, regular, and high strength PCC) investigated in this research indicated that the mechanical properties (split tensile strength and static modulus) of the PCC materials improved with age. Static modulus of the cored PCC specimens varied between 1.1 and 5.5 million psi (7.6 and 37.9 GPa). Both the design mix and age had significant effects on the magnitude of the static modulus.
- Split tensile strength of the low strength and regular strength mix PCC were nearly identical at an early age (28 days).
- Laboratory testing of the PCC should not rely on the use of molded specimens. Strengths among the cored specimens were 5 to 20% higher than those exhibited by corresponding molded specimens.
- Static modulus of the PCC at 1 year can be estimated relatively accurately either from the compressive strength or the split tensile strength (using the ACI or NCHRP recommended formula). However, any of the empirical approach had a tendency to overpredict the modulus at 28 days.

Asphalt Concrete (AC)

- Resilient modulus of asphalt concrete core specimens ranged from 1.44E+5 to 9.93E+5 psi (1.0 GPa to 6.9 GPa) as the temperature varies between 41 and 104 °F (5 and 40 °C). Temperature had a significant effect on the magnitude of the resilient modulus.
- A much stronger correlation was observed between the resilient modulus and indirect tensile strength than between the resilient modulus and bulk specific gravity.
- Creep modulus of asphalt concrete ranged from 0.51 to 177 ksi (3.5 MPa to 1.22 GPa) for the standard thickness specimens, and from 6.1 to 88 ksi (42.1 to 607 MPa) for the non-standard specimens. Temperature plays a major role in the magnitude of creep modulus.
- Comparing the test results obtained for the surface and intermediate layers, there was no difference between them in terms of the bulk specific gravity. However, the resilient modulus of the intermediate layer specimens were generally higher than that exhibited by the surface layer specimens. This may be attributed to the fact that larger size aggregates were used in the intermediate layer.

7.3 Implementations

Based on the accomplishments made in the current study, the following implementation plans are suggested by the authors:

a) Laboratory test descriptions and results summarized in the current document can enhance highway engineers' understanding of the state of the laboratory testing of pavement materials, and general properties/behaviors of the highway pavement materials utilized in the Ohio-SHRP Test Road.

b) Engineers and researchers interested in the Ohio-SHRP Test Road project can access both the general project information and properties of construction materials instantly through the attached CD-ROM computer database.

c) Various material properties measured in the laboratory under the current study will be useful for preparing the data inputs during the implementation of the M-E procedure to the Ohio-SHRP Test Road. Preliminary review of several documents found many material properties measured in the current study to be designated as material property inputs into the M-E procedure.

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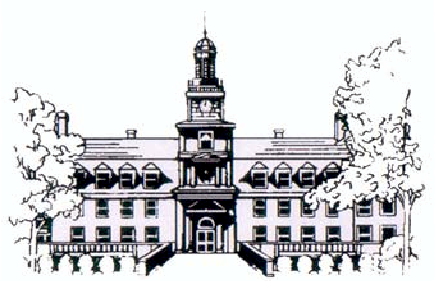

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