STRUCTURAL ANALYSIS OF NEW ENGLAND SUBBASE MATERIALS AND STRUCTURES

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16. Abstract

Properties of subbase materials used in New England have been compiled and a diversity of materials and practices has been catalogued. Most New England states have specifications for reclaimed materials, but Connecticut and Massachusetts are the only states routinely using reclaimed materials in the construction of the subbase layer. Connecticut allows stockpiles of reclaimed materials maintained at inspected private sites and blended there to meet specifications. Massachusetts specifies full depth in place milling of the entire pavement structure down to the subgrade, and the addition of virgin materials may be required to meet specifications.

The resilient modulus of subbase materials (E_{SB}) had been selected as the primary parameter for structural analysis of New England subbase materials, and the AASHTO T292-91 procedure had initially been used. However, the AASHTO TP46-94 procedure was finally selected to determine moduli of subbase materials with and without reclaimed asphalt pavement (RAP), since it uses larger specimens (150 mm dia. x 300 mm ht.) which represent the field samples better, a servo-hydraulic actuator, and more appropriate equipment configuration including LVDT location which leads less testing error. Virgin aggregate and reclaimed subbase materials provided by the participating state transportation agencies had been tested. Fundamental properties, coefficients of permeability, resilient moduli and layer coefficients for ten subbase materials have been determined in the present study. In addition, milled asphalt pavements have been blended for optimization of layer coefficient. All blends have coefficients better than the virgin aggregates. Based on results of the study, procedures for State agencies to develop optimum properties have been recommended.

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Preface

This is the final report of the research project, entitled "Structural Analysis of New England Subbase Materials and Structures." The goal was to develop optimum reclaimed material blends for subbase layers, and to recommend appropriate testing for State agencies in New England to develop optimum properties for specific sources and various combinations of blended material projects.

The research project was conducted by the Department of Civil and Environmental Engineering at the University of Rhode Island (URI) under the contract No. NETC 94-1 to the New England Transportation Consortium (NETC). The work presented herein was accomplished by a team including Dr. K. Wayne Lee, Mr. Milton Huston, Mr. Jeffrey Davis and Mr. Sekhar Vajjhalla. We would like to acknowledge the assistance provided by the faculty, staff and students in the Department of Civil and Environmental Engineering at URI.

The development and completion of this project could not have occurred without the cooperation and encouragement of many people in each of the six participating states and FHWA, particularly members of the technical committee: Mr. Michael Sherrill (Chair), Mr. Ted Batco, Mr. Philip Dunn, Mr. Clement Fung, Mr. Michael Hedges, Mr. Max Grogg, Mr. Paul Matthews, and Mr. Leo Stevens, Jr. (Former Chair). Members of Advisory Committee, Dr. Charles E. Dougan and Mr. Gerald M. McCarthy also provided essential continuing support and assistance throughout the program. Thus, this was truly a joint effort upon the part of many individuals and agencies, and members of URI research team appreciate those efforts.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1 INTRODUCTION

Designs for pavement structures are mainly based upon the strength characteristics of the materials used within the structure. However, it has been observed that materials used in the subbase layer have received the least amount of attention to define strength. Currently, a number of methods exist to characterize the support strength of granular subbase materials. The 1993 AASHTO Guide for Design of Pavement Structures (AASHTO Guide) recommends assigning appropriate layer coefficients based on the resilient modulus value expected (AASHTO 1993). Resilient modulus is defined as deviator stress over recoverable strain measured during specific incremental loading sequences which attempt to recreate vehicular loading conditions on pavement structures. Resilient modulus testing procedures have undergone rapid changes over the last few years. Furthermore, there is no comparative analysis between states for procedures and materials currently being used for the subbase layer. Therefore, the New England Region requires an upgrading and standardization of procedures for the determination of resilient modulus values of subbase materials.

The transportation group at the University of Rhode Island (URI) has done significant research in the area of materials used for pavement structures for the Rhode Island Department of Transportation (RIDOT) (Kovacs et al. 1991) (Lee et al.1994a) (Lee et al. 1994b). Therefore, the URI research team is in a good position to expand its initial works in other New England States. In order for the AASHTO Guide recommendations to be implemented regionally, subbase material properties should be of great benefit for the improvement of road structural designs, since the region shares a similar climatic and geological characteristics.

Subbase structural designs today are complicated by State agencies' interest in utilizing recycled materials blended with existing aggregates. Some States have performed analysis of these materials; however, no history of these analyses are available to aid in future analysis of the various recycled materials in structural subbases. On a regional basis, aggregate types common to more than one State could provide a basis for the development of optimum structural and drainage characteristics for these aggregate types blended with several individual or combined recycled materials.

Therefore, the purposes of this research are: (1) to compile a database on aggregate properties by aggregate types common to New England; (2) to characterize natural aggregates and recycled material/aggregates blends provided by participating State agencies; (3) to develop optimum performance characteristics for aggregate type and recycled blends which are provided by State agencies; and (4) to recommend appropriate testing for State agencies to develop optimum properties for specific sources of blended materials.

Chapter 2 discusses the current status of knowledge. Chapter 3 provides a compilation of aggregate material properties and specifications from the New England States. Chapter 4 presents an evaluation of granular subbase materials and bulk stress analysis for estimating a layer coefficient. Chapter 5 evaluates subbase materials with reclaimed materials. Chapter 6 provides the conclusions and recommendations of this study.

CHAPTER 2 CURRENT STATUS OF KNOWLEDGE

2.1 Fundamental and/or Traditional Tests for Subbase Materials

Gradation or grain size distribution curves are used to help describe and classify a subbase material. A material composed of one size is called uniform and one with a wide range of grain sizes is well graded. The amount and type of fine grains (fines) in a subbase material are very important in assessing the properties of this granular material. Volumetric strain is directly affected by the presence of finer particles in the mixes. The effect of aggregate grading was studied by Shaw (1980). A comparison was made between 40 mm maximum sized broadly graded crushed rock roadbase material and a 3 mm single sized stone from the same source. The broadly graded material was found to be much stiffer than the single sized stone, partly due to the large difference in maximum particle size. It has been observed that the resistance to permanent deformation generally increases for well-graded materials as compared to that of open-graded materials.

The Atterberg limit tests are necessary to classify the subbase materials based upon the fine portion of the aggregates in a sieve analysis (ASTM D4318). Atterberg limits tests are conducted on materials passing the number 40 sieve, grain sizes which are not large enough to characterize by gradation. The plastic limit of a subbase material is the lowest water content at which it acts as a plastic material. This is determined when it breaks apart when rolled 3.175 mm(1/8 in.) thick. Materials that cannot be rolled to a thread at any water content are non-plastic. The purpose of the plasticity requirement for subbase materials is to limit variation of shear strength due to water content fluctuation and also because aggregate including more plastic materials are generally weaker. However, achieving the plasticity requirement gives little or no guarantee of good material performance under trafficking (Cheung 1994). The liquid limit is defined as the water content at and above which it behaves as a liquid. This can be determined to the material in a brass cup, cut with a standard groove, to a closure of 12.7 mm (1/2 in.), when dropped 25 times.

Compaction requirements are measured in terms of dry density of subbase materials. The maximum dry density and optimum moisture content (OMC) for compactive effort are basic properties to construct granular subbase layers. These properties are determined by compaction curve, i.e., a moisture density curve or a proctor curve. The 1993 AASHTO guide for Design of Pavement Structures (AASHTO Guide) recommends that untreated aggregate subbase should be compacted to 95% of maximum laboratory density, or higher, based on AASHTO Designation T180, Method D, or the equivalent ("Guide" 1993).

Subbase material can be classified by the AASHTO soil classification system (AASHTO M145). This classification system identifies granular materials such as sand, gravel and stone fragments based upon gradation and Atterberg limits. Most state departments of transportation have subbase specifications with granular material proportions ("Specifications" 1995) ("Specifications" 1995a) ("Specifications" 1995b) ("Standard" 1990) ("Specifications" 1998) ("Subbase" 1990).

2.2 Strength Tests (for Structural Analysis)

The California Bearing Ratio (CBR) test was developed by the California Highway Department to evaluate the strength and swelling potential of soil (Franco and Lee 1987).

Three specimens are compacted using 10, 30, and 65 blows at OMC. They are soaked and swelling is measured. Resistance to penetration by a standard 3-inch2 piston at a loading rate of 0.05 in./min. is recorded (AASHTO T193-93). The resistance at intervals is recorded, plotted and corrected. The resistance at 0.1 inch is the normal value used for CBR. Dry density vs CBR is plotted and CBR at 95% of max. dry density is used for design. CBR is an empirical strength test, and provides no information on material response under rapid traffic loadings.

Elastic Modulus is the key parameter used for performance prediction. The AASHTO Guide Part II provides procedures for assigning appropriate layer coefficient based on resilient modulus for subbase materials (E_{sb}). E_{SB} is a repeated loading test response. This test is conducted in a triaxial device using a 100 mm x 200 mm (4 in. x 8 in.) specimen and pneumatic actuator to determine E_{SB} (AASHTO designations T292-91 and T294-92 [SHRP P-46]).

Since grain sizes of 1 1/2 in. or larger are common in subbase, AASHTO Provisional Standard TP46 has increased the specimen size to give a more realistic test sample. In addition it has been determined that with small repeated loading a hydraulic device is much more accurate. Normally, a pneumatic system provides the cheaper solution for applying axial load, but the ability to create controlled load pulses degenerates rapidly when the magnitude or the frequency of repeated loading is high (Cheung 1994). Therefore, it has been suggested that hydraulic device should be used on a 150 mm x 300 mm (6 in. x 12 in.) specimen.

Specimens are subjected to a series of deviator stresses at varying confining pressures. This repeated loading simulates the repeated axial stress incurred from vehicular traffic. The axial deviator stress is defined as the relation between the applied axial load (P) over the cross section area of the sample (A):

$$\sigma_d = P/A \quad \text{(2-1)}$$

The axial strain ε_a is defined as the relation between axial deformation (Δ) over the gage length (L_g):

$$\varepsilon_a = \Delta/L_g \quad \text{(2-2)}$$

Therefore, the E_{sb}, which is an estimate of the dynamic Young's Modulus, can be determined as the ratio of repeated axial deviator stress to the recoverable or resilient axial strain ε_r, i.e.:

$$E_{sb} = \sigma_d / \varepsilon_r \quad \text{(2-3)}$$

Resilient Modulus tests performed on granular materials and subgrade soils have demonstrated the highly significant effect of confining pressure on moduli values (Lee et al. 1994)

The AASHTO Guide recommends the following equation to determine the resilient moduli of course-grained cohesionless soils and granular subbase materials:

$$M_r = K_1\,(\theta)^{k2} \quad \text{(2-4)}$$

θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$; K_1 and K_2 = experimental constants determined from the regression analysis with a set of test results.

However, the AASHTO TP46 protocol recommends the following equation to determine the resilient modulus of coarse-grained cohesionless soils:

$$M_R = K_1\,(S_C)^{K2}(S_2)^{K5} \quad \text{(2-5)}$$

S_C = cyclic stress; K_1, K_2 and K_5 = experimental constants determined from the regression analysis on a set of test results.

2.3 Drainage Considerations

Subsurface drainage cannot be eliminated from consideration in the design of pavement structures. It is recommended that the normal pavement design practice be followed to develop the general cross-sections, then these can be analyzed for subsurface drainage. This approach involves less confusion than trying to incorporate detailed drainage analysis in the initial design. Methods to control groundwater and infiltration must be considered in the structural design of pavement.

If the pavement structure and subgrade can become saturated, by groundwater, and/or infiltration, its ability to support the dynamic loading imposed by traffic can be greatly impaired. In asphalt pavement systems, this impairment is primarily the result of the temporary development of very high pore water pressures and the consequent loss of strength in unbound base, subbase and subgrade under dynamic loading. In some instances, the pressures induced in the free water may be sufficient to cause it to be ejected through cracks in the pavement surface along with suspended fines.

Another adverse effect that uncontrolled moisture can have on pavement systems is the frost action. Frost action requires the presence of a readily available supply of subsurface moisture, frost susceptible soils, and a sustained period of subfreezing temperatures. If all these requisites are satisfied, then moisture will migrate through the capillary fringe toward the freezing front to feed the growth of ice lenses. During the active freezing period, the growth of ice lenses can result in substantial heave of the overlying pavement structure. This can cause significant damage to a pavement, particularly if the differential frost heaving is experienced. However, the most potentially destructive effect of frost action is associated with the loss of support during spring thaw. The thawing of ice lenses leaves the subgrade soil saturated, or possibly supersaturated, resulting in a substantial reduction in its strength. Since the thawing generally takes place from the top down, the only way the moisture can drain from the subgrade soil is by flowing into any available voids that may exist in the pavement structure. If the pavement structure, particularly base and/or subbase is not adequately drained, it may become saturated with the water being squeezed from the subgrade, and the destructive mechanisms previously discussed may become operative. The resulting pavement deterioration is generally referred to as spring break up.

The frequent or sustained presence of excess moisture in pavement components and intermittent exposure to cycles of freezing and thawing can result in the loss of structural integrity. The main source of water that infiltrates into the pavement structure is also precipitation. It is very likely that the lack of adequate subsurface drainage also leads to shortened life and large annual expenditures e.g., increased maintenance and rehabilitation costs.

Capillary moisture is held in the pores above the level of saturation (water table), free water surface, or phreatic line under the action of surface tension forces. The height of the capillary fringe and the shape of the moisture-tension curve is a function of the pore size distribution in soil, which is related to its grain size distribution and density. The degree of saturation resulting from capillarity is also a function of the history of the position of the water table. Thus the only means available for control of capillary moisture are through lowering the water table with appropriate sub-drainage or prcviding for a positive barrier against capillary rise.

Generally, seepage is defined as the movement, or flow, of a fluid through a permeable or porous medium. In particular, the fluid which we are concerned is water, and the permeable porous media are soils, rock and the structural elements of pavements. The porosity is defined as the ratio of the volume of pore spaces to the total volume of the material. The extent to which porous media will permit fluid flow, i.e., its permeability, is dependent upon the extent to which the pore spaces are interconnected and the size and shape of the interconnections (FHWA 1980).

The AASHTO Guide recommends determination of a structural number for each layer, based upon layer thickness and a layer coefficient using the following equation:

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3 \tag{2-6}$$

Where,

SN = Structural number,

a_i = layer coefficient,

D_i = layer thickness, and

m_i = drainage coefficient

The drainage coefficient m_i is included for the unbound layers of the pavement structure. The AASHTO Guide provides recommended values for m_i as a function of the quality of drainage and the percent of the time during the year a pavement structure would normally be exposed to moisture levels approaching saturation. These values apply to untreated subbase layers.

It is recommended in the AASHTO Guide that in areas of poor drainage a drainage layer be included. It was stated earlier that open graded materials would have better permeability. The inclusion of a drainage layer below the subbase is recommended for this purpose. A measure of the subbase permeability should be performed for cases where the subgrade drainage is considered good and the additional drainage layer is not considered necessary.

2.4 Current New England Practices on Subbase Materials and Structures

The primary function of the subbase is structural as stated in the AASHTO Guide. The secondary function of providing a working platform for construction equipment is also structural. The third function, when a dense graded material is chosen, would be to prevent intrusion of fine-grained roadbed soils into base courses. Lastly, when a drainage layer is not included in the pavement design, the subbase would be used for the drainage function.

Current New England practices on subbase materials and structures have been gathered from participating states, in the present study. Connecticut, New Hampshire, Vermont and Maine assume a drainage coefficient of 1, which equates to fair drainage for 5%-25% saturation time in the AASHTO Guide. Vermont constructs pavements with a drainage course below the subbase. New Hampshire uses the subbase primarily for drainage (Lee et al. 1999). Rhode Island relies on the subbase layer to perform the additional function of preventing the accumulation of free water.

Tire chips have been used as subgrade insulation for a rural road in Richmond, Maine. They were placed as an insulating layer to limit frost penetration beneath a gravel subbase. Frost penetration was reduced 15% - 37% depending upon the structural and drainage layer design depth. Clearly, the tire chips significantly reduced the depth of frost penetration and the amount of frost heave (Humphrey and Eaton 1995). In Georga, Vermont shredded tires were designed to serve as a subbase layer on a rural highway test section. The shredded tires were designed to serve as both a drainage layer and a barrier between wet silty sand subgrade and the gravel base, in an area with a high water table. The tire chip layer enhanced the poor-quality gravel by cutting off the capillary rise of subsurface water and by reducing the moisture content of the gravel through good drainage. The muddy road conditions prevalent in past spring seasons did not recur following the placement of the tire chip layer. The use of tire chips at a cost of $\$1.30/m^3$ ($\$1.00/yd^3$) reduced the need for additional gravel that would cost $\$5.00/m^3$ ($\$3.85/yd^3$). An asphalt emulsion chip seal placed on the initial test section revealed only minor distress through its first year of service (Frascoia and Cauly 1995).

It has been found that each New England State has its own design and typical cross-section which is shown in Chapter 3. Each state has its own criteria for subbase performance characteristics. This project has attempted to develop a uniform design feature and/or optimum design features from characteristics provided by each state. However, it is our intention to develop uniform test criteria from which designs can be performed.

CHAPTER 3 PROPERTIES OF SUBBASE MATERIALS USED IN NEW ENGLAND

Natural aggregate properties for the different aggregate types available for structural subbase materials in New England were collected with the assistance of the New England Transportation Consortium (NETC) technical committee: Connecticut Department of Transportation (ConnDOT), Rhode Island Department of Transportation (RIDOT), Vermont Agency of Transportation (VAT), Maine Department of Transportation (M DOT), Massachusetts Highway Department (MAHWD), New Hampshire Department of Transportation (NHDOT). Some data have been abstracted from existing state specifications and design guides. Data collected included gradation and corresponding structural properties, wear, permeability and frost susceptibility.

3.1 Virgin Aggregate Material Properties

The types and properties of typical subbase materials (aggregates) used in the New England region are provided as specified by each state. Material type, particle size, plasticity, resistance to abrasion and soundness are the properties collected. The compiled information is summarized by state.

3.1.1 State of Connecticut

(1)Type: Bank Run or Crushed Gravel

(2)Acceptable Gradation by Percent Passing by Weight:

Mesh Size	% Passing
5 in.	100
3 1/2 in.	90-100
1 1/2 in.	55-95
1/4 in.	25-60
No.10	15-45
No.40	5-25
No.100	0-10
No.200	0-5

(3) Plasticity:

-#100 <4%	No Test Required.
%4< -#100 <8%	Shall not have sufficient plasticity to perform AASHTO T90.
-#100 >8%	Be washed. Determine the additional material by AASHTO T146. The combined material shall not have sufficient plasticity to perform AASHTO T90

(4) Resistance to Abrasion: Less than 50% (AASHTO T96)

(5) Soundness: Course aggregate less than 15% after 5 cycles
Magnesium Sulfate Soundness (AASHTO T104)

3.1.2 State of Maine

(1) Type: Sand or gravel of hard durable particles

(2) Acceptable Gradation by Percent Passing by Weight:

Mesh Size	Percent Passing			
	Type D	Type E	Type F	Type G
¼ in.	25 - 70	25 - 100	60 - 100	-
No. 40	0 - 30	0 - 50	0 - 50	0 - 70
No. 200	0 - 7	0 - 7	0 - 7	0 - 10

3.1.3 State of Massachusetts

(1) Type: Processed Gravel, Dense Graded Crushed Stone

(2) Acceptable Gradation by Percent Passing by Weight:

Mesh Size	Process Gravel (% Passing)	Dense Graded Crushed Stone (% Passing)
75mm	100	100
50mm	100	100
37.5mm	70-100	70-100
19.0mm	50-85	50-85
4.75mm	30-60	30-55
300μm	NA	8-24
75 μm	0-10	3-10

Note: NA stands for not applicable

Resistance to Abrasion: Less than 45% (AASHTO T96)

3.1.4 State of New Hampshire

(1) Type: (typically sandy gravel)

(2) Acceptable Gradation by Percent Passing by Weight

Mesh Size	Sand (% Passing)	Gravel (% Passing)
6 in.	100	100
No.4	70-100	25-70
No.200	0-12	0-12

3.1.5 State of Rhode Island and Providence Plantations

(1) Type: Gravel Borrow

(2) Acceptable Gradation by Percent Passing by Weight:

Mesh Size	% Passing
3"	60-100
1/2"	50-85
3/8"	45-80
#4	40-75
#40	0-45
#200	0-10

3.1.6 State of Vermont

(1)Type: Gravel and Dense Graded Crushed Stone

(2)Acceptable Gradation by Percent Passing by Weight:

Mesh Size	Gravel	Crushed Gravel fine	coarse	Dense Graded Crushed Stone
100mm			95-10	
90mm				100
75mm				90-100
50mm		100		75-100
37.5mm		90-100		
25mm				50-80
12.5mm				30-60
4.75mm	20-60	30-60	25-50	
150μm	0-12	0-12		
75μm	0-6	0-6		0-6

(3)Resistance to Abrasion:

Crushed Stone - Less than 40% (AASHTO T96)
Igneous Rock - Less than 50% (AASHTO T96)

Most highway specifications for subbase materials specify a grain size distribution that will provide a dense, strong mixture. Strength, or resistance to shear failure, in road bases, subbases, and other aggregates that carry loads is increased greatly if the mixture is dense graded. Excessive amounts of fines may result in weak mixtures as the large grains are not in contact with each other (Atkins 1983). The increased strength of a dense graded material is dependent on grain to grain contact.

3.2 Properties of Reclaimed Asphalt Pavement (RAP)

Reclaiming methods for subbase materials in New England vary considerably by state. In order to get a perspective of the properties of reclaimed asphalt pavement

(RAP), a brief description of reclaiming methods has been provided for each New England State. Material type and properties are provided in the same format as the previous section.

3.2.1 State of Connecticut

(1) Type: Reclaimed subbase may contain 2% asphalt cement by weight.

(2) Method:

Stockpiles of reclaimed construction material (cement concrete, asphalt pavement) are maintained at inspected private sites and blended there to meet specifications. These materials are tested for environmental acceptability.

(3)Acceptable Gradation by Percent Passing by Weight:

Mesh Size	% Passing
5 in.	100
3 1/2 in.	90-100
1 1/2 in.	55-95
1/4 in.	25-60
No.10	15-45
No.40	5-25
No.100	0-10
No.200	0-5

(4)Plasticity:

-#100 ≤4%	No Test Required.
%4< -#100 ≤8%	Shall not have sufficient plasticity to perform AASHTO T90.
-#100 >8%	Be washed. Determine the additional material by AASHTO T146. The combined material shall not have sufficient plasticity to perform AASHTO T90

(5) Resistance to Abrasion: Less than 50% (AASHTO T96)

(6) Soundness: Course aggregate less than 15% after 5 cycles.

Magnesium Sulfate Soundness (AASHTO T104)

3.2.2 State of Maine

(1) Type: No reclaiming specification yet

(2) Method: RAP is used on top of Northern Maine friable gravel to provide a structural cold reclaimed base layer to prevent degradation of the subbase during construction. RAP is not blended with the virgin granular material.

(3) Specification: experimental projects are underway using rubber tire chips within deep embankments.

3.2.3 State of Massachusetts

(1) Type: Reclaimed Asphalt (cold in place)

(2) Method: Full depth in place milling of the entire pavement structure down to the subgrade. The site Engineer may require the addition of virgin materials to meet specifications.

(3) Specification: Crushed asphalt pavement, crushed cement concrete, and gravel borrow meeting gravel borrow specifications. Processed glass aggregate is acceptable but not currently used.

(4) Acceptable Gradation by Percent Passing by Weight:

Mesh Size	% Passing
75mm	100
37.5mm	70 – 100
19mm	50 – 85
4.75mm	30 – 60
300mm	8 – 24
75mm	0 – 10

(5)Resistance to Abrasion: Less than 50% (AASHTO T96)

3.2.4 State of New Hampshire

(1) Type: no current specification

(2) Method: Reclaimed stabilized base course with a minimum bitumen content of 3% is being used.

3.2.5 State of Rhode Island and Providence Plantations

(1) Type: Reclaimed Borrow

(2) Method: Gravel borrow may be reclaimed within the project limits or as approved by the Engineer from other sites. Sometimes milled asphalt (RAP) is blended with virgin aggregate on the site.

(3) Specification: Subbase borrow in all cases may contain recycled materials. A reclaimed processed material specification allows RAP, concrete pavement and other materials.

(4)Acceptable Gradation by Percent Passing by Weight

Mesh Size	% Passing
3"	100
1 1/2"	70-100
3/4"	50-85
#4	30-55
#50	8-25
#200	2-10

3.2.6 State of Vermont

(1)type: PGA glass allowed

(2)method: 10% by mass allowed to meet virgin material specification

3.3 Typical Pavement Cross Sections

Typical pavement cross sections were prepared from questionnaires for the Research Project entitled "The Development of Design Parameters for Pavement Structures in Rhode Island" (Lee et al. 1996). These typical pavement cross sections were presented to NETC Technical Committee members for review at a special session held at the University of Rhode Island on April 5, 1996. In order to determine a representative subbase resilient modulus, additional input values were needed for the ELSYM5 computer program. On December 4, 1996 initial values on the revised cross sections were sent to each Technical Committee member for additional review. Wheel load and pressure on the pavement were abstracted from a previous research project "Development of a Uniform Truck Management System" (Lee, et al. 1990). In any case, the minimum federal single statuary axle limit of 20,000 lb. (5,000 lb. per wheel) was applied at 100 psi tire pressure (STAA 1982). Figures 3.1 through 3.6 present the typical pavement cross sections with elastic modulus, Poissons ratio, tire loading and in some cases dry density to be used for further analysis.

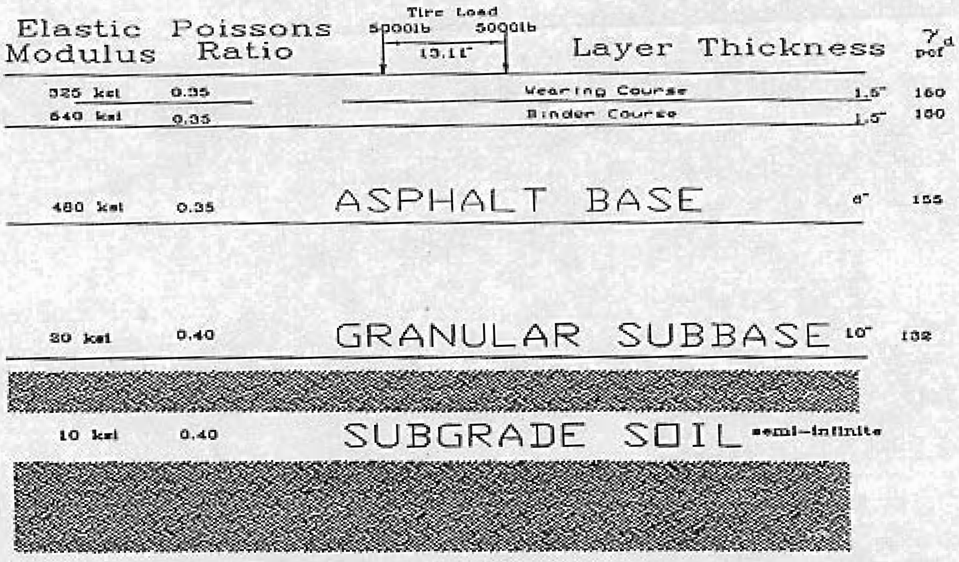


Figure 3.1 Typical Cross Section of Pavement Structure for the State of Connecticuit

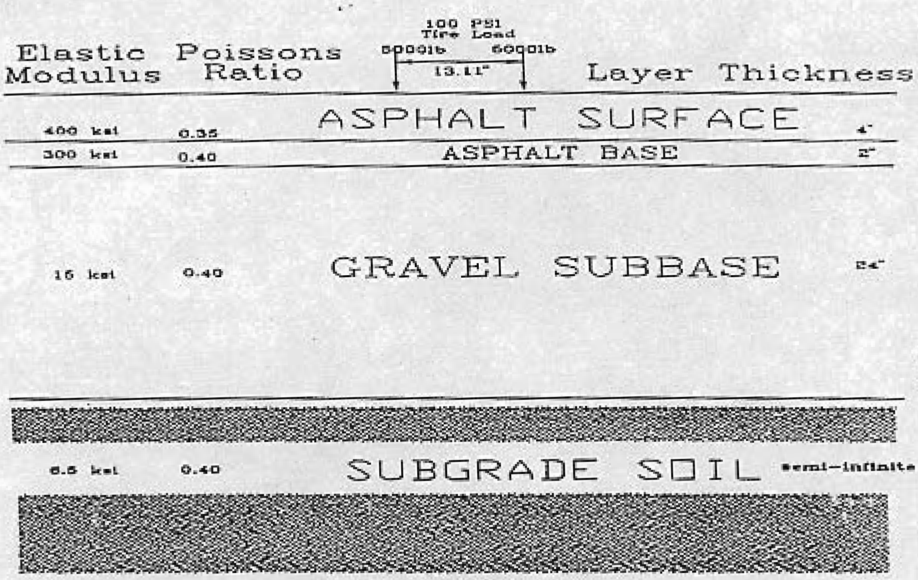


Figure 3.2 Typical Cross Section of Pavement Structure for the State of Maine

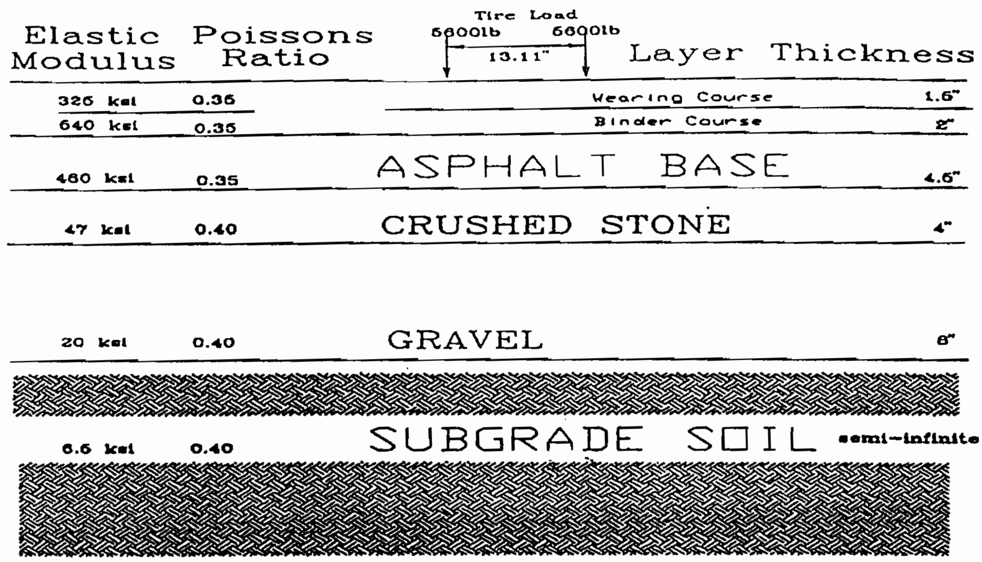


Figure 3.3 Typical Cross Section of Pavement Structure for the State of Massachusetts

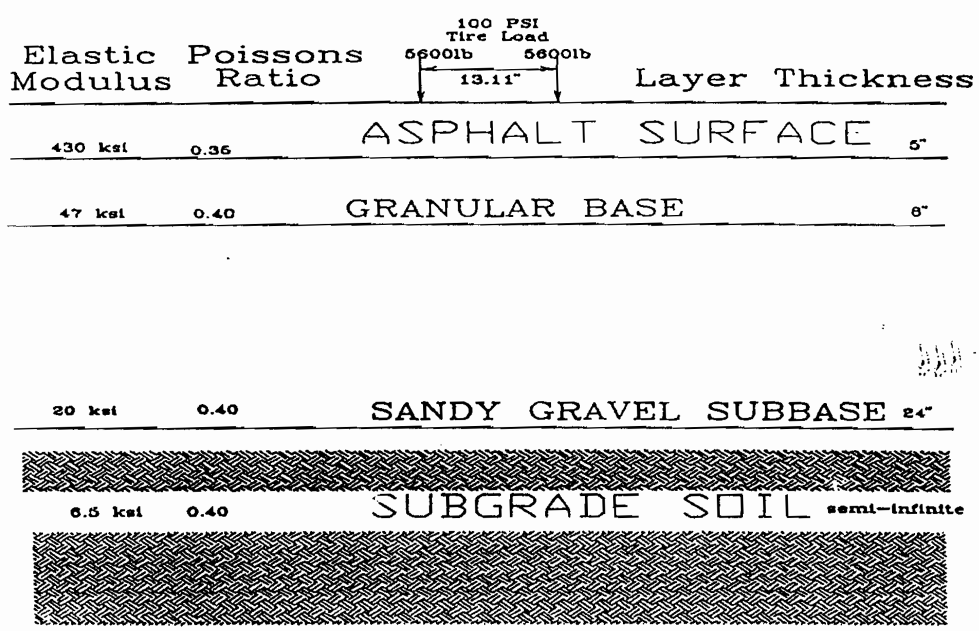


Figure 3.4 Typical Cross Section of Pavement Structure for the State of New Hampshire

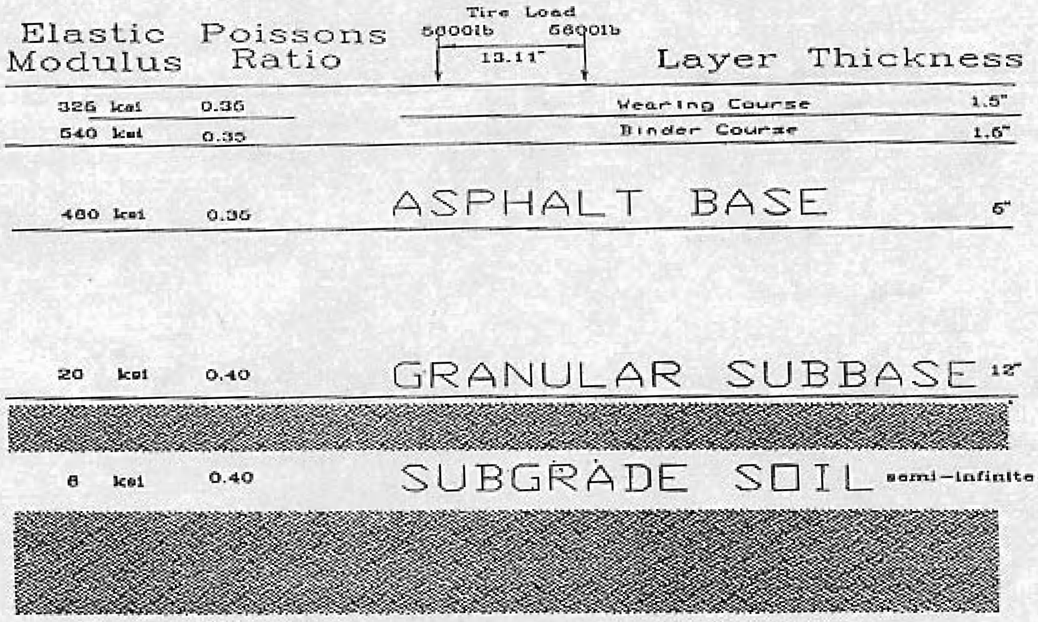


Figure 3.5 Typical Cross Section of Pavement Structure for the State of Rhode Island

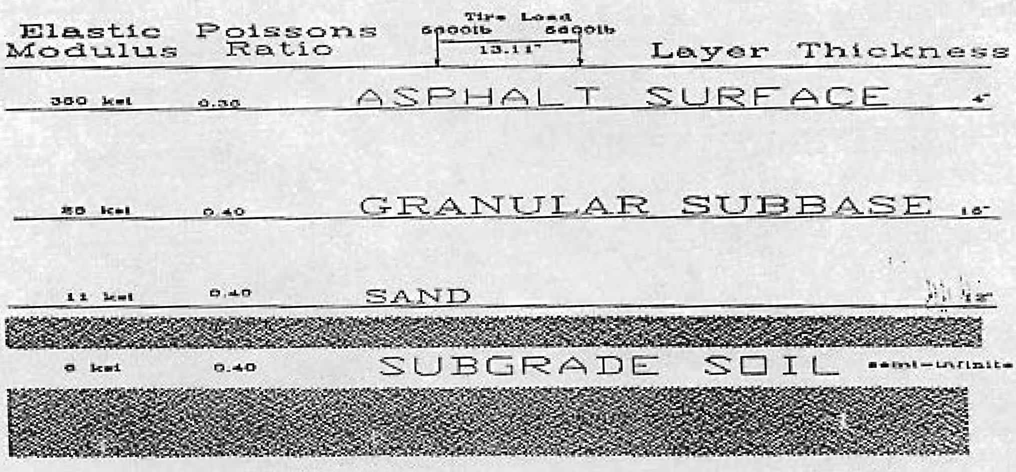


Figure 3.6 Typical Cross Section of Pavement Structure for the State of Vermont

CHAPTER 4 STRUCTURAL ANALYSIS OF GRANULAR SUBBASE MATERIALS

The subbase layer is the portion of the flexible pavement structure between the roadbed soil and the base course. It usually consists of a compacted layer of granular material, or of granular material with reclaimed asphalt pavement (RAP). This chapter deals with natural granular subbase and chapter 5 discusses subbase with RAP.

Granular subbase materials were collected from six New England States for structural analysis. The AASHTO Interim Guide typically used the California Bearing Ratio (CBR) to determine the Soil Support value (S) for the structural analysis of subbase materials. However, it is too empirical and most agencies are not using CBR any more. Since layer coefficients have been assumed to be indicators of the strength of pavement materials in the 1993 AASHTO Guide, an attempt was made to estimate coefficients for structural subbase materials. In the present study, the resilient modulus (E_{SB}) was utilized to estimate the layer coefficients.

This chapter discusses the experimental program fundamental test results and analysis, resilient modulus test results, and layer coefficient for natural subbase materials. It also includes permeability test results and determination of a drainage coefficient (m_i).

4.1 Experimental Program

Granular subbase materials collected from New England States were classified in accordance with AASHTO Designation M145-87 "The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes". ("Standard" 1996). Three fundamental tests were performed for this purpose: sieve analysis (AASHTO T27-88), liquid limit (AASHTO T89-93) and plastic limit (AASHTO T90-92). To determine optimum moisture content (OMC) and maximum dry density (γ_d) of each subbase material, a series of tests was conducted in accordance with AASHTO T180-90 (using a 10 lb rammer and an 18 in. drop).

Although there are many types of material properties for assessing the structural properties of pavement structures, resilient modulus has been adopted as a basis for design in the AASHTO Guide. However, the proposed testing procedure, i.e. AASHTO T274-82, needed to be modified, because most specimens were failed before data collection phase. Therefore, improved testing procedures were developed, e.g., URI procedure (Kovacs et al. 1991), AASHTO T292-91, and AASHTO T294-92. The interim study was carried out in accordance with the AASHTO T292-91 procedure. This test was performed on the 100 mm x 200 mm (4 in. x 8 in.) cylindrical specimen with a pneumatic actuator.

The URI research team has been performing the resilient modulus test with equipment purchased from the H&V Materials Research and Development, Inc., Corvallis, Oregon, July 1989 ("Resilient" 1989). The URI team modified this equipment to perform more user friendly tests (Jin et al. 1991). The schematic of this system is shown in Figure 4.1.

In September 1994 a Provisional Standard (AASHTO TP46) was published in conjunction with Strategic Highway Research Program (SHRP) researchers ("Standard" 1997). Since a grain size of 38.1mm (1 1/2 in.) is common in subbase materials, AASHTO has increased the specimen size to be more realistic. In addition, it has been observed that a hydraulic device is much more accurate than a pneumatic system with small repeated loading. Although a pneumatic system normally provides the economical solution for applying axial load, the ability to create controlled load pulses degenerates rapidly when the magnitude or the frequency of repeated loading is high. It is a general agreement among transportation engineers that AASHTO TP46 with a 150 mm x 300 mm (6 in. x 12 in.) specimen should be used whenever the proper equipment is available.

The URI research lab is now equipped with an Instron servo-hydraulic testing machine (Model No. D11961), LVDT's and a load cell from Instron Corporation. Additionally, a 152 mm x 381 mm (6 in. x 15 in.) split mold, triaxial chamber, and the compactor purchased from Law Engineering Inc., Atlanta, Georgia in 1997. A series of testing were performed in accordance with Provisional Standard TP46, on all of the materials tested with the AASHTO T292-91 procedure. Both test results provided the final structural characteristics of subbase materials and pavement design parameters for the New England region.

In addition, the permeability test was performed in accordance with AASHTO T215-70 to determine the coefficient of permeability for each sample. A compaction permeameter with a 150 mm (6 in.) cylinder diameter was purchased for this purpose. Table 4-1 summarizes the list of tests performed in this study.

4.2 Fundamental Properties of New England Subbase Materials

4.2.1 Gradation

The project began in the fall of 1995. Most of the subbase materials were collected during the winter of 1995 due to the regional nature of this project and the coordination efforts required. These materials were provided from state inspected stock piles rather than ongoing construction projects. The materials were selected by the individual state as representative of subbase materials used for primary state routes (not interstate highways).

All samples provided by the NETC States were tested in accordance with AASHTO T27-88 "Sieve Analysis of Fine and Coarse Aggregates". The exception to this was Rhode Island Route 2 subbase material, which was taken directly from the site for prior projects (Lee et al. 1994).

Test results are plotted in Figures 4.2 through 4.9 for different subbase materials from six New England states. The majority of the materials tested fall within the band set by State Specification. Maine Sabattus gravel had more 12.7mm (0.5 in.) and 4.75mm (No.4) than allowed by state specification. Rhode Island gravel is above state specifications at the 12.7mm (0.5 in.), 9.5mm (.0375 in.) and 0.075mm (No.200) mesh sizes.

4.2.2 Classification of Subbase Materials

Subbase materials were classified in accordance with AASHTO M145-87. Most subbase aggregates were in the A-1-a classification as would be expected (Table 4.2). Only the Massachusetts Processed Gravel and the Rhode Island Route 2 materials were classified A-1-b. Both classifications usually contain stone fragments, gravel and sand as their significant constituent materials.

4.2.3 Moisture-Density Relations

The procedures of AASHTO T180-93 (ASTM D1557-91) using a 44.5 N (10 lbf) mechanical rammer producing a compactive effort of 56,000 ft-lbf/ft^3 were followed to determine the optimum moisture content (OMC) and maximum dry density (γ_{MAX}). Testing was performed using a six inch diameter mold with a volume of 0.726 cubic feet by AASHTO method D. The results are summarized in Table 4.3. Compaction curves have been plotted as shown in Figures 4.10 through 4.16. The 100% saturation or zero air voids (Z_{av}) was determined using the following formula:

$$Z_{av} = [(G_s)(\gamma_w)]/[1+(M_c)(G_s)] \qquad (4.1)$$

where

G_s = subbase specific gravity

γ_w = 62.43, and

M_c = moisture content

4.3 Resilient Modulus of Granular Subbase Materials

4.3.1 AASHTO T292-91 Procedure

Currently AASHTO recommends the designation T292-91 for preparing and testing untreated subgrade soils and untreated subbase and base materials to determine the resilient modulus. The physical testing conditions are to represent a simulation of stress states of materials beneath flexible pavements subjected to moving wheel loads. The subbase materials were tested at 20°C (68°F) (room temperature) and at the OMC.

A minimum of 90 percent by weight of the subbase material used to prepare the specimen had a maximum particle size finer than 1/6 the specimen's diameter (4 in./6 = .67 in. or 17 mm). The maximum size of the remaining material was no larger than 1/4 of the specimen's diameter (1 in. or 25mm). The coarse material was discarded and noted. This material was prepared at OMC, then was kept in a double plastic bag sealed overnight. This allows the moisture to equilibrate in the sample for even distribution. Prior to testing a fraction of this material was taken to determine moisture content. An additional sample was taken after testing for the same purpose. The two values were averaged, and reported as the representative moisture content.

The previously described material was compacted into a split mold in 5 equal layers with 25 blows per layer using a 5.5 lb rammer and 12 inch drop to obtain a specimen of 100 mm (4 in.) diameter and 200 mm (8 in.) height. With this compaction procedure, a 90 percent maximum dry density (AASHTO T-180) was usually achieved. (Lee, et al.1994)

The subbase specimen was confined in the triaxial chamber using air pressure to achieve the required stress. The triaxial chamber is housed in the walk-in environmental chamber to maintain a temperature of 20^{0}C (68^{0}F). A pneumatic actuated loading ram produces the required deviator stress. The pulse duration time (on time) and pulse interval (off time) controls the loading and unloading time. They are metered in terms of percent on the control cabinet. For pulse duration 100% corresponds to about 0.5 sec; and for pulse interval 100% indicates about 2.0 sec. Therefore, for a test duration of 0.1 sec. loading and 0.9 sec. unloading it is required to set the pulse duration at about 20% and pulse interval at about 45% (Kovacs et al. 1991). Pulse times of 0.03 to 0.05 seconds are appropriate for full-depth construction with 5 to 12 inches of asphalt concrete and with vehicle speeds of 50 to 60 mph (Barksdale 1971). Resting time between individual pulses of about 0.7 to 2 seconds is similar to actual conditions in the field (Terrel et al. 1974). Load (pulse) duration for specimens of cohesive soils of 0.05 sec. to 0.1 sec. are recommended in Table 1 of AASHTO T292-91. This designation provides no recommended load duration for subbase materials. A load duration of 0.05 was chosen because it produced the optimum load ramp for monitoring with H&V equipment. Loading sequences used were in accordance with Table 6 of AASHTO T292-91.

Vertical deformation of the subbase specimen is measured by a pair of 100 mm (4 in.) Linear Variable Differential Transformers (LVDTs) attached to the sides of the specimen within the chamber. The deviator stress was determined by a resistive element load cell capping the specimen. Calibrated output voltage is recorded and manipulated with a software program named Resilient Modulus (RM) in a personal computer. The confining stress is inputed into the RM program for each loading sequence. The resilient modulus of the specimen is calculated and printed by the RM program. The output provides load (lbs), axial strain, bulk stress (psi), confining stress (psi) and modulus (psi).

The result of a single test for granular materials can be presented in a mathematical form that directly incorporates the stress sensitivity of the modulus value in terms of the bulk stress θ (first stress invariant) by $E_{SB} = k_1 \theta^{k2}$. The constants k_1 and k_2 are obtained from regression analysis of the test results and depend on the type of material and physical properties of the specimen during the test (Rada and Witczak 1981). Results for this project are presented in this form in Figures 4.17 through 4.24.

A first degree regression analysis was performed on the 18 data points generated during the testing by the MR program. The linear regression equation (Log E_{SB} = Log k_1 + $k_2$logθ) provides the best fit straight line for the 18 data points. Test results are judged upon their production of linear results. Perfect linear results indicate that modulus varies with bulk stress proportionally. A correlation coefficient (the product-moment) R^2 measures the strength of linear association between two variables. The correlation coefficient is always between -1 and 1. A coefficient of 1 means all the points lie on a perfect straight line, and dependent variable (logE_{SB}) increases as independent variable (Logθ) increases: a coefficient of -1 means all the points lie on a perfectly straight line and dependent variable decreases as independent variable increases. Correlation is a statistic which is sensitive to outliers. The quantity R^2 is the proportion of variance from the regression line calculated with the following formula (Hutchinson 1993):

$$R^2 = \frac{\Sigma[\text{estimate line}(y)-\text{mean}(y)]^2}{\Sigma[(\text{actual}(y)-\text{mean}(y)]^2} \qquad (4\text{-}2)$$

where

$y = \log E_{SB}$

It was the URI team's criteria to achieve R^2 values above 0.7. In some cases the material was not stable enough to provide this result. The material was tested again; and the result was reported if it was reproducible.

Test results based on the AASHTO T292-91 procedure are summarized in Table 4.4. The AASHTO Guide recommended bulk stress state asphalt concrete thickness greater than 4 inches equals 5 psi (Sec. II-22). Therefore, the bulk stress for AASHTO T292-91 procedure has been assumed as 5 psi for the graphical comparison in Figure 4.25. It should be noted that bulk stress is dependent upon the pavement structure and traffic loading. Detailed bulk stress analysis will be discussed in a later section.

4.3.2 AASHTO TP46 Procedure

The materials collected from six New England states were again tested in accordance with the procedure of AASHTO TP46. The physical testing conditions are to represent a simulation of stress states of materials beneath flexible pavements subjected to moving wheel loads. Similar to the AASHTO T292-91 procedure, the subbase materials were tested at 20°C (68°F) (room temperature) at the OMC.

The granular base and subbase materials are classified as either Type 1 or Type 2. Material Type 1 includes all untreated granular base and subbase material and all untreated subgrade soils which meet the criteria of less than 70 percent passing the 2.00-mm (No. 10) sieve and less than 20 percent passing the 75-µm (No. 200) sieve, and which have a plasticity index of 10 or less. Material Type 2 includes all materials which do not meet the criteria for material Type 1. All the procured New England materials based on their sieve analysis fall under Type 1 classification.

The particle size exceeding 25 percent of the mold diameter were discarded. A step by step procedure for sample preparation (Data sheet UG-2) is shown in Figure 4.26. Once the appropriate amount of water is added to the soil and mixed thoroughly, the mixture was placed in a plastic bag. The bag was sealed, and it was placed in a second bag, and sealed. The sample was cured for 16 to 48 hours. The mass of the wet soil and container was determined to the nearest gram.

A split mold, with an inside diameter of 152 mm having a height of 381 mm (so that there is sufficient height to allow guidance of the compaction head for the final lift) was used to prepare a specimen. Using the Data Sheet UG-2, the amount of material required per layer was estimated to obtain the desired density. Specimens were compacted in 6 equal layers in a split mold mounted on the base of the triaxial cell. The vibratory compactor is an electric rotary with a rated input of 750 to 1,250 watts and capable of 1800 to 3000 blows per minute. The thickness of the compactor head was 13 mm thick and has a diameter of 146 mm.

The chamber is made of polycarbonate acrylic which is a see through material to facilitate the observation of the specimen during testing. The procedure requires a loading device, which is capable of applying repeated cycles of load with a duration of 0.1 sec loading and 0.9 sec unloading.

In order to check for leakage caused by poor connections, holes in the membrane, or imperfect seals at the cap of the base, the specimens bottom drainage line is connect to the vacuum source through the medium of bubble chamber. When the leakage has been eliminated, the vacuum supply was disconnected. The air pressure supply line was connected to the triaxial chamber and the specified pre-conditioning pressure of 15 psi was applied to the test specimen.

The total load applied to the sample (P_{max}) including the contact and cyclic (resilient) loads is as follows:

$$P_{max} = P_{contact} + P_{cyclic} \quad \text{......................................} \quad (4\text{-}3)$$

Where $P_{contact}$ = contact load ($0.1P_{max}$)

P_{cyclic} = cyclic axle load ($P_{max} - P_{contact}$)

Vertical deformation of the subbase specimen is measured by a pair of Linear Variable Differential Transformers (LVDTs) fixed opposite sides of the piston rod outside the test chamber as shown in Figure 4.27. The cyclic stress was determined by a load cell capping the piston as shown in Figure 4.28. The output is sent to an 8500 Instron tower which is in turn recorded in a software program named LabView from National Instruments customized by Instron Corporation. If the total vertical permanent strain reaches 5 percent during conditioning, the conditioning process would be terminated and a notation would be added to the report form. However, all the subbase materials used in this study did not constitute any such problems. Once the testing sequence is complete, the material was subjected to quick shear test. The output provides cyclic stress (S_c), confining pressure (S_s), axial strain, regression constants (K_1, K_2, and K_5) and quick shear results.

The result of a single test for granular materials can be presented in a mathematical form that directly incorporates the stress sensitivity of the modulus value in terms of the cyclic stress (S_c) and confining pressure (S_s) by $M_R = K_1(S_c)^{K2}(S_s)^{K5}$. The constants K_1, K_2, and K_5 are obtained from regression analysis of the test results and depend on the type of material and physical properties of the specimen during the test (Rada and Witczak 1981). Results of this project are presented in this form in Figures 4.29 through 4.36. Test results based on AASHTO TP46 procedure are summarized in Table 4.5.

4.4 Stress Analysis and Resilient Moduli for New England Subbase Materials

Because of the nonlinear (stress-dependent) properties of most granular materials, the resilient modulus test is conducted at combinations of confining pressure and deviator pressure ratios (Rada and Witzak 1981). These external pressures applied to a cylindrical specimen produce strains.

In a two dimensional analysis with the stress components σ_x, σ_y and τ_{xy} at any point of a plate in a condition of plane stress or plane strain, the stress acting on any plane through this point perpendicular to the plate and inclined to the x or y axes can be calculated from the equations of statics. For each pair of parallel sides of a cubic element, one symbol is needed to denote the normal component of stress and two more symbols to denote the two components of shearing stress. To describe the stresses acting on six sides of the element three symbols, i.e., σ_x, σ_y and σ_z are necessary for normal or principle stresses (acting perpendicular to a chosen cubic element orientation) Figure 4.37; and six symbols, i.e., τ_{xy}, τ_{yz}, τ_{xz}, τ_{zx}, τ_{yx} and τ_{zy} for shearing stresses (Figure 4.38).

Let us consider the particular case of deformation of the rectangular parallelepiped on which $\sigma_z = \sigma$, $\sigma_y = -\sigma$ and $\sigma_x = 0$ (Figure 4.39). Cutting out an element by planes parallel to the x axis and 45^0 to the y and z axes, it may be seen by summing up the forces (y&z) along and perpendicular to bc (the 45^0 plane) that the normal stress in the sides of this element is zero and the shearing stress on the sides is $\tau = 1/2(\sigma_z\text{-}\sigma_y) = \sigma$. Such a condition of stress is called pure shear. The elongation of the vertical element is equal to the shortening of the horizontal elements. The relation between strain and stress is defined by the constants E (Modulus of Elasticity in tension and compression) and ν (Poisson's Ratio) (Timoshenko and Goodier 1970).

The principal stresses and principal axes can represent the state of stress by components in any set of x, y and z axes. No matterwhat the orientation chosen for the axes must give the same three roots for three homogeneous linear equations which include stress and the plane on which they act and the shear deformation. Choosing the principal axes themselves for the x, y, and z axes, the static equation may be written with the magnitude of the principal stresses equal to S_1, S_2 and S_3 as follows:

$$\sigma_x + \sigma_y + \sigma_z = S_1 + S_2 + S_3 \text{......................} (4\text{-}4)$$

The expressions on the left are "stress invariants" (Timoshenko and Gordier 1970).

A triaxial chamber suitable for use in repeated load testing for resilient modulus is similar to that used in common triaxial testing. Air provides the confining pressure to the specimen, i.e. σ_s (static). The principal stress previously described as σ_z is composed of the confining stress σ_s, and a deviator stress σ_d, produced by a pneumatic loading ram to reproduce the stress of rapid wheel loading on a pavement structure (Figure 4.40). It can be clearly seen in the diagram that $\sigma_{s2} = \sigma_{s3}$ on the cylindrical specimen.

Stresses that occur in a pavement structure are present in two forms: static stresses from the overburden pavement materials and dynamic (deviator) stresses caused by moving wheel loads.

In order to calculate these stresses an appropriate depth of analysis must be selected. The URI research team had used the midpoint of the subbase for previous analysis of this layer (Lee et al. 1994). This will produce a 50% reliability factor since bulk stress decreased with the depth of analysis. The dynamic axle loading to be used in the deviator stress calculation is based upon the federal legal axle load for a single axle (Lee et al. 1990). It was calculated using a multi-layer elastic program ELSYM5 (Ahlborn 1972). ELSYM5 is a simple linear elastic analysis program capable of computing stresses, strains and deflections at chosen depths in a multi-layered pavement structure. It requires thickness, elastic modulus, Poisson's ratio, wheel load and tire pressure as input parameters. It was suggested that in cases where ELSYM5 calculated negative (tensile) confining stresses, σ_3 and the intermediate principal stress σ_2 were assumed to be the same when calculating the first stress invariant(Parker and Elton 1989). This suggestion has been adopted in the present study.

A sample ELSYM5 elastic analysis at the subbase mid-depth is shown in Figure 4-41. In addition a graph of the results at the top, one third and midpoint of the subbase was plotted in Figure 4-42. The elastic modulus values used in this example have been evolved through two resilient modulus testing programs at URI (Kovacs et al.1991; and Lee, et al. 1994). Seed modulus values used for ELSYM analysis are summarized in Table 4.6. The present values used were abstracted from the recent laboratory test results:

layers 1, 2 and 3 from Table 4.7 Summary of Resilient Moduli at 68^0F for Asphalt Mixtures Prepared in the Laboratory (Lee et al. 1994b).

layer 4 from Table 3.15 Resilient Moduli of Subbase Materials and Subgrade Soils Using URI Method (Lee et al. 1994a).

layer 5 from Table 5.1 A Sample Calculation of Effective Mr for Rhode Island Rt 2 Subgrade (Lee et al.1994a).

Certain basic properties are assumed when applying elastic analysis. Forces that produce deformation do not exceed the limit where permanent deformation occurs and therefore bodies undergoing the action of external forces are *perfectly elastic*. The matter of elastic bodies is assumed *homogeneous* and continuously distributed over its volume and physical properties are uniform. The body is *isotropic* so elastic properties are the same in all directions. As long as the geometric dimensions of the body are very large in comparison to a single crystal these assumptions are appropriate.

Under the action of external forces, internal forces are produced between the parts of a body. It is assumed that internal forces are uniformly distributed over a cross sectional area in the same manner as hydrostatic pressure is continuously distributed over the surface in which it acts. The internal forces produce internal stresses, defined as the force per unit surface area.

A simple case of a prismatic bar subjected to tension forces uniformly distributed over both ends will produce internal forces on a midpoint cross sectional area. The intensity of this distribution (stress) may be calculated by dividing the total tensile force P by the cross-sectional area A. In the general case the stress is not uniformly distributed over the cross section. Therefore forces acting on different elements within a material are reduced to resultants. The value of P/A will give us the magnitude while the limiting direction of the P resultant will give us the direction. The side of a cubic element perpendicular to an axis are given the notation of that axis (i.e., the y axis σ_y). The notations σ_x, σ_y, σ_z are required necessary to describe the stresses acting on the six sides of a cubic element.

Hooke's law describes a linear relationship between the components of stress and strain. The extension of an element in the x direction is accompanied by lateral strain. The equation for this contraction contains a constant (ν) called *Poisson's Ratio.*

If an element is subjected simultaneously to the action of normal stresses on the x, y and z planes uniformly distributed over its sides, the resultant components of strain can be obtained from the superposition of the strain components of the three equations:

$$\varepsilon_{x\,[strain]} = \nu(\sigma_x/E) \qquad (4.5)$$

$$\varepsilon_{y\,[strain]} = -\nu(\sigma_x/E) \qquad (4.6)$$

$$\varepsilon_{z\,[strain]} = -\nu(\sigma_z/E) \qquad (4.7)$$

where

E = modulus of elasticity

This has been found consistent with numerous test measurements.

The results of this stress analysis are applicable to any kind of continuous medium (i.e., viscous fluid or plastic solid). In the general case of stress distribution in a three dimensional or cubic element there are three components of stress σ_x, σ_y and σ_z. Since stresses vary continuously over the body of a very small tetrahedron, the stress acting on a plane will approach the stress on a parallel plane to the point of interest when this element is made infinitesimal. Since the element is so small, we can assume the stresses to be uniform.

Consider the normal component of stress acting on a plane. As the plane rotates about the point of interest, the end of the *vector* always lies on the surface of the second degree. It is well known that in the case of a surface of the second degree, it is always possible to find for the axes x, y, z such directions that the shear terms of the equation vanish and the *resultant* stresses are perpendicular to the planes on which they act. We call these stresses the *Principal Stresses* at the point, their directions the principal axes, and the planes on which they act the *principal planes.* The stress at this point is completely defined, if the directions of the principal axes and the magnitudes of the three principal stresses are given. If the coordinate axes are taken in the direction of the principal axis, the shearing stresses are zero (Timoshenko and Gordier 1970).

The static stress z axis component may be calculated based upon each layer thickness of the pavement structure, and an estimated dry density for the material. The static stress for the x and y axis uses the geo technical coefficient of earth pressure at rest. The formula is provided with a sample calculation for a typical pavement cross section in the State of Rhode Island. The elastic modulus of the subbase layer was computed using data produced at the URI lab. (Figure 4.43).

It should be noted that as realistic modulus values have been applied to the ELSYM5 elastic analysis, the deviator stress values have decreased. It was also confirmed that the modulus of the subgrade soil is the primary design parameter for a pavement design. This has been verified during ELSYM5 analysis in which the modulus of the subgrade soil was decreased to a lower assumed value. The elastic modulus of subbase is highly influenced by the bulk stress calculation. When materials are subjected to higher levels of stress, the moduli value will increase accordingly. Resilient modulus of subbase materials according to AASHTO T292-91 and AASHTO TP46-94, with bulk stresses analyzed at mid-depth have been compiled in Table 4.7 and Table 4.8 respectively. Corresponding layer coefficients were also included. Figures 4.44 and 4.45 show graphical presentation of resilient moduli in ascending order by procedures of AASHTO T292-91 and TP46-94, respectively.

Concurrent research by at the University of Illinois (UofI) at Urbana-Champaign on subbase material and crushed aggregate in particular may provide good comparison (Garg and Thompson 1997). The testing procedure was developed by the UofI research team incorporating a 300 mm (6 in.) mold. The results were analyzed in the Rada & Witzak model (Rada and Witzak 1981). Base and subbase material CL-5sP and CL-6sP had 15 percent and 100 percent crushed/fractured particles, respectively. The results compared at computed Massachusetts bulk stress (7 psi) would range from 10 ksi to 18 ksi. Using the computed Vermont bulk stress (13 ksi) they range from 12 ksi to 20 ksi. URI team results by AASHTO T292-91 for Massachusetts and Vermont crushed stone are 7 ksi and 9 ksi, respectively. It was observed that URI laboratory results are definitely lower when compared to those of the UofI study. However URI test results by AASHTO TP46-94 were 12 and 14.5 ksi for Massachusetts and Vermont crushed stone, respectively. It should be noted that both values are comparable with the ones of Uof I research team.

Table 4.1 List of Tests Performed to Characterize Subbase Materials

Test Name	AASHTO Designation	Parameter To Be Determined
Sieve Analysis	M43-88	Gradation
Liquid Limit	T89-93	Liquid Limit
Plastic Limit	T90-92	Plastic Limit
Proctor	T180-90	Max. Dry Density (MDD) Opt. Moisture Cont. (OMC)
Permeability	T215-70	Coefficient of Permeability (k)
Elastic Modulus for Subbase Materials	T292-91	Resilient Modulus (E_{SB}) Layer Coefficient (a_3)

Table 4.2 Classification of Subbase Materials

State, **Material Name** **(Material Code)**	**Classification of** **Subbase Materials**	**Plasticity** **Index** **(PI)**
Connecticut Bank Run Gravel (CT/BRG)	A-1-a	Non-plastic
Maine Frenchville Gravel (ME/FG)	A-1-a	Non-plastic
Maine Sabbatus Subbase (ME/SB)	A-1-a	Non-plastic
Massachusetts Crushed Stone (MA/CS)	A-1-b	Non-plastic
Massachusetts Processed Gravel (MA/PG)	A-1-a	Non-plastic
New Hampshire Sandy Gravel (NH/SG)	A-1-a	Non-plastic
Rhode Island RT 2 Sandy Gravel (RI/SG)	A-1-b	Non-plastic
Vermont Crushed Stone (vt/cs)	A-1-a	Non-plastic

Note: PI = LL - PL : If the plastic limit (PL) = 0, the material is non-plastic and the liquid limit (LL) test may be omitted.

Table 4.3 Summary of Compaction Test Results

Material	Proctor Test AASHTO T 180-90	
	OMC %	γ_d Kg/m^3 (pcf)
Connecticut Bank Run Gravel (CT/BRG)	8.6	2020.8 (126.1)
Maine Frenchville Subbase (ME/FG)	6.1	2329.0 (145.4)
Maine Sabbatus Subbase (ME/SG)	8.1	2181.1 (136.1)
Massachusetts Crushed Stone (MA/CS)	7.1	2235.5 (139.5)
Massachusetts Processed Gravel (MA/PG)	8.5	2003.2 (125.0)
New Hampshire Sandy Gravel (NH/SG)	8.6	1888.0 (117.5)
Rhode Island Sandy Gravel* (RI/SG)	6.0	2078.5 (129.7)
Vermont Crushed Stone (VT/CS)	8.0	2162.4 (135.0)

*These values were abstracted from the research report (Lee, Marcus and Mooney 1996)

Table 4.4 Summary of Resilient Modulus Test Results (AASHTO T292-91)

Material Code	Source	K_1	K_2	R^2	Moisture Content,%
CT BRG	Bank Run	2,518	.62	.86	8.0
ME FG	Processed	6,830	.47	.80	5.8
ME SG	Bank Run	2,111	.70	.93	7.4
MA CS	Processed	1,326	.84	.79	6.0
MA PG	Processed	3,058	.58	.95	8.3
NH SG	Bank Run	2,365	.68	.82	8.6
RI SG	Bank Run	5,809	.37	.84	6.0
VT CS	Processed	1,333	.75	.98	7.8

Note: $E_{SB} = K_1\theta^{K2}$ (AASHTO Guide)

Table 4.5 **Summary of Resilient Modulus Test Results (AASHTO TP46)**

State/ Material	Cyclic Stress S_c	Conf ining Stress S_3	Regression Constants K1	K2	K5
CT/BRG	4.48	1.99	9112	0.14308	0.33267
ME/FG	5.18	2.59	9131	0.12119	0.38923
ME/SG	5.18	2.59	7412	0.18762	0.40123
MA/CS	5.15	1.93	6630	0.20832	0.38239
MA/PG	5.15	1.93	8520	0.14347	0.37425
NH/SG	3.53	3.33	11673	-0.0114	0.36883
RI/SG	4.95	1.93	10201	0.11055	0.35028
VT/CS	11.46	1.78	7525	0.16153	0.45062

Table 4.6 Seed Modulus Values for ELSYM5 Analysis

Layer	1991	1994	Present
1	350.0	297.4	325.0
2	350.0	465.3	540.0
3	350.0	585.3	480.0
4	35.0	20.4	20.3
5	9.0	14.3	6.5

Note: Modulus Values in ksi

Table 4.7 **Resilient Moduli of New England Subbase Materials (Determined by AASHTO T292-91 procedure)**

Material	$\Sigma\sigma_s$ psi	$\Sigma\sigma_d$ psi	θ psi	Modulus Equation	E_{sb} (psi)	a_3
CT/BRG	1.99	4.48	6.47	$2517.2\theta^{0.62}$	8,010	0.05
ME/FG	2.59	5.18	7.79	$6830.4\theta^{0.47}$	17,945	0.13
ME/SG	2.59	5.18	7.79	$2111.0\theta^{0.70}$	8,883	0.06
MA/CS	1.93	5.15	7.08	$1325.8\theta^{0.84}$	6,862	0.03
MA/PG	1.93	5.15	7.08	$3058.8\theta^{0.58}$	9,518	0.06
NH/SG	3.33	3.53	6.86	$2365.1\theta^{0.68}$	8,760	0.06
RI/SG	1.93	4.95	6.87	$5808.7\theta^{0.37}$	11,851	0.09
VT/CS	1.78	11.46	13.24	$1332.9\theta^{0.75}$	9,251	0.06

Note: 1. Bulk stress was analyzed at mid-depth.

2. $a_3 = 0.227\ (\log_{10}E_{SB}) - 0.839$ (Source: 1993 AASHTO Guide)

Table 4.8 Resilient Moduli of New England Subbase Materials (Determined by AASHTO TP46-94 procedure)

Material	S_3 psi	S_c psi	Modulus Equation	E_{sb} (psi)	a_3
CT/BRG	1.99	4.48	$9112(S_c)^{0.14308}(S_3)^{0.33267}$	14,198	0.10
ME/FG	2.59	5.18	$9131(S_c)^{0.12119}(S_3)^{0.38923}$	16,142	0.12
ME/SG	2.59	5.18	$7412(S_c)^{0.18762}(S_3)^{0.40123}$	14,784	0.11
MA/CS	1.93	5.15	$6630(S_c)^{0.20832}(S_3)^{0.38239}$	11,995	0.09
MA/PG	1.93	5.15	$8520(S_c)^{0.14347}(S_3)^{0.37425}$	13,786	0.10
NH/SG	3.33	3.53	$11673(S_c)^{-0.0114}(S_3)^{0.36883}$	17,932	0.13
RI/SG	1.93	4.95	$10201(S_c)^{0.11055}(S_3)^{0.35028}$	15,327	0.11
VT/CS	1.78	11.46	$7525(S_c)^{0.16153}(S_3)^{0.45032}$	14,469	0.11

Note: 1. Stresses were analyzed at mid-depth.

2. $a_3 = 0.227 (\log_{10}E_{SB}) - 0.839$ (AASHTO 1993 Guide)

AASHTO T292-91 RESILIENT MODULUS

TRIAXIAL CHAMBER WITH INTERNAL LVDT'S AND LOAD CELL

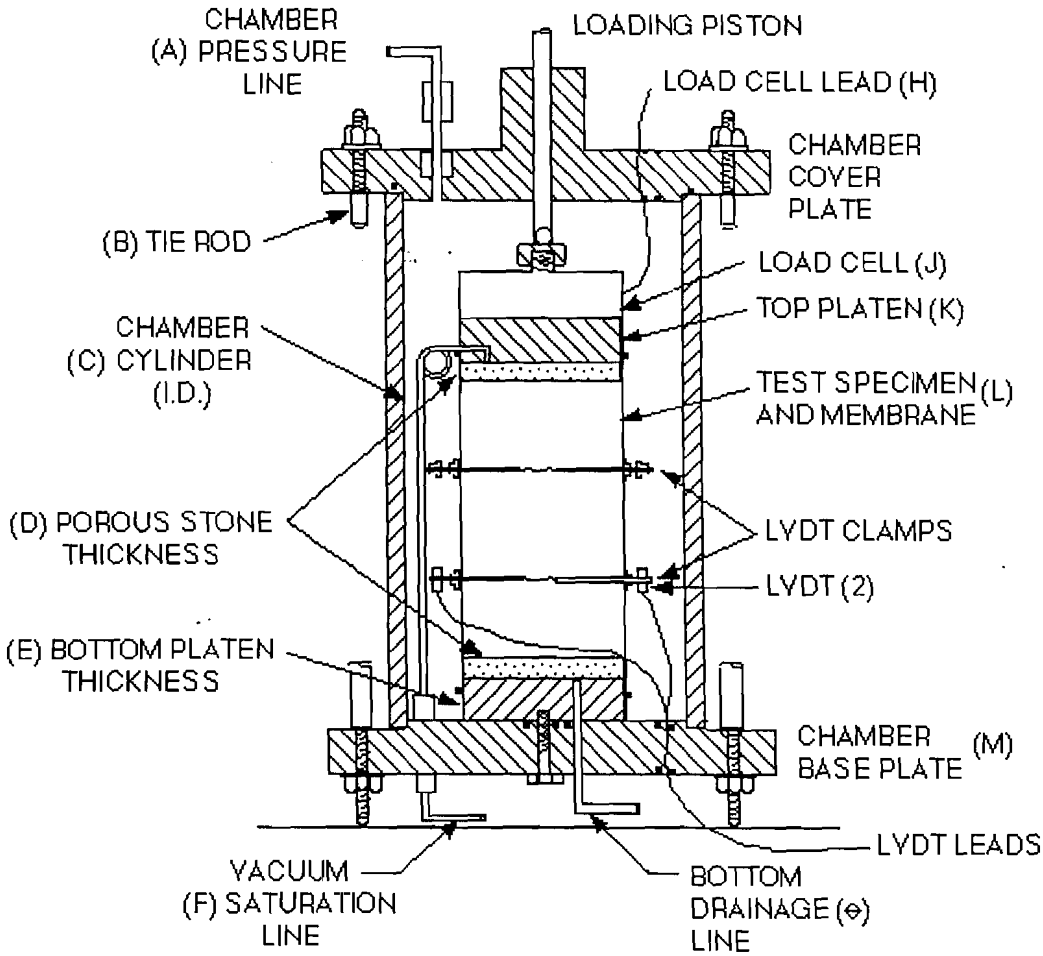


TABLE OF MEASUREMENT (TYPICAL)

DIMENSION	A	B	C	D	E	F	G	H	I	J	K	L	M	N	
METRIC, mm	6.4	12.7	152.4	6.4	38.1	6.4	12.7	NOTE 1	191	NOTE 1	38.1	NOTE 2	25.4	6.4	
ENGLISH, in	0.25	0.50	6.00	0.25	1.50	0.25	0.50		0.75		1.50		1.0	0.25	

NOTE:
1. Dimensions vary with manufacturer.
2. Dimensions vary with specimen size.

Figure 4.1 H & V pneumatic "Resilient Modulus Repeated Load Test System" with L = 100 mm (4 in.) x 200 mm (8 in.)

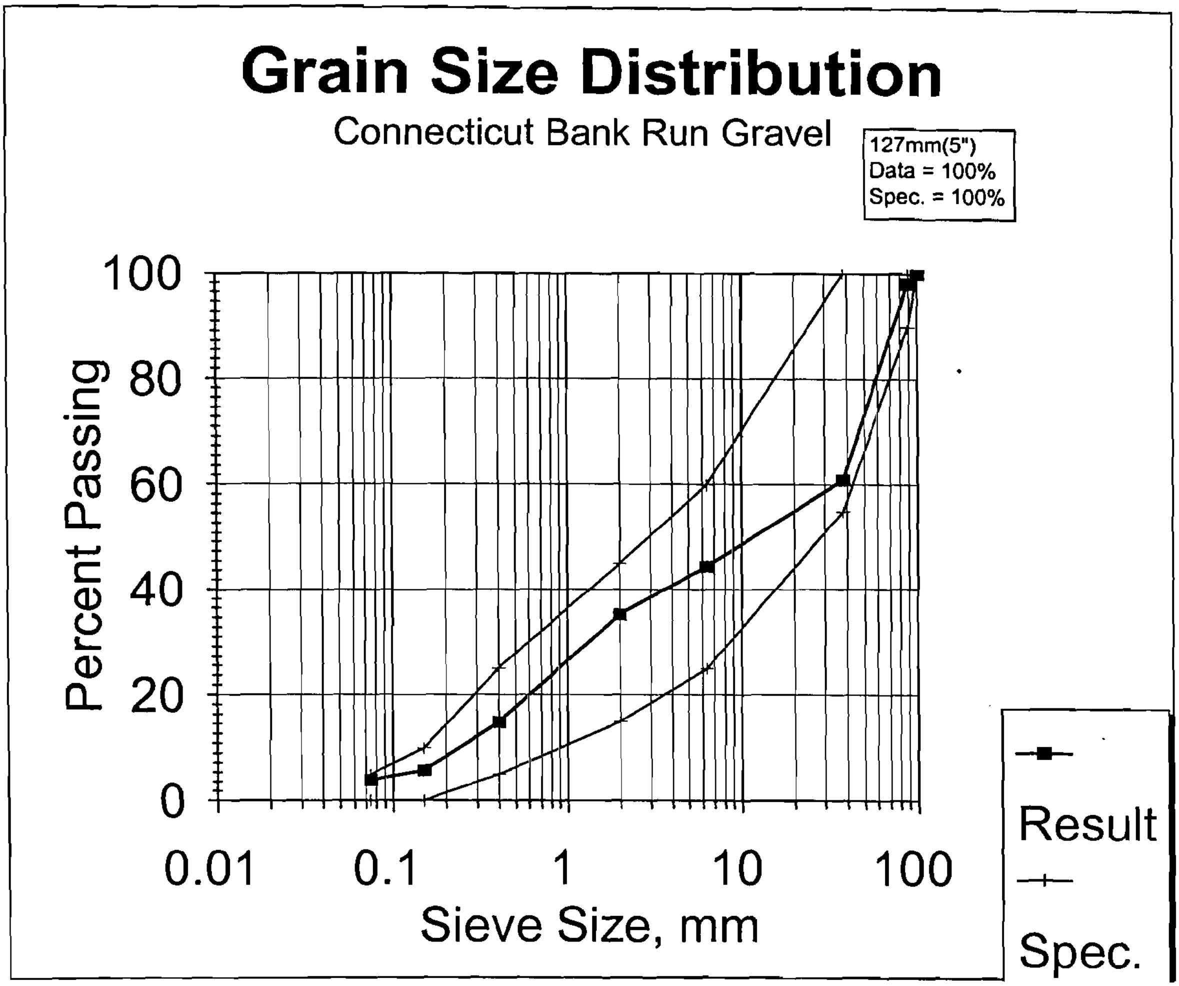


FIGURE 4.2 Connecticut Bank Run Gravel Grain Size Distribution

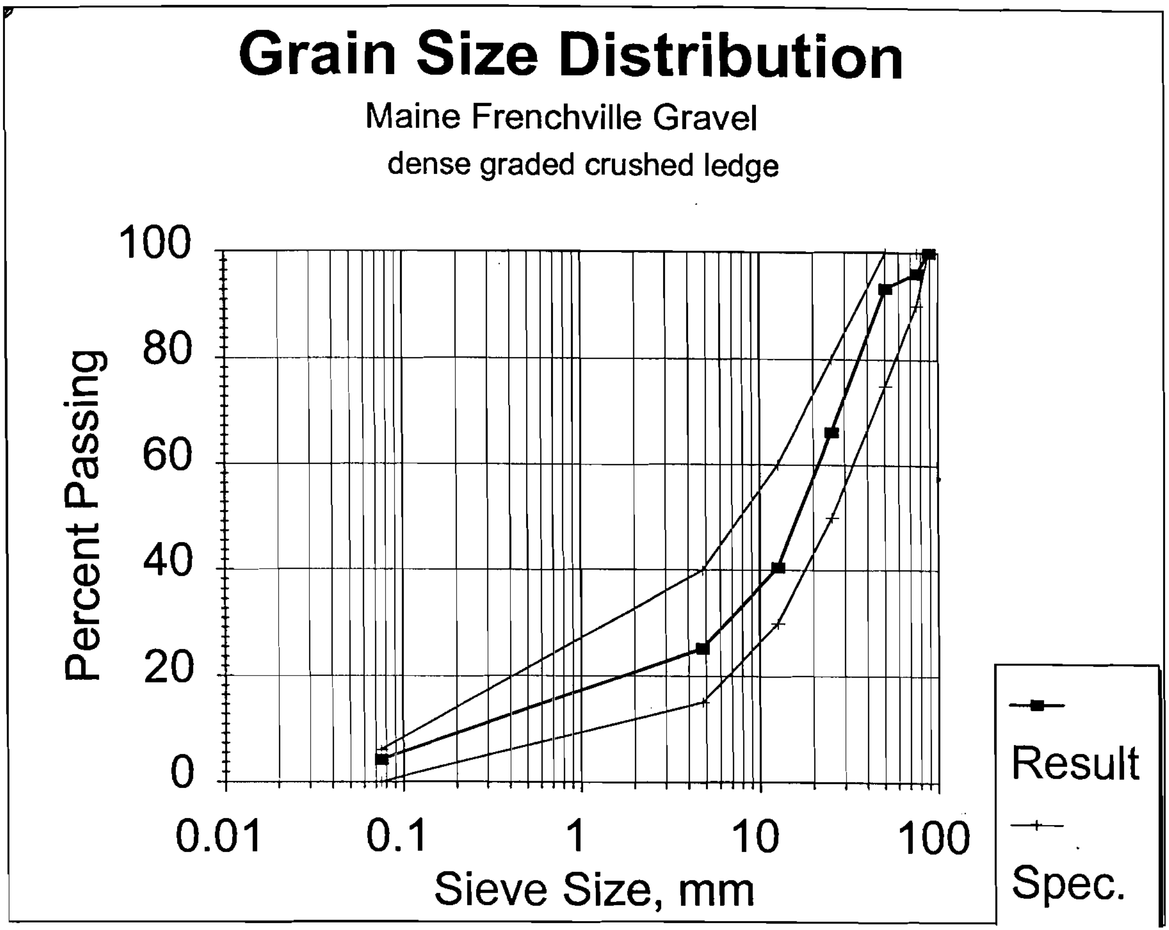


Figure 4.3 Maine Frenchville Gravel Bank Run Gravel Grain Size Distribution

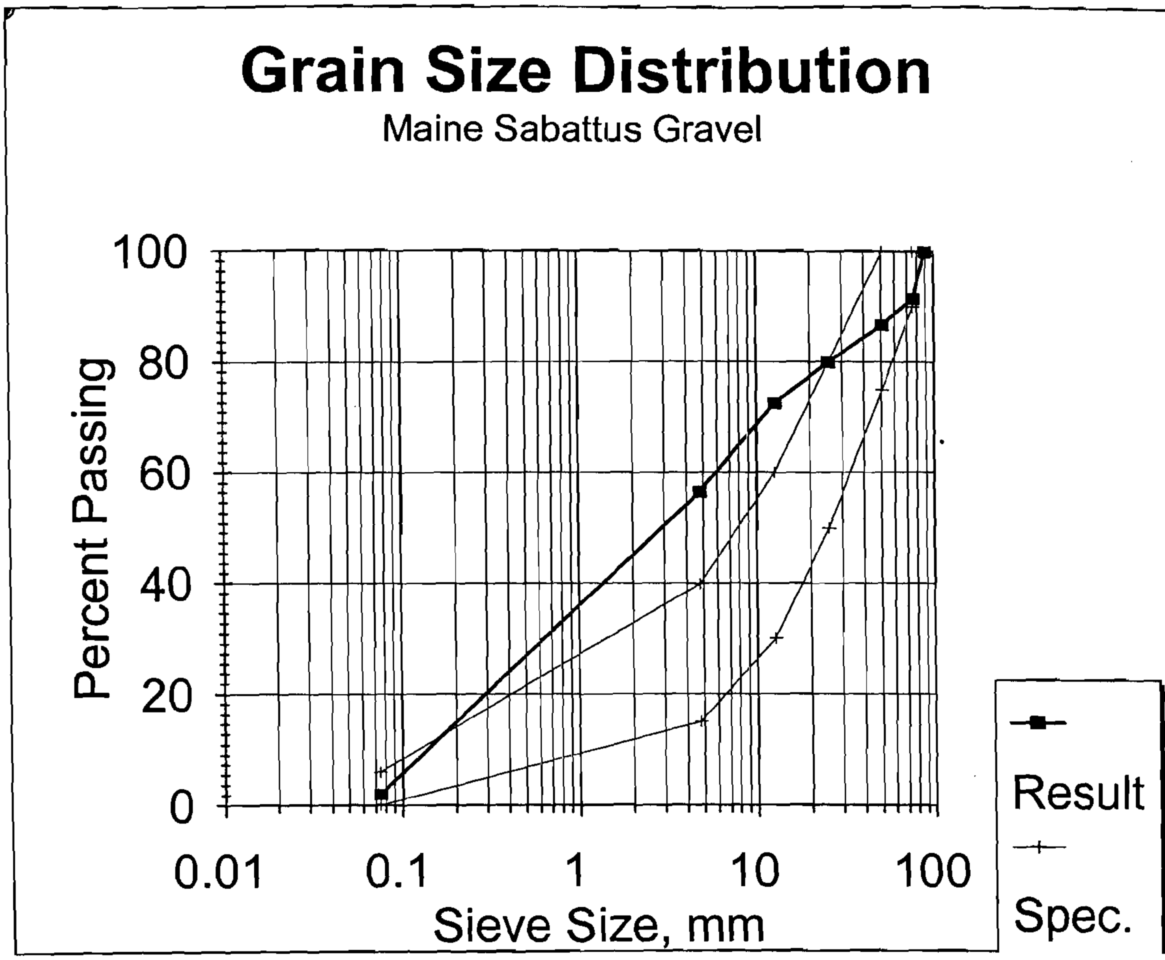


Figure 4.4 Maine Sabattus Gravel Grain Size Distribution

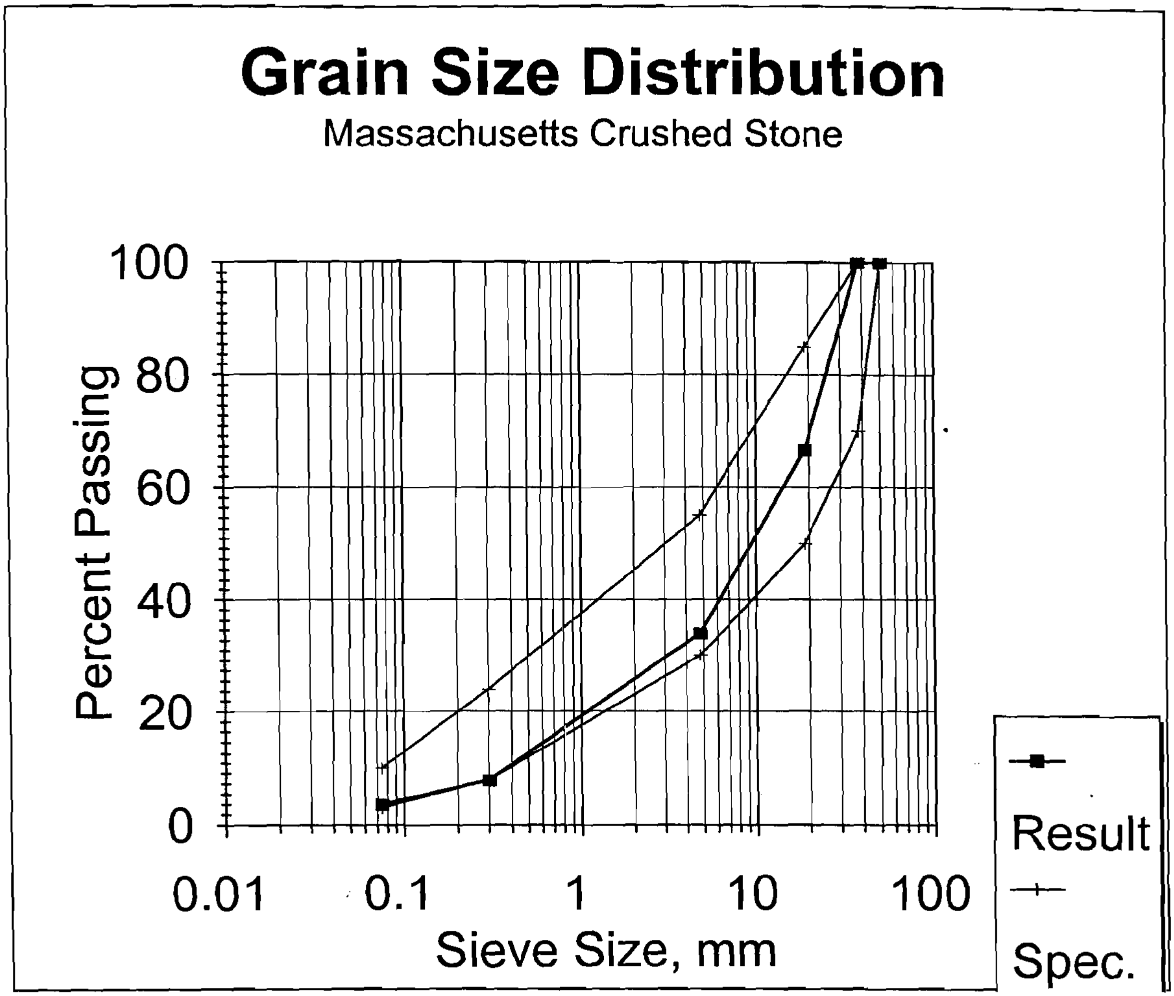


Figure 4.5 Massachusetts Crushed Stone Grain Size Distribution

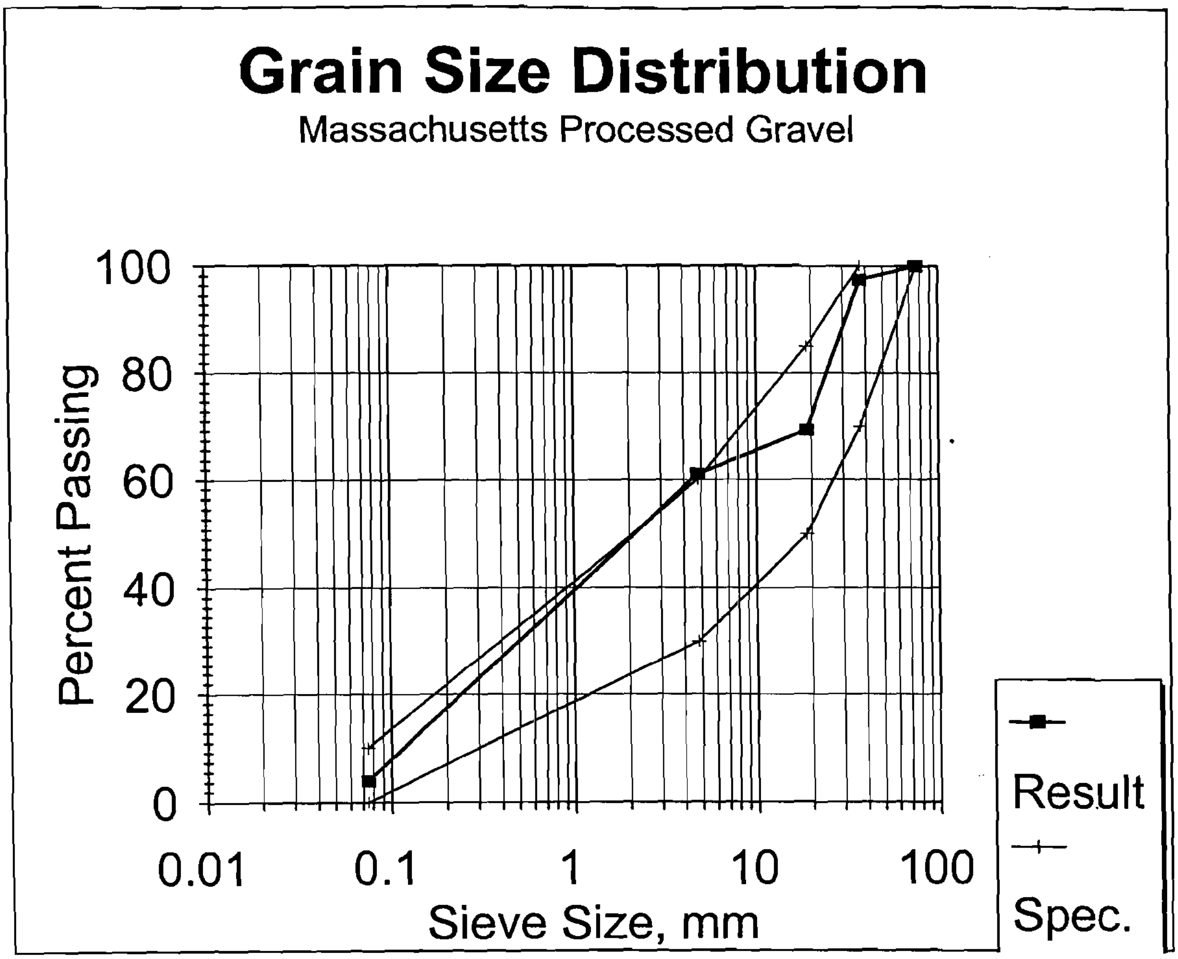


FIGURE 4.6 Massachusetts Processed Gravel Grain Size Distribution

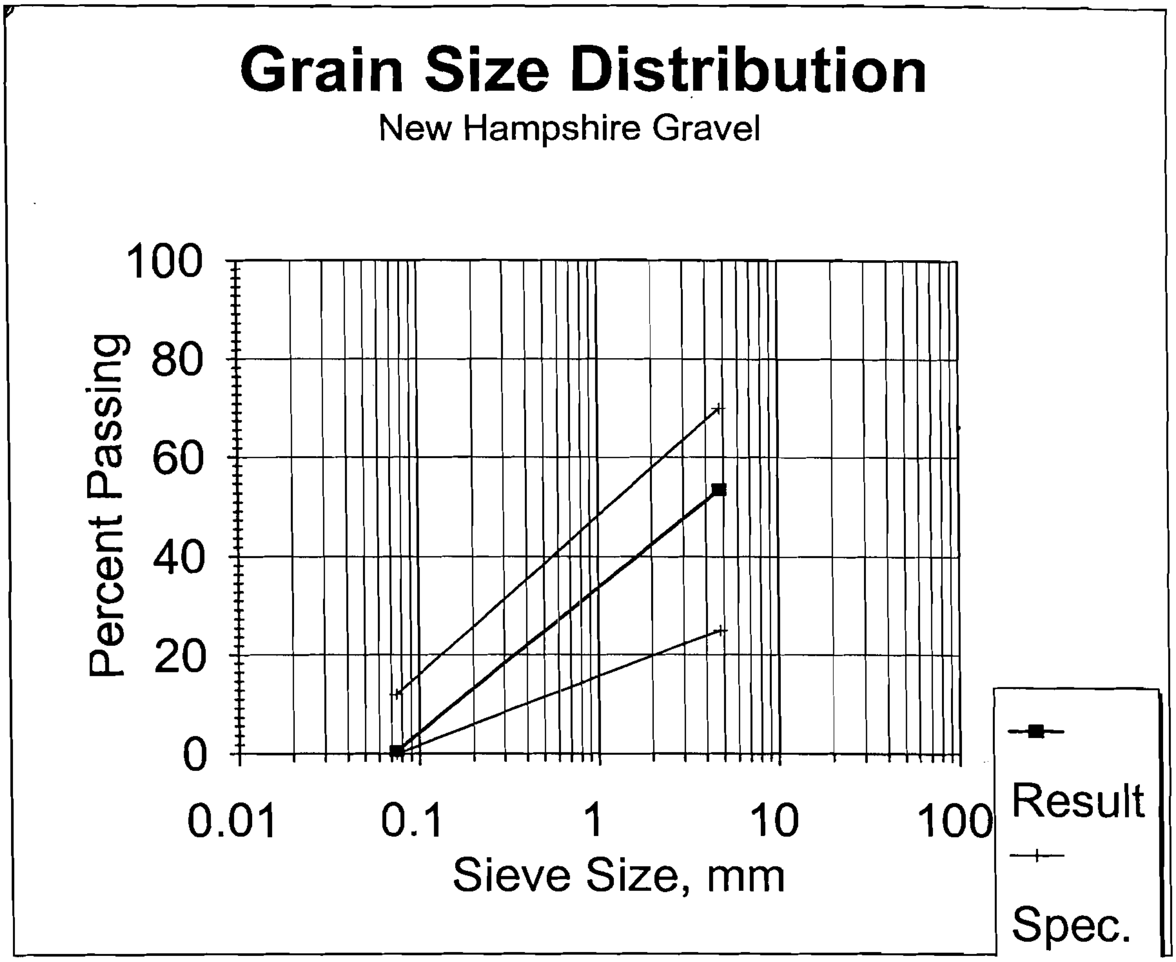


FIGURE 4.7 New Hampshire Sandy Gravel Grain Size Distribution

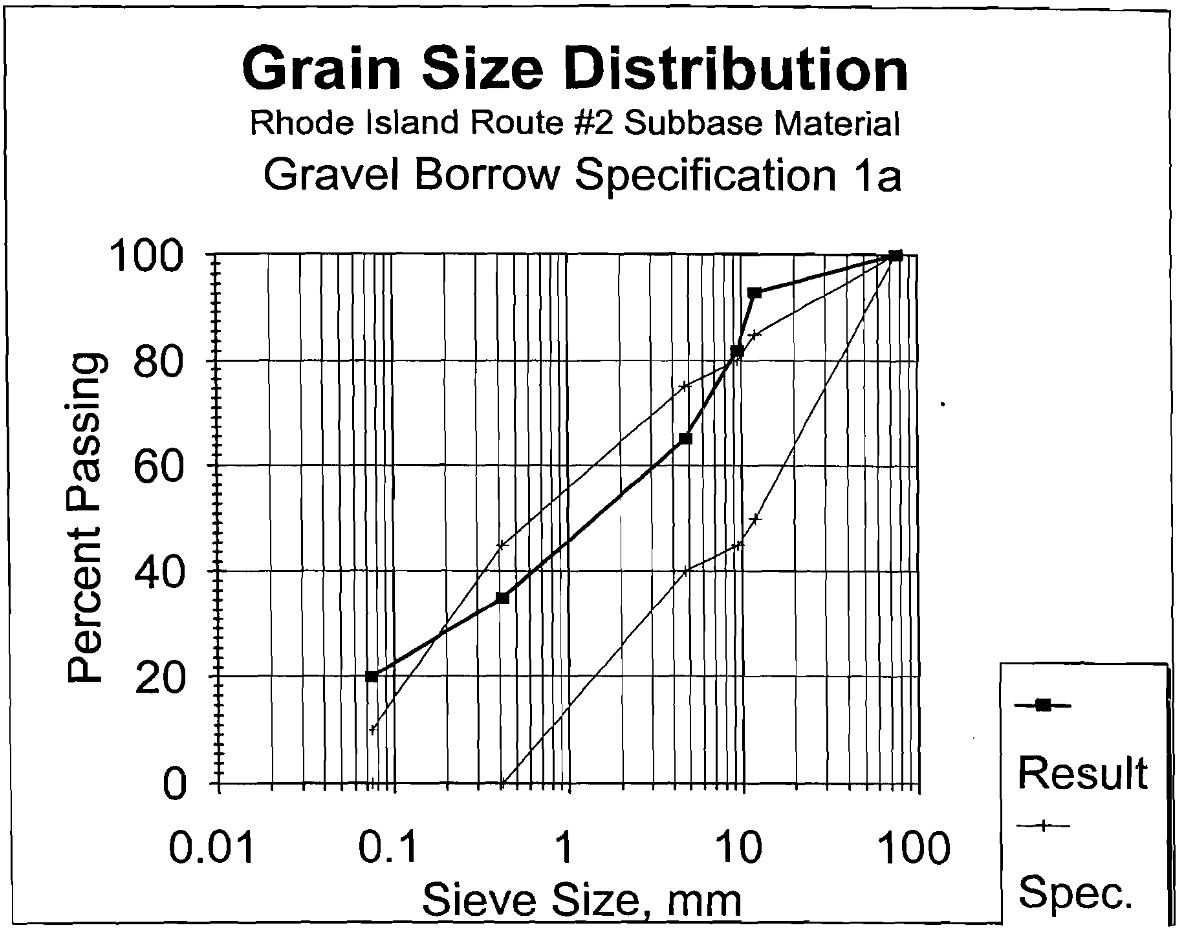


FIGURE 4.8 Rhode Island Rt. 2 Grain Size Distribution

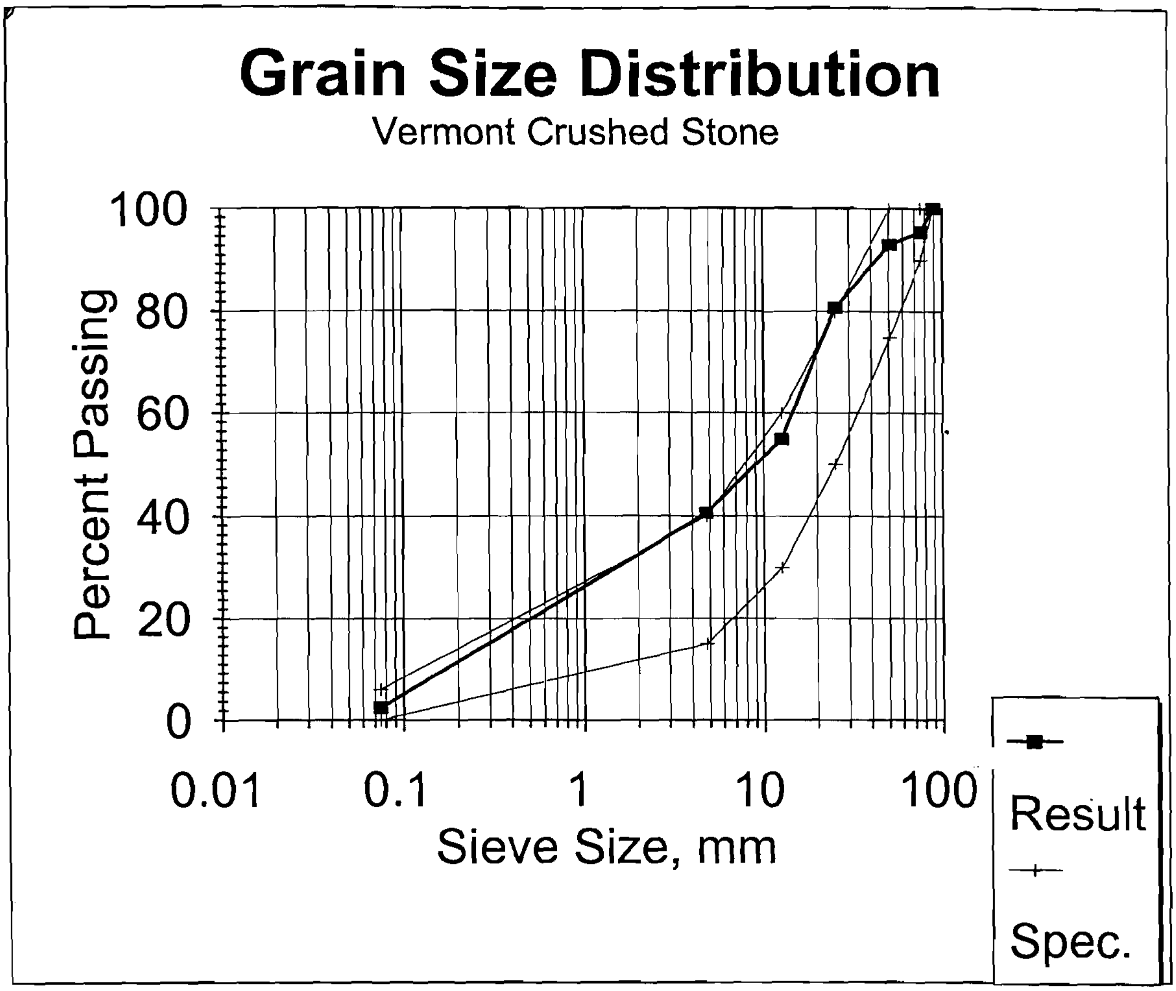


Figure 4.9 Vermont Crushed Stone Grain Size Distribution

PROCTOR TEST PLOT

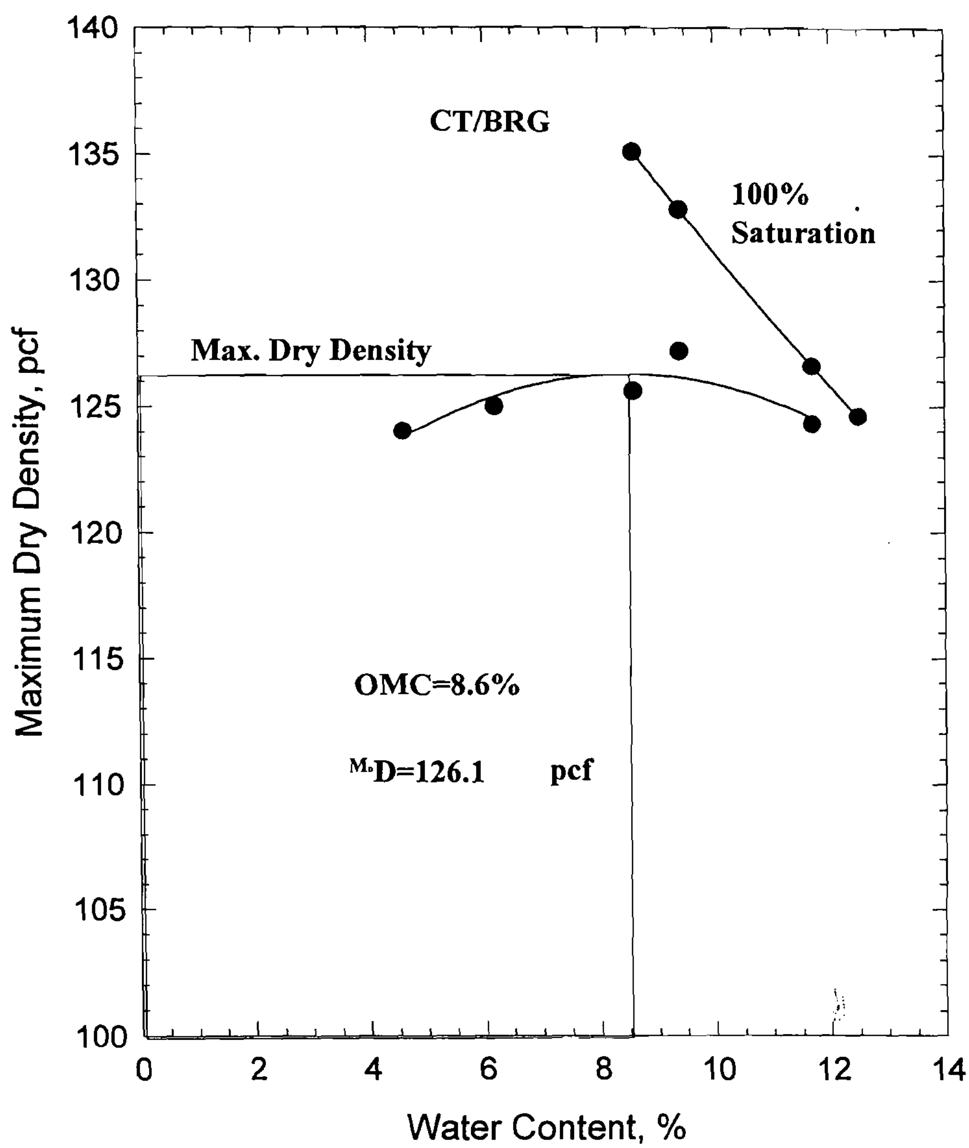


Figure 4.10 Connecticut Bank Run Gravel Proctor Plot

PROCTOR TEST PLOT

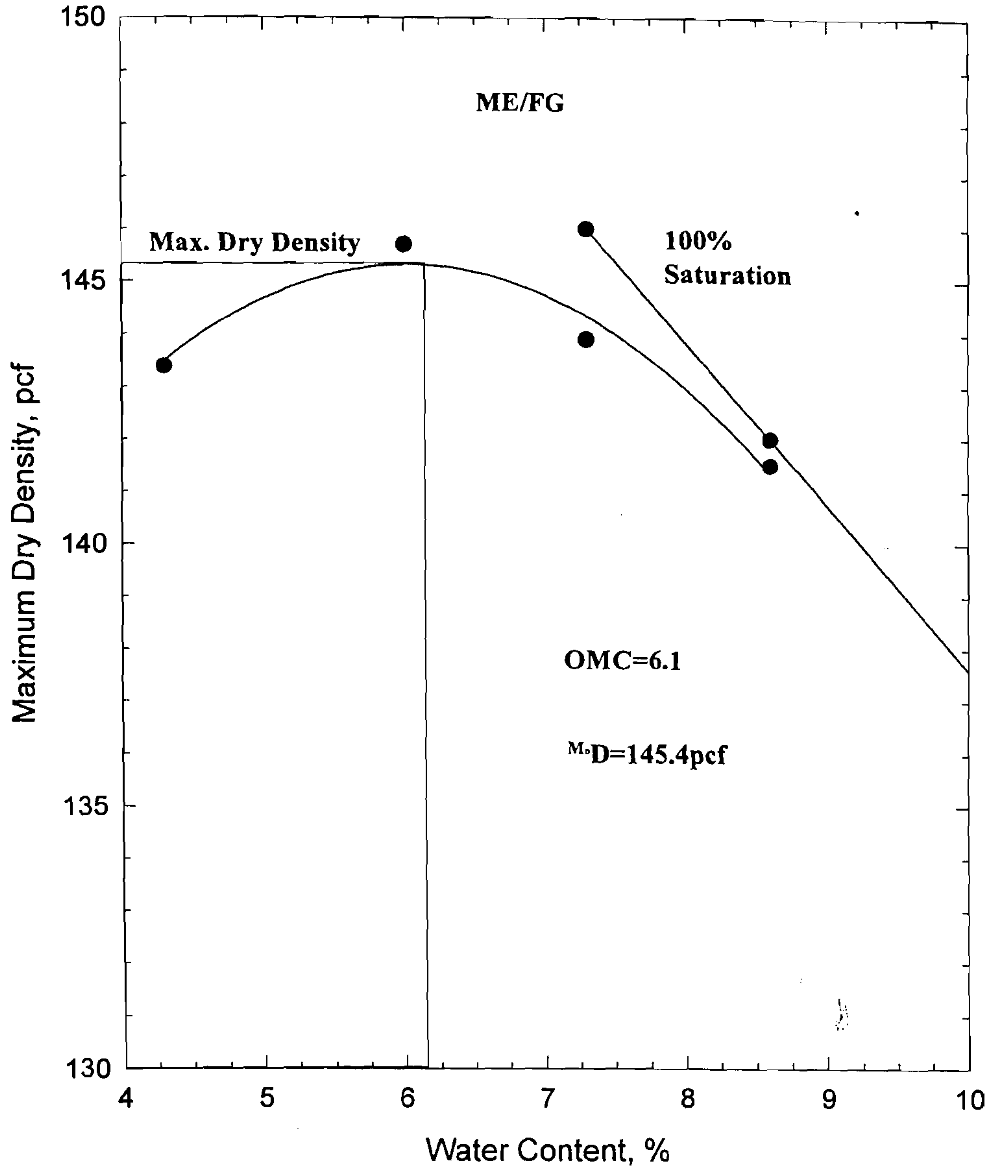


Figure 4.11 Maine Frenchville Gravel Proctor Plot

PROCTOR TEST PLOT

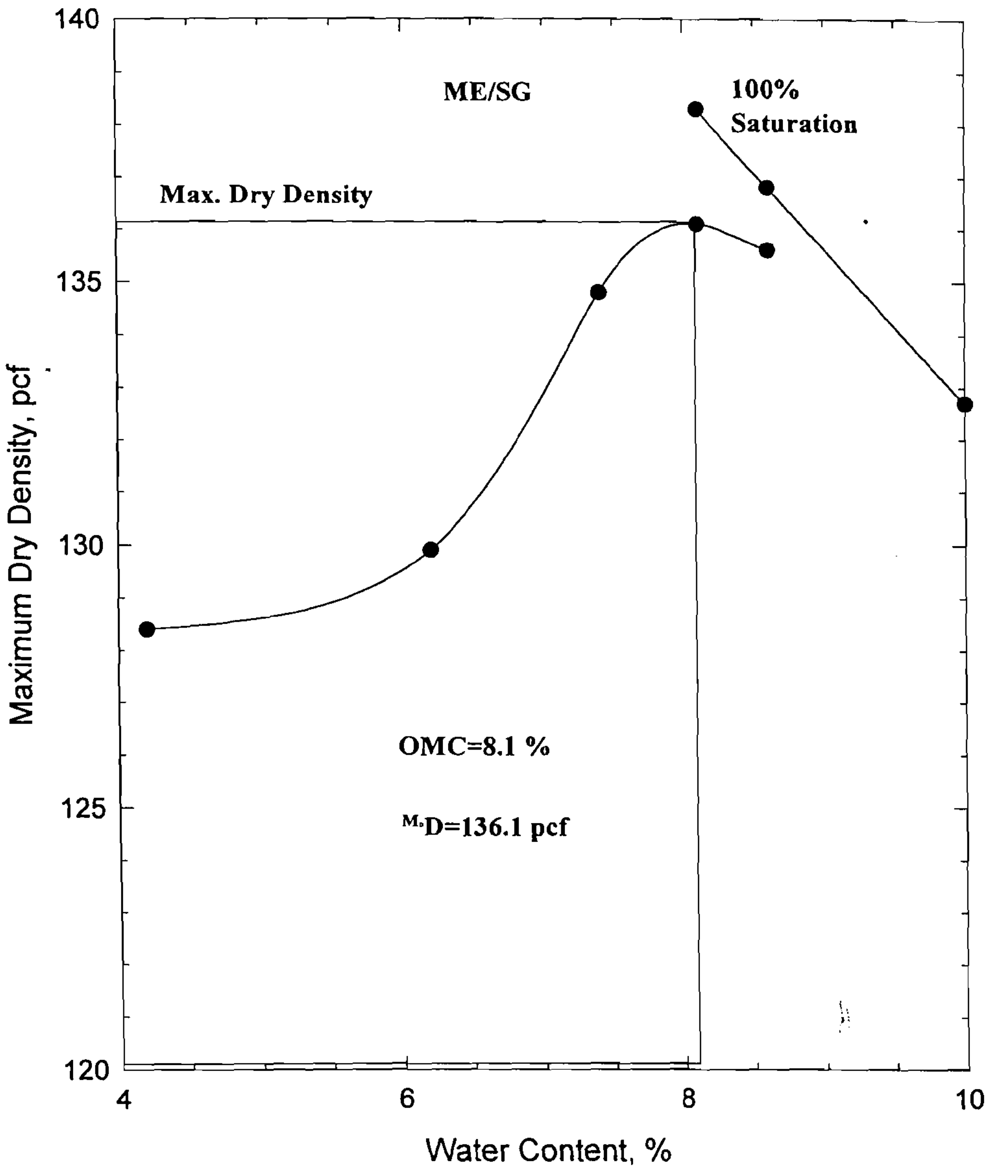


Figure 4.12 Maine Sabbatus Gravel Proctor Plot

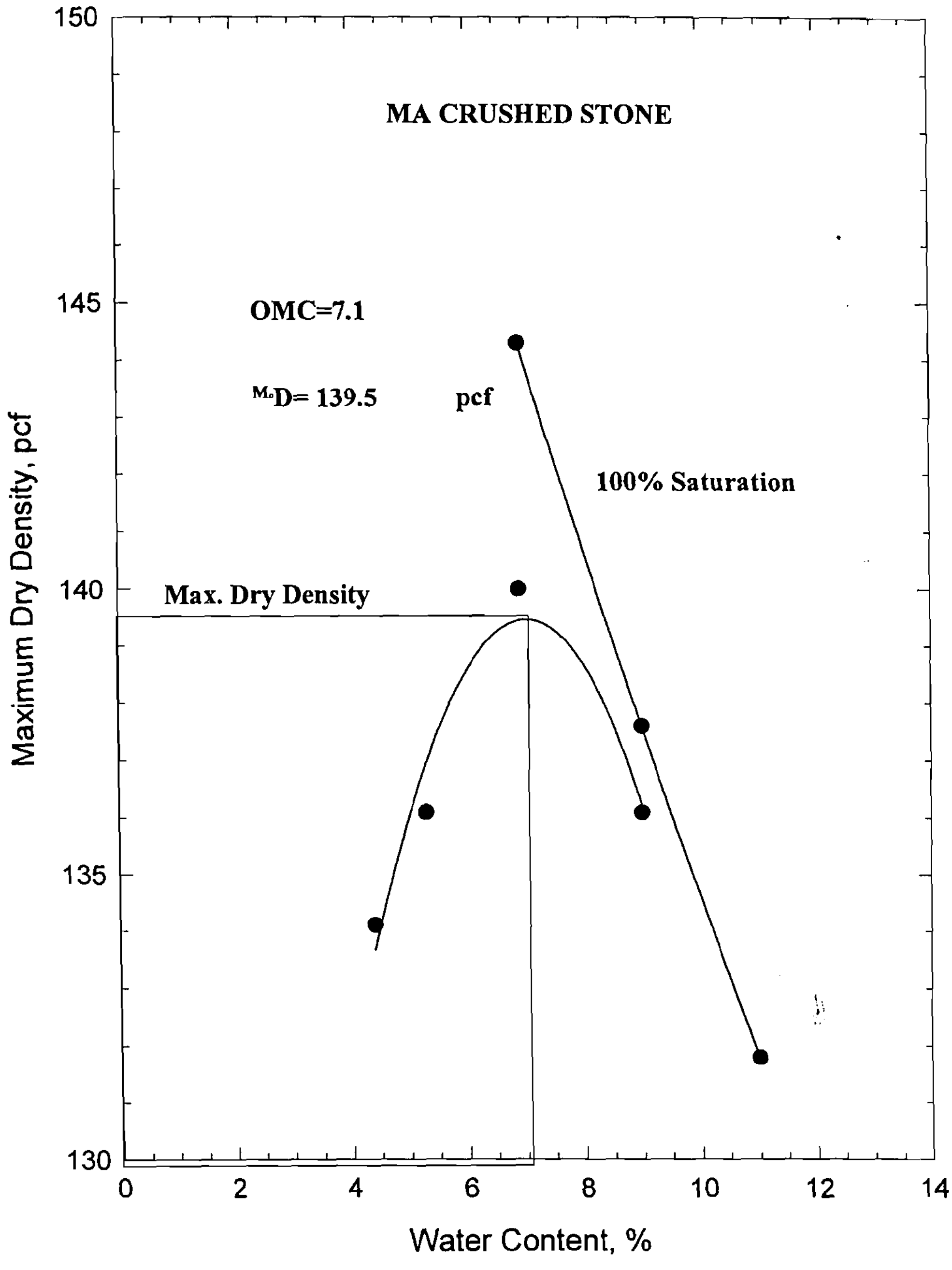


Figure 4.13 Massachusetts Crushed Stone Proctor Plot

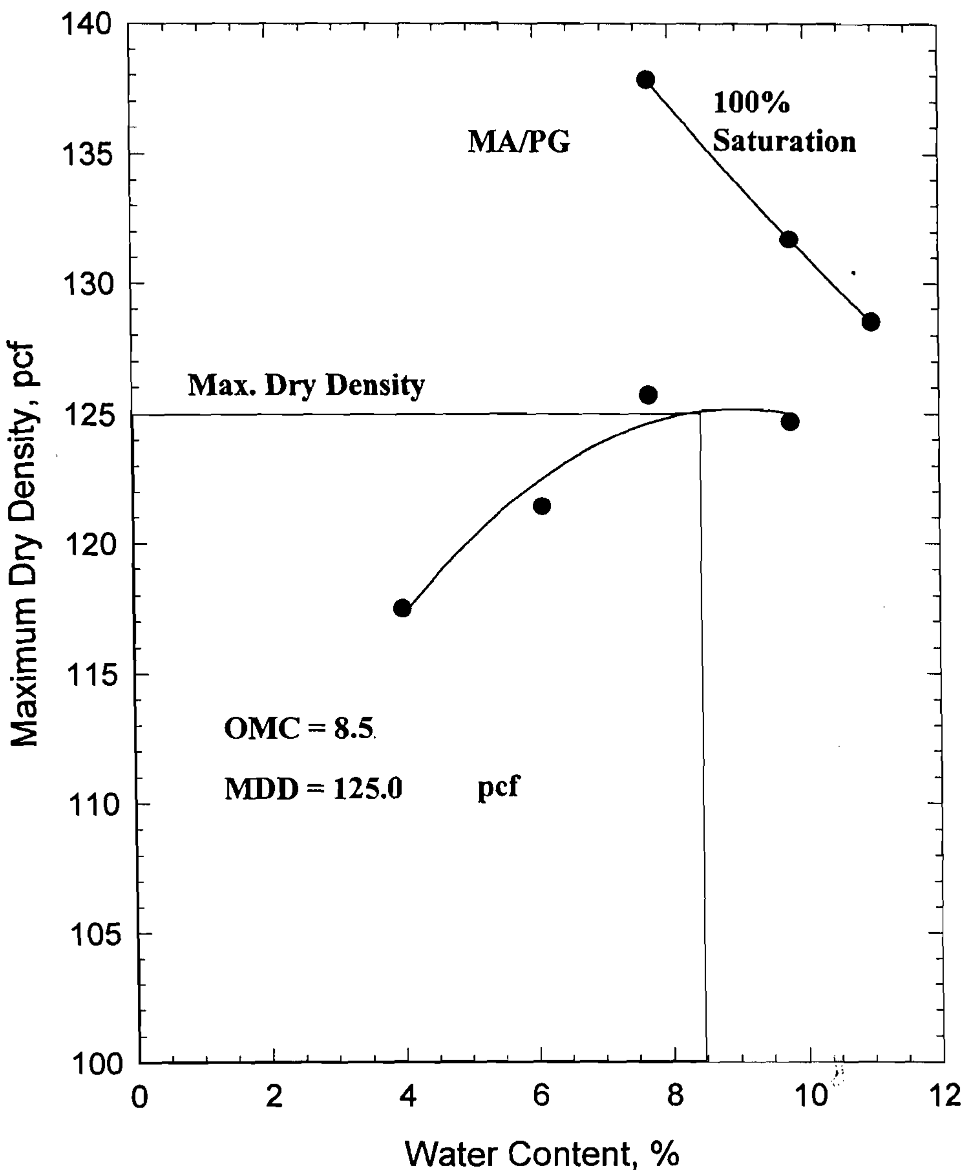


Figure 4.14 Massachusetts Processed Gravel Proctor Plot

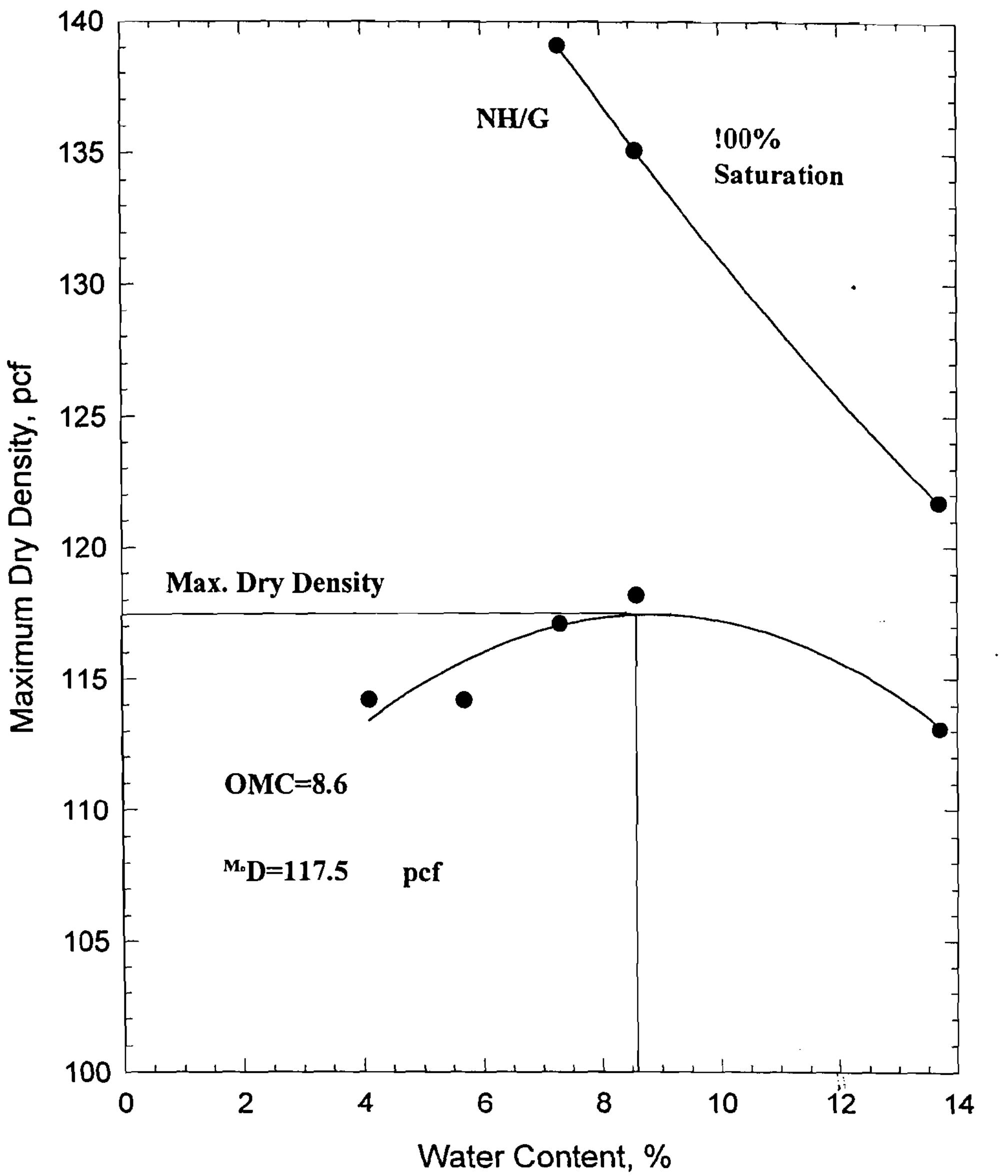


Figure 4.15 New Hampshire Sandy Gravel Proctor Plot

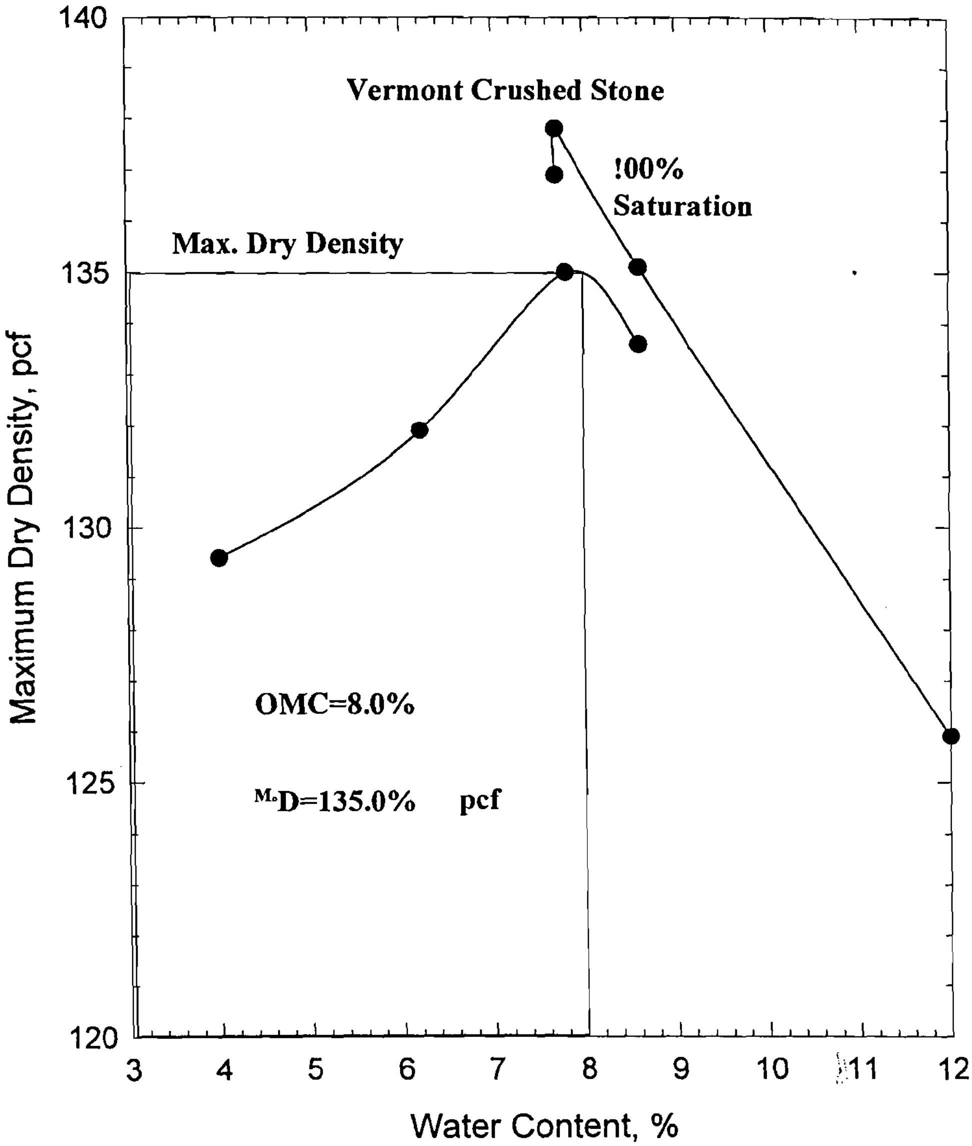


Figure 4.16 Vermont Crushed Stone Proctor Plot

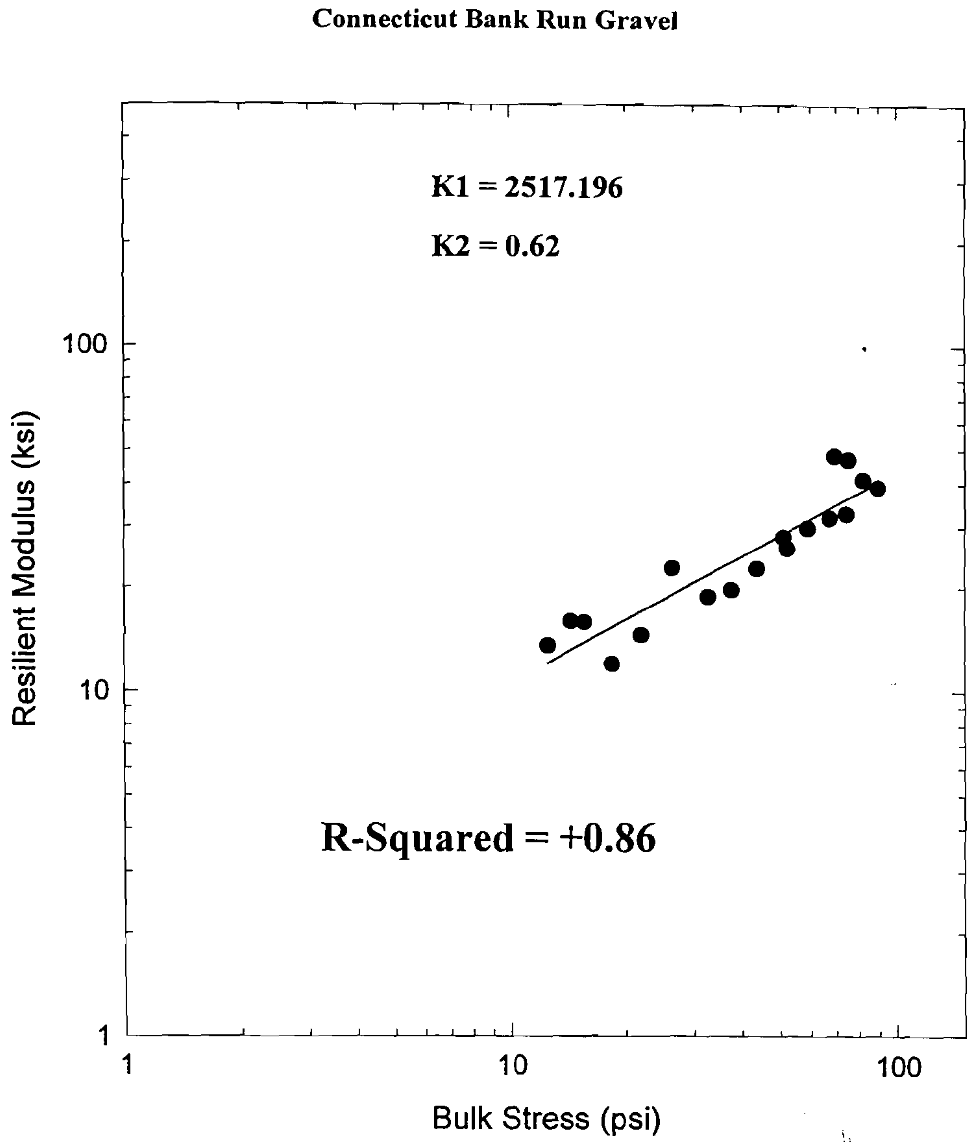


Figure 4.17 Results of Resilient Modulus Testing for Connecticut Bank Run Gravel

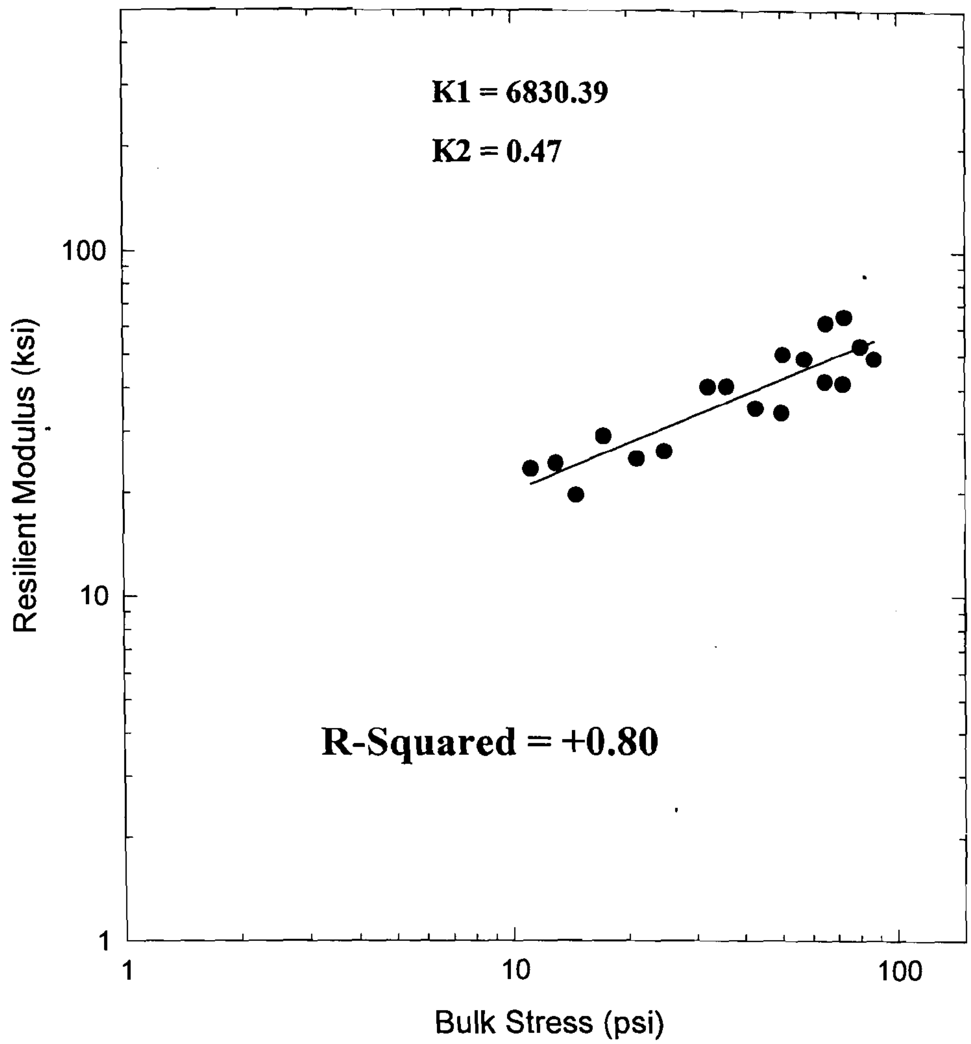


Figure 4.18 Results of Resilient Modulus Testing for Maine Frenchville Subbase

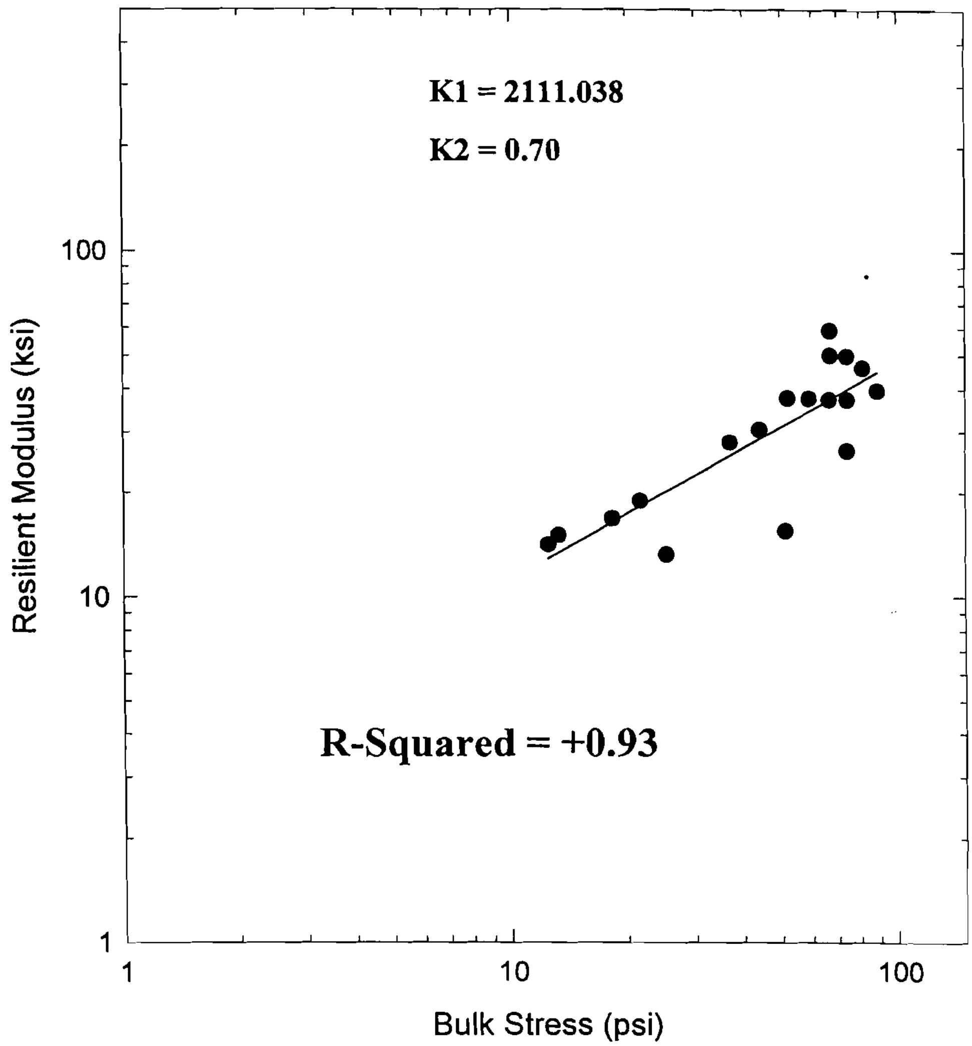


Figure 4.19 Results of Resilient Modulus Testing for Maine Sabbatus Subbase

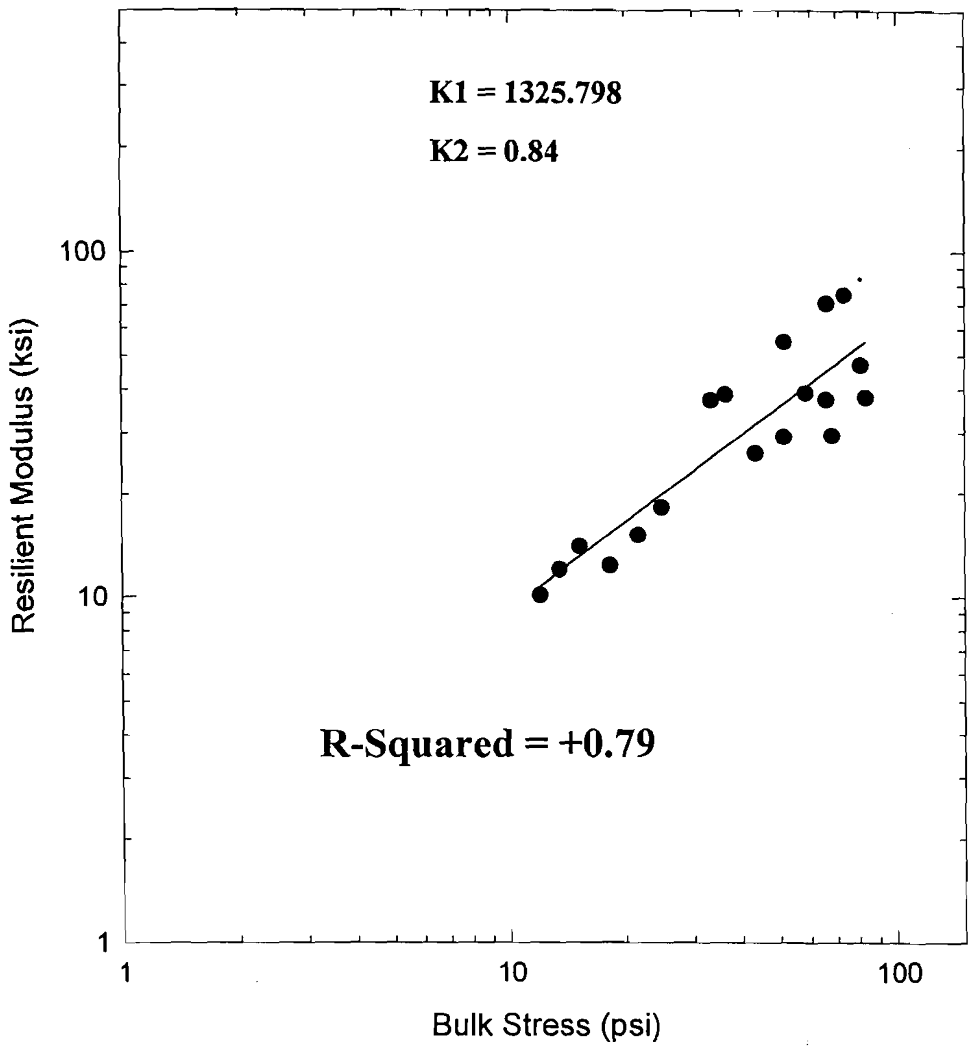


Figure 4.20 Results of Resilient Modulus Testing for Massachusetts Crushed Stone

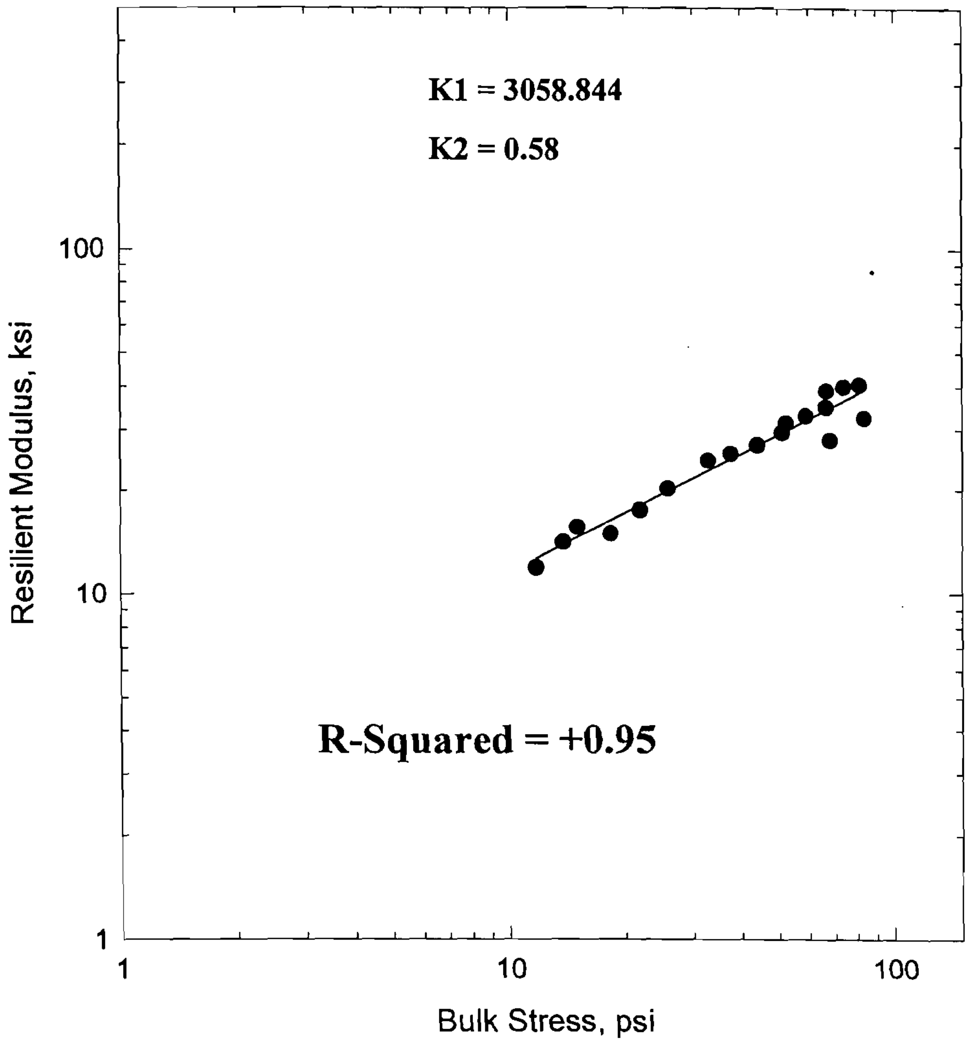


Figure 4.21 Results of Resilient Modulus Testing for Massachusetts Processed Gravel

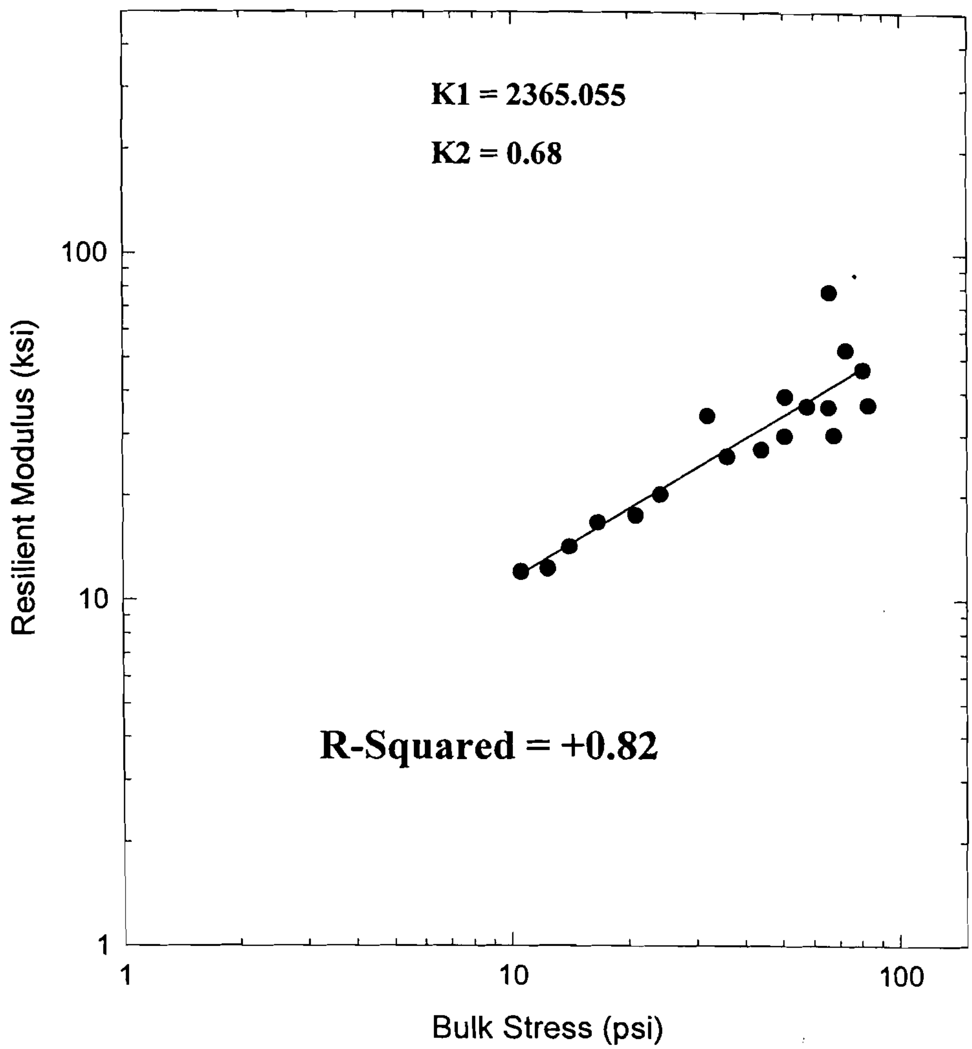


Figure 4.22 Results of Resilient Modulus Testing for New Hampshire Sandy Gravel

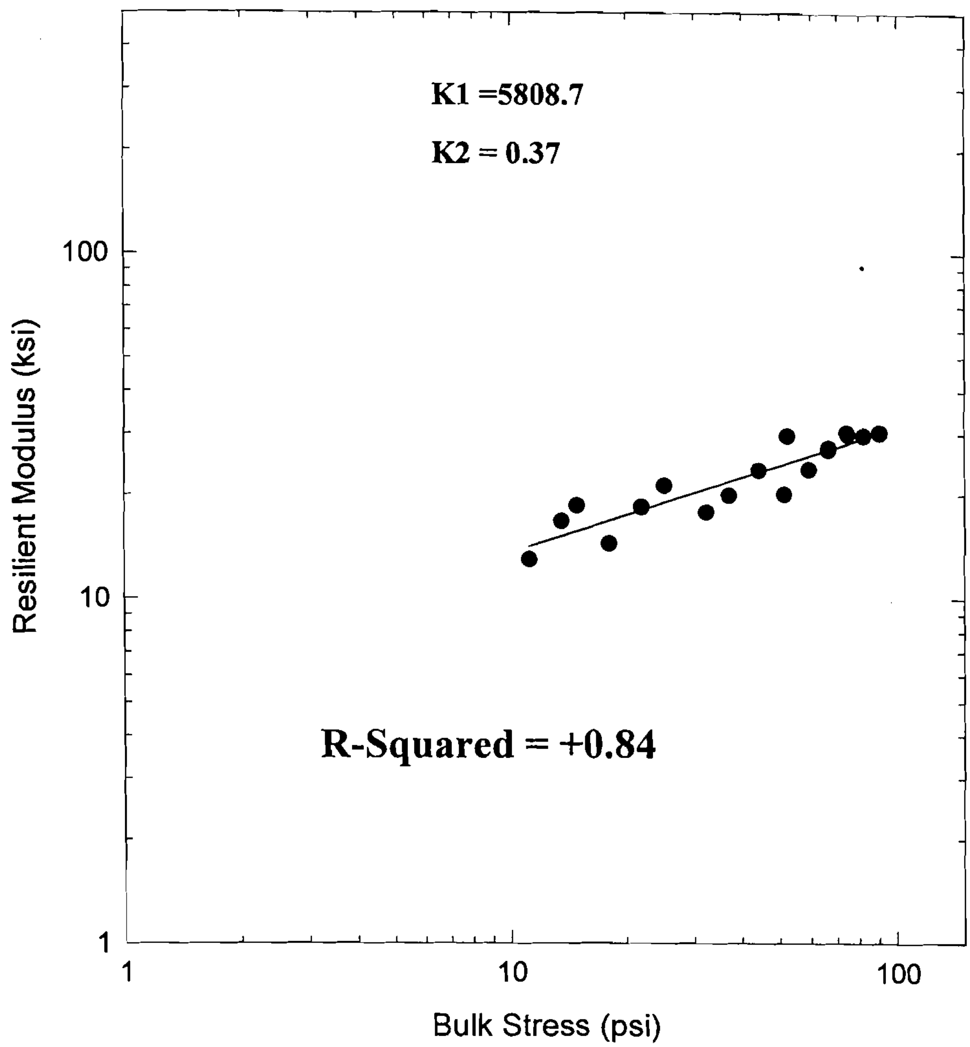


Figure 4.23 Results of Resilient Modulus Testing for Rhode Island Subbase (Route 2)

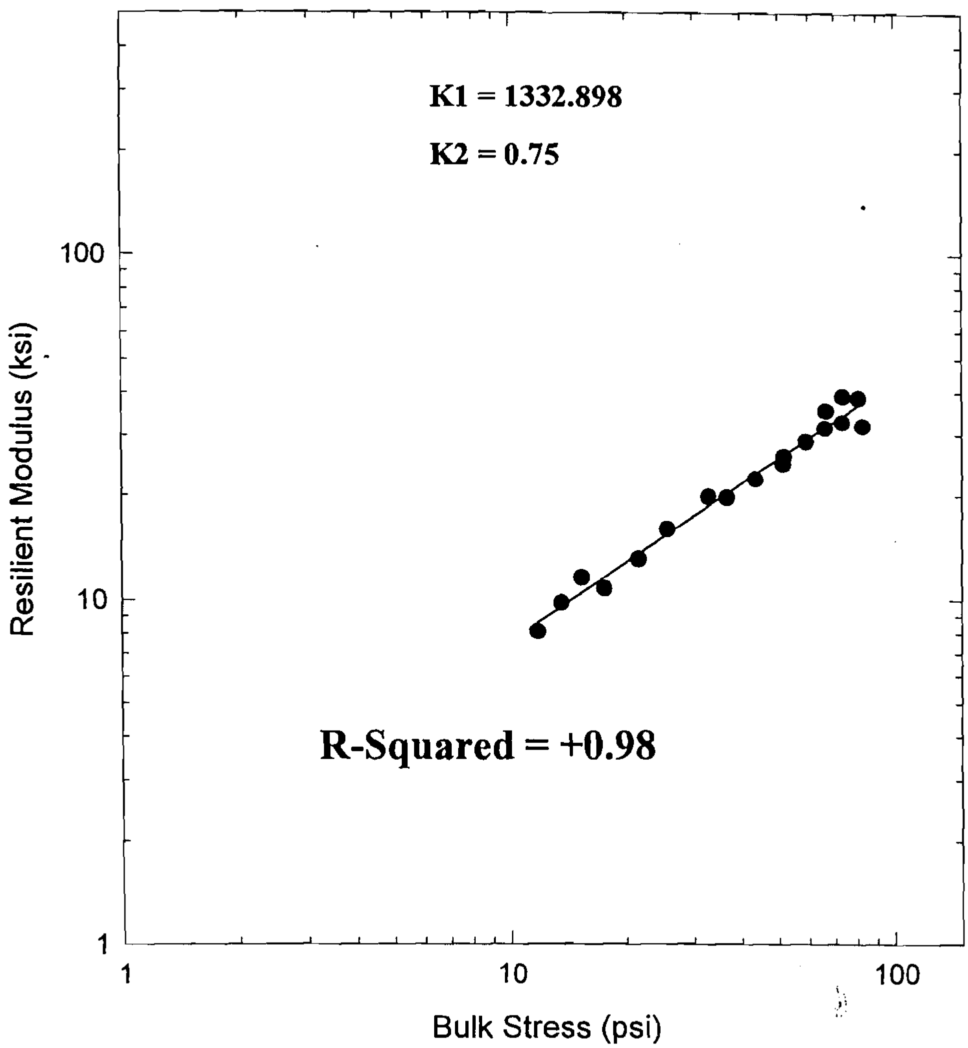


Figure 4.24 Results of Resilient Modulus Testing for Vermont Crushed Stone

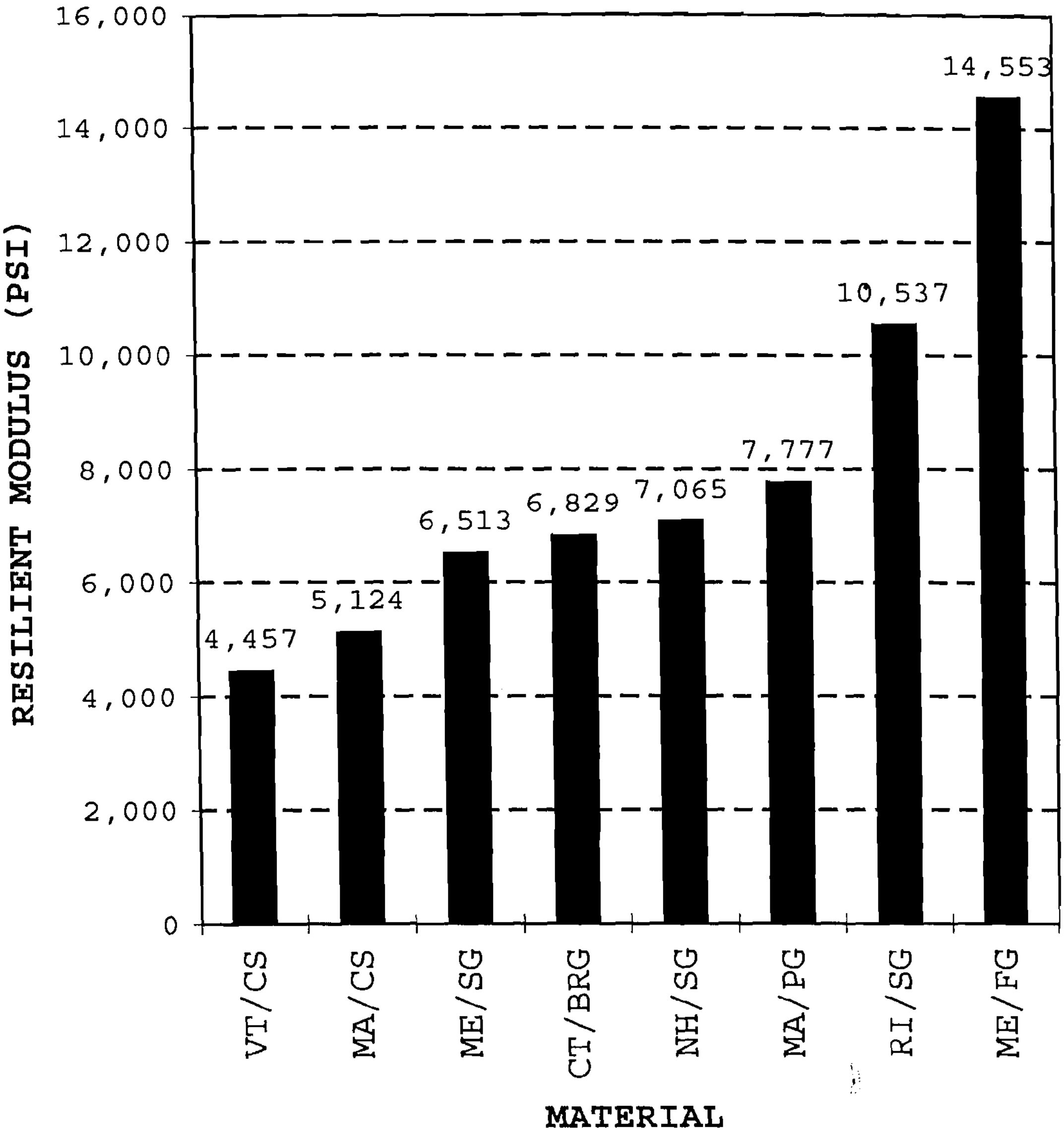


Figure 4.25 Comparison of E_{SB} with 5 psi Bulk Stress Assumed

URI RESILIENT MODULUS TEST
DATA SHEET UG-2

REMOLDING UNBOUND GRANULAR BASE/SUBBASE SAMPLES USING MOISTURE/DENSITY *(PROCTOR)* TEST RESULTS

(SUPPLEMENTAL DATA SHEET FOR AASHTO PROV. STND. TP46-94)

Project Name: ______________
Project I.D.: ______________
Material Source: ______________
Material Description: ______________
Layer Material *(circle one)* : Subgr/Subbase/Base

AASHTO Standard: TP46-94
SHRP Protocol P46
Prepared By: ______________ Date: ______
Reviewed By: ______________ Date: ______
Sampling Date: ______________

Worksheet

A.	Maximum Dry Density *(from Moisture/Density Report)*	A= ______	pcf
B.	Optimum Moisture Content *(from Moisture/Density Report)*	B= ______	%
C.	Target Dry Density of Compacted Specimen, *[0.95*A]*	C= ______	pcf
D.	Target Wet Density of Compacted Specimen, *[C(1+B/100)]*	D= ______	pcf
E.	Air dry Weight of Total Sample *(15hrs. min. at 140F)*	E= ______	grams
F.	Amount of Water Required to Achieve Optimum Moisture, *[E*((B-g)/100)]*	F= ______	ml
G.	Initial Weight of Container +Wet Soil	G= ______	grams
H.	Inside Diameter of Mold *(minus 2*membrane thickness)*	H= ______	in.
I.	Target Volume of Compacted Specimen, *{[(H/12)^2*pi/4]*12/12}*	I= ______	cu.ft.
J.	Volume of Each Compacted Layer, *[I/6]*	J= ______	cu.ft.
K.	Wet Weight *(portion of G)* Required per Layer, *[(D*J)*454]*	K= ______	grams
L.	Final Weight of Container + Wet Soil	L= ______	grams
M.	Wet Weight of Soil Used, *[G-L]*	M= ______	grams

Moisture Contents

		Air dry	Compacted	
a.	Tare No.:	______	______	
b.	Tare Weight:	______	______	grams
c.	Wet Wt. + Tare *:	______	______	grams
d.	Dry Wt. + Tare:	______	______	grams
e.	Weight of Water (c-d):	______	______	grams
f.	Dry Weight (d-b):	______	______	grams
g.	Moisture Content (e/f *100):	______	______	%

* *Use approximately 500 grams from material in L above for compacted* .

After Test Moisture Content

h.	Tare No.:	______	
i.	Tare Weight:	______	grams
j.	Wet Wt. + Tare:	______	grams
k.	Dry Wt. + Tare:	______	grams
l.	Weight of Water (j-k):	______	grams
m.	Dry Weight (k-i):	______	grams
n.	Moisture Content (l/m*100):	______	%

Actual Sample Dimensions

o. Height	h1=	______	in.
	h2=	______	in.
	h3=	______	in.
	havg=	______	in.
p. Diameter	d1=	______	in.
	d2=	______	in.
with rubber membrane	d3=	______	in.
	davg=	______	in.

Membr. thick 1 = ______ Membr. thick 2 = ______

q. Volume V = ______ cu.ft.
V = [(havg*pi*davg^2)/4]/1728

r. Wet Density = ______ pcf
WD = (M/454)/V

s. Dry Density = ______ pcf
DD = [WD/(1+g/100)]

filename: type1dat.xls

Form UG-2, 9/23/97

Figure 4.26 URI Resilient Modulus Test Data Sheet UG_2

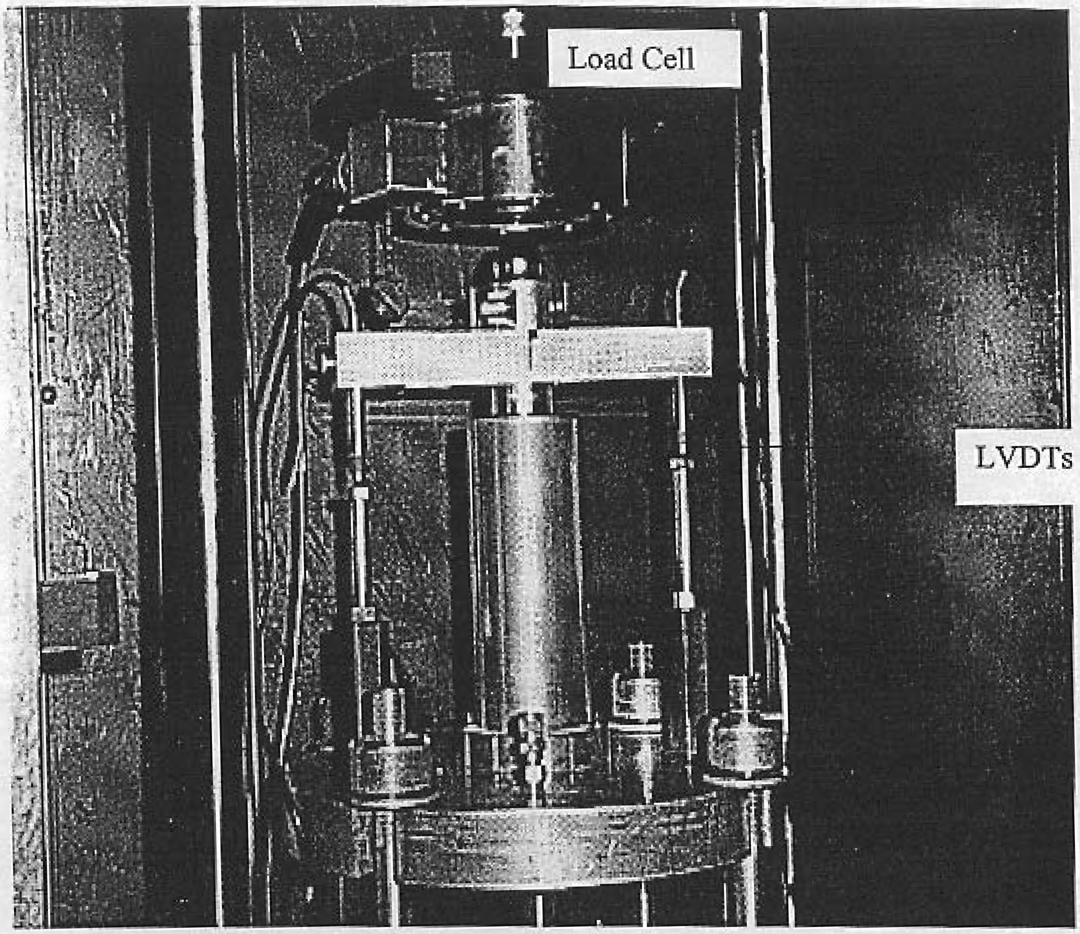


Figure 4.27 Location of LVDTs according to TP46 procedure

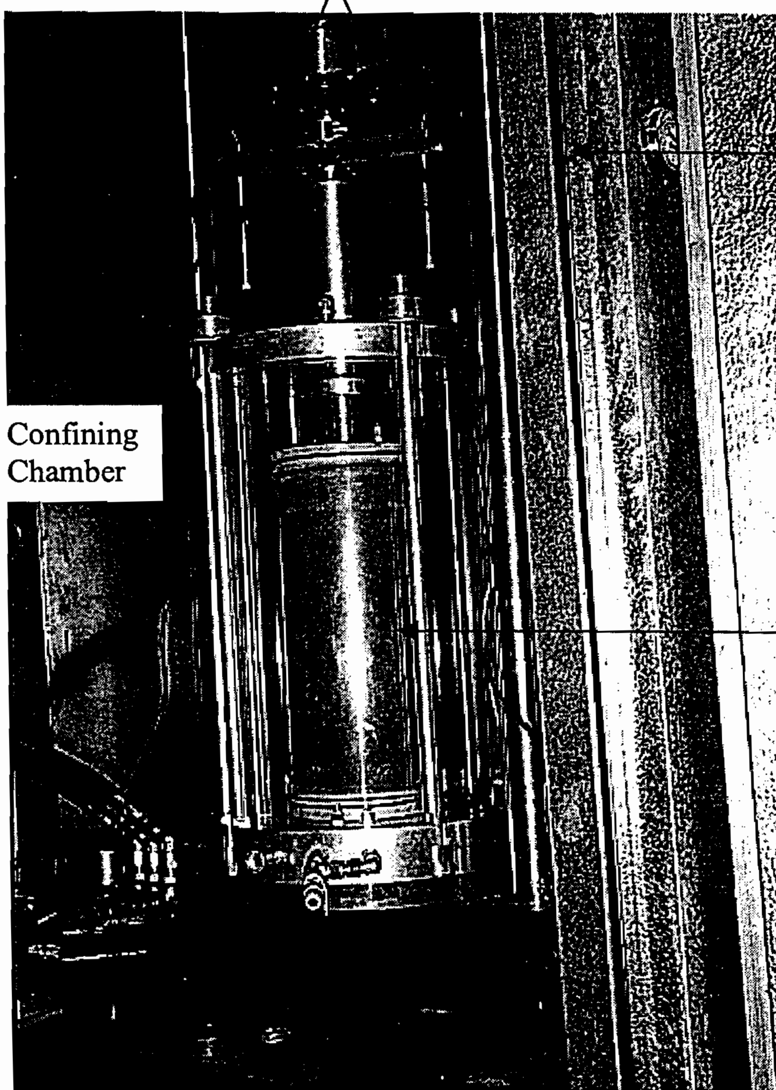


Figure 4.28 Assembled Equipment of TP46

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Connecticut
2. MATERIAL DESCRIPTION: Bank Run Gravel
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 10-23-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 =	9,112
K2 =	0.14308
K5 =	0.33267
R^2 =	0.83

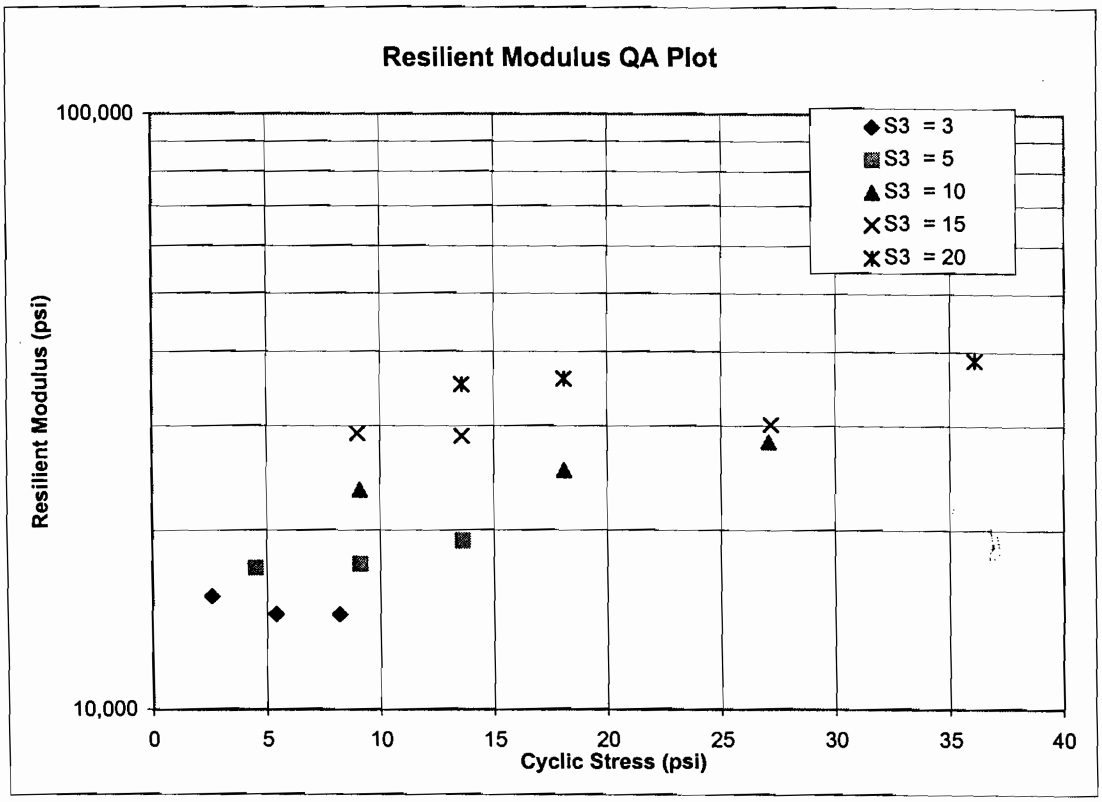


FIGURE 4.29 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC

PROJECT ID: 94-1

1. MATERIAL SOURCE: Maine
2. MATERIAL DESCRIPTION: Frenchville Gravel
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 10-27-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 9,131

K2 = 0.12919

K5 = 0.38923

R^2 = 0.83

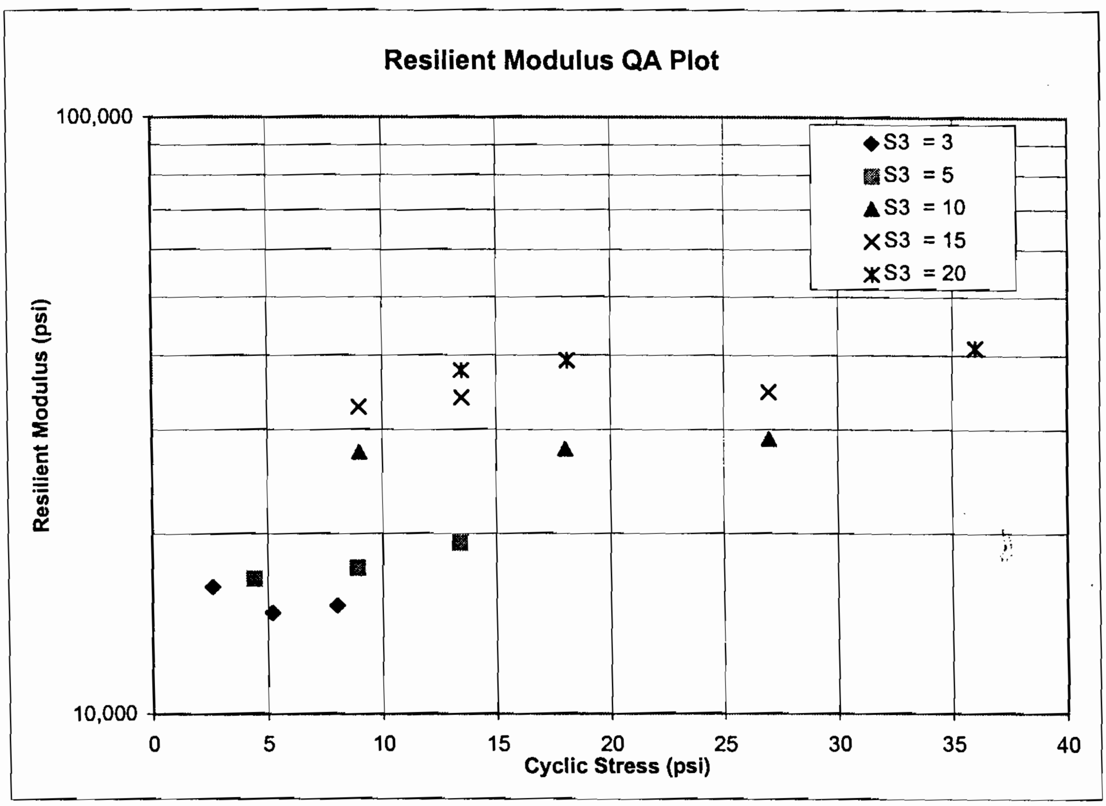


FIGURE 4-30 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE: Maine
2. MATERIAL DESCRIPTION: Subbatus Subbase Gravel
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 10-29-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 7,412
K2 = 0.18762
K5 = 0.40123
R^2 = 0.83

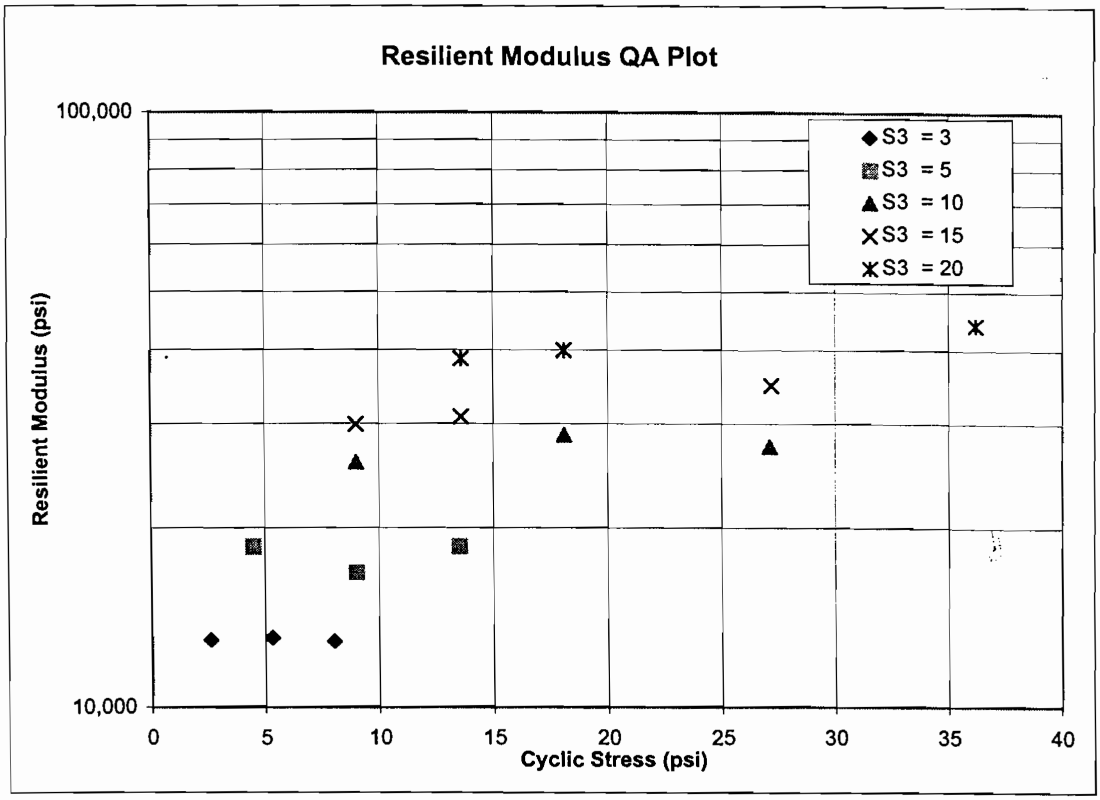


FIGURE 4.31 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Massachusetts
2. MATERIAL DESCRIPTION: Crushed Stone
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 10-29-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 6,630
K2 = 0.20832
K5 = 0.38239
R^2 = 0.85

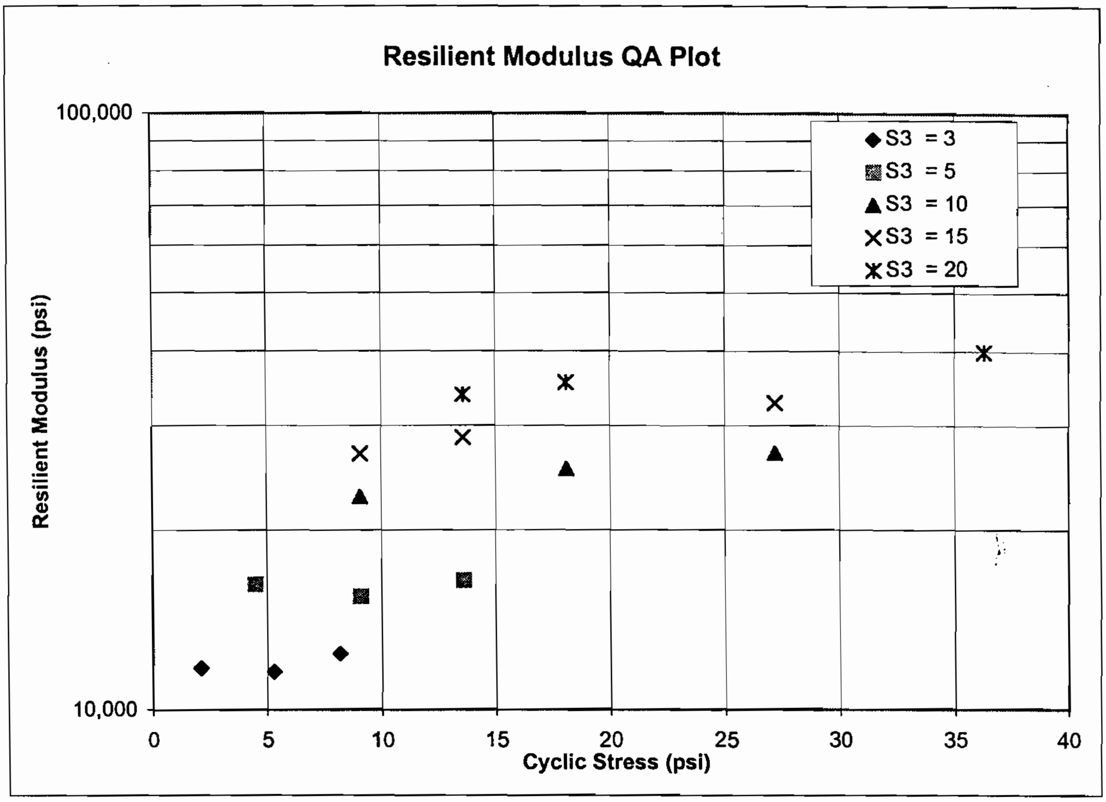


FIGURE 4.32 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE: Massachusetts
2. MATERIAL DESCRIPTION: Processed Gravel
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 09-30-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 8,520
K2 = 0.14347
K5 = 0.37425
R^2 = 0.83

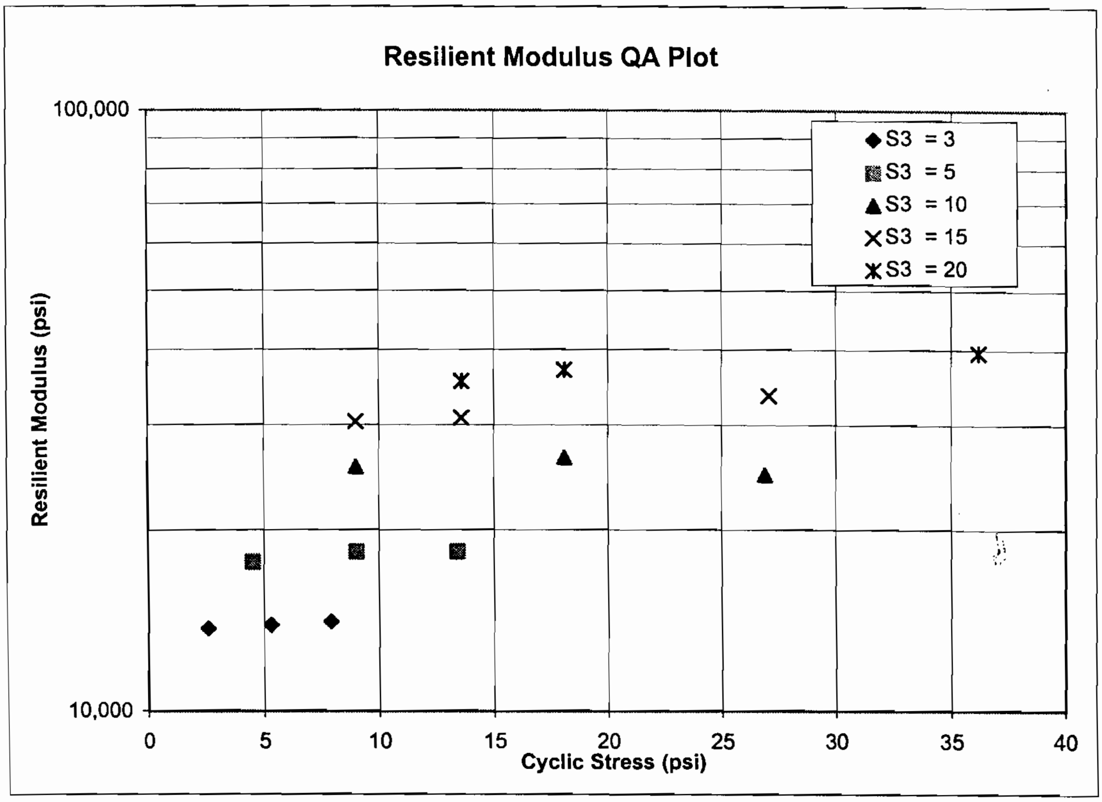


FIGURE 4.33 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE New Hampshire
2. MATERIAL DESCRIPTION: Sandy Gravel
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 11-19-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 11,673
K2 = -0.01140
K5 = 0.36883
R^2 = 0.76

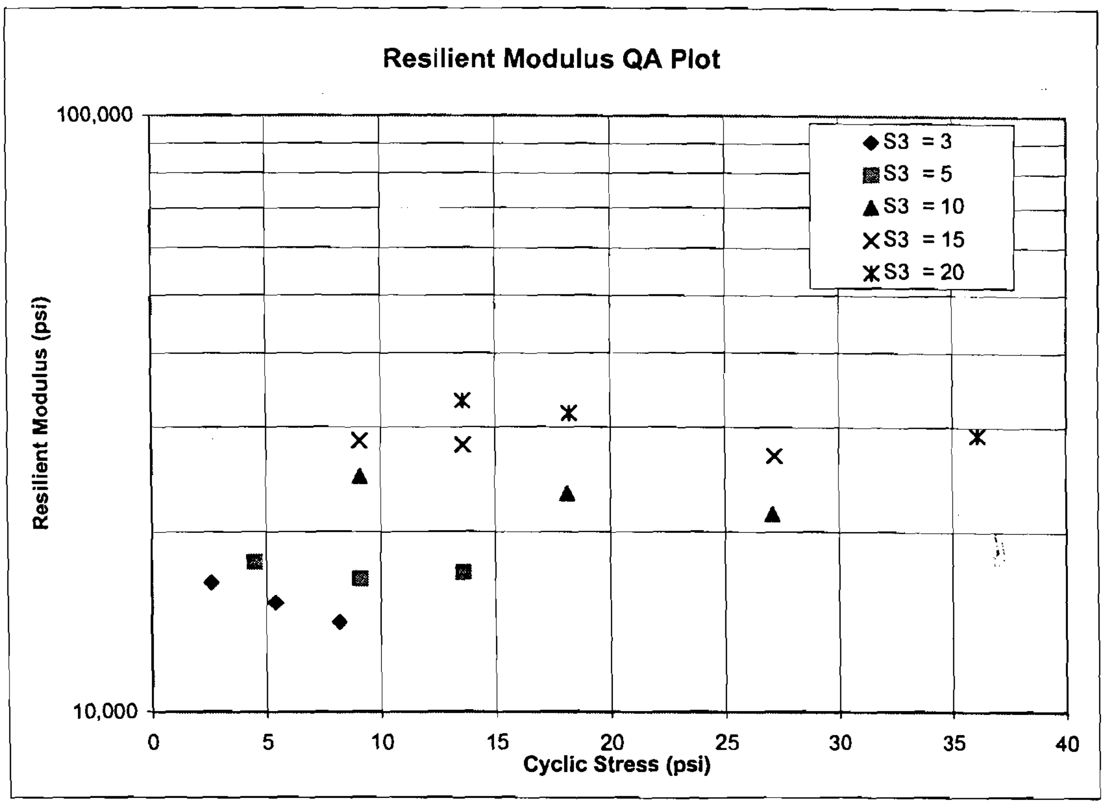


FIGURE 4.34 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE: Rhode Island
2. MATERIAL DESCRIPTION: Sandy Gravel
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 10-25-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 10,201
K2 = 0.11055
K5 = 0.35028
R^2 = 0.77

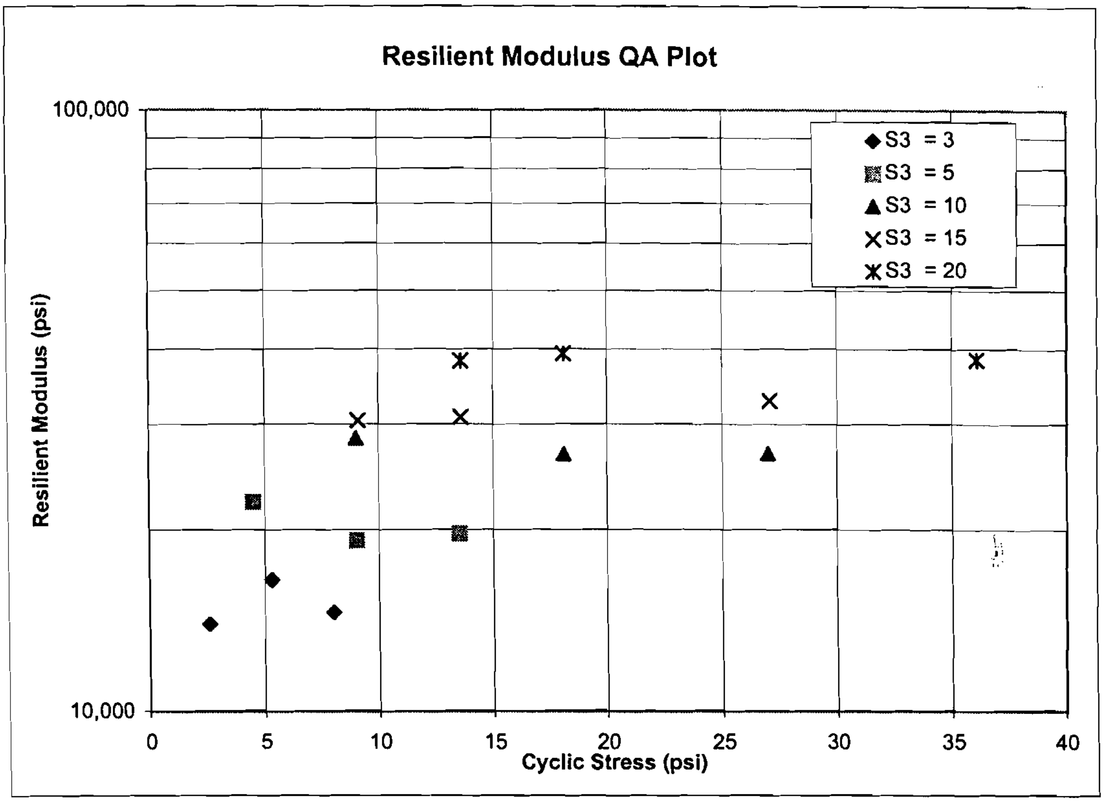


FIGURE 4.35 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Vermont
2. MATERIAL DESCRIPTION: Crushed Stone
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: I
5. TEST DATE: 10-28-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 =	7,525
K2 =	0.16153
K5 =	0.45062
R^2 =	0.84

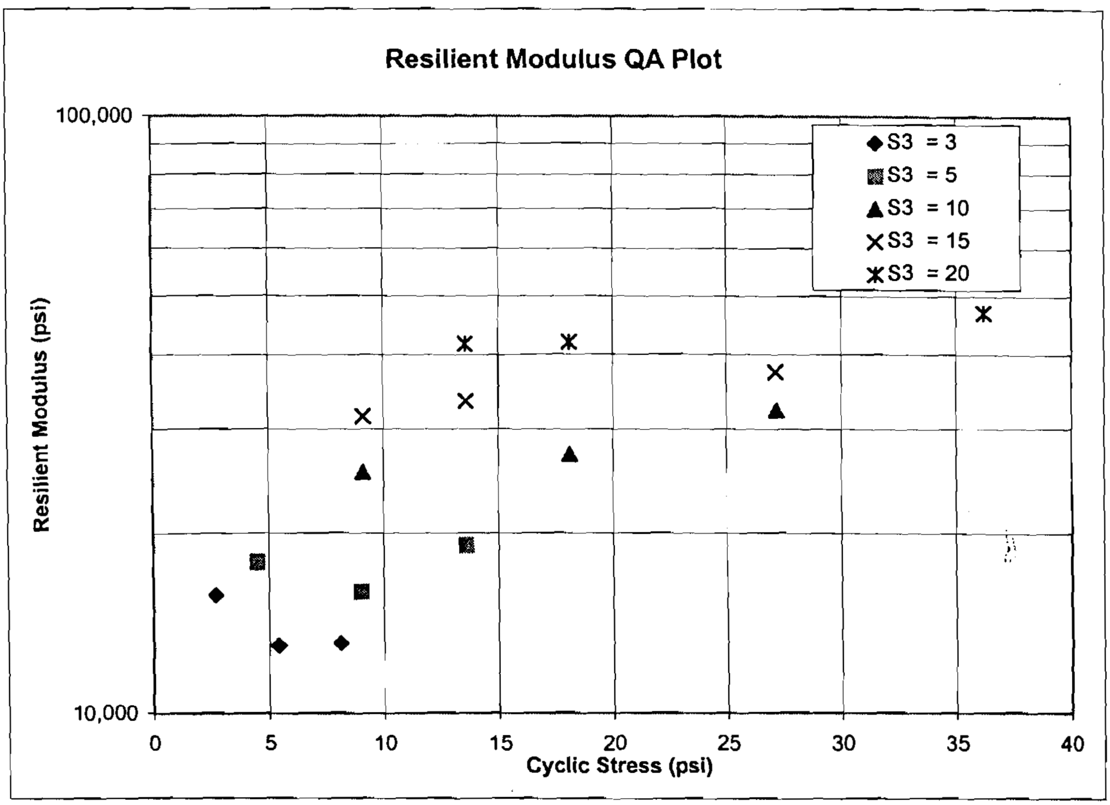


FIGURE 4.36 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

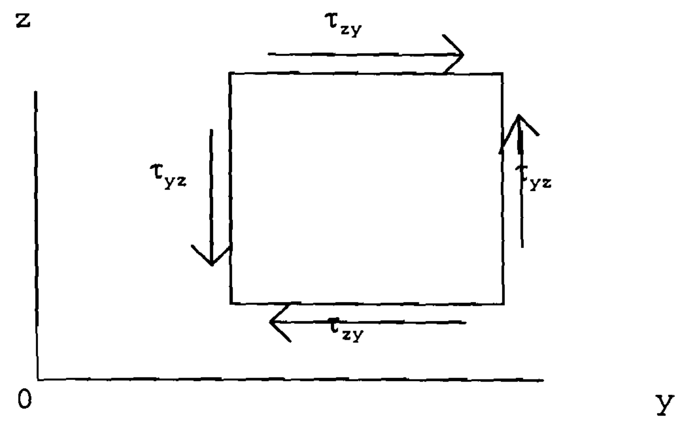


Figure 4.37 Shear Stresses on an Element

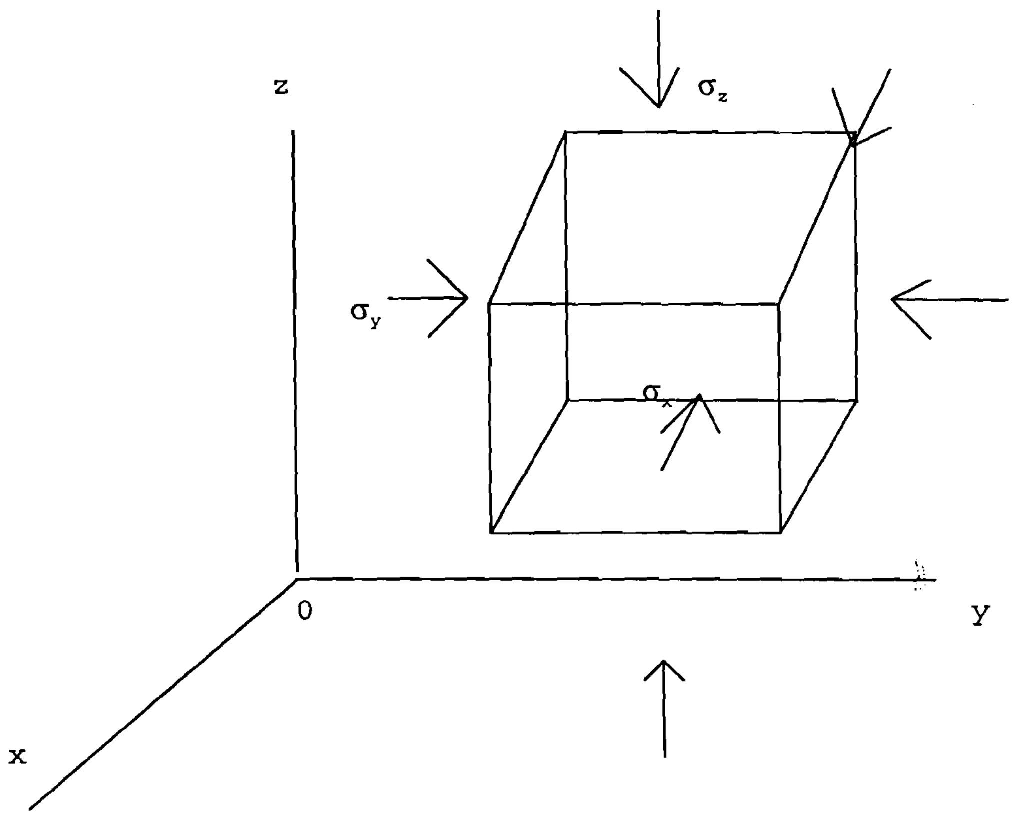


Figure 4.38 Principle Stresses on an Element

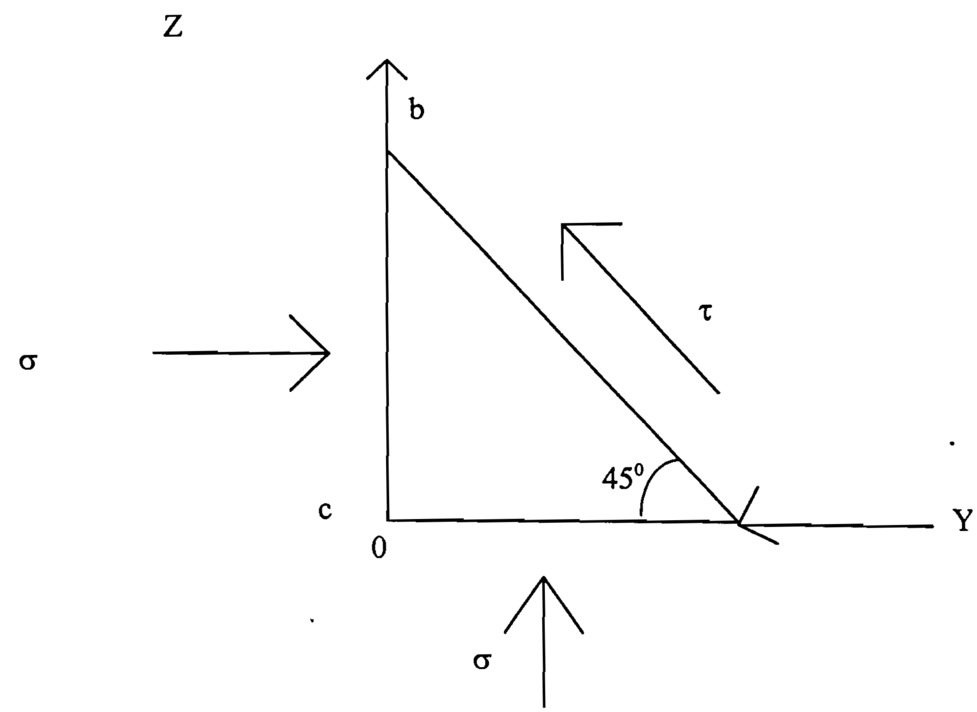
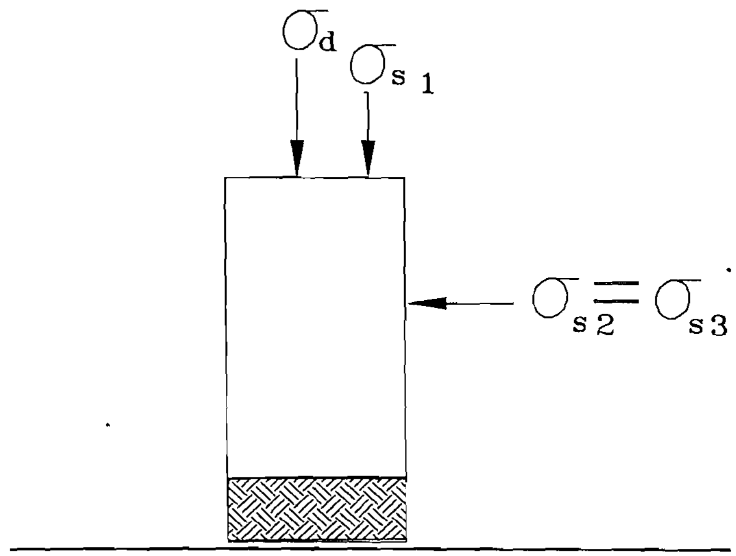


Figure 4.39 Pure Shear

Principle Stresses on a Specimen
in a Triaxial Chamber



Principal Stresses on a Subbase Element

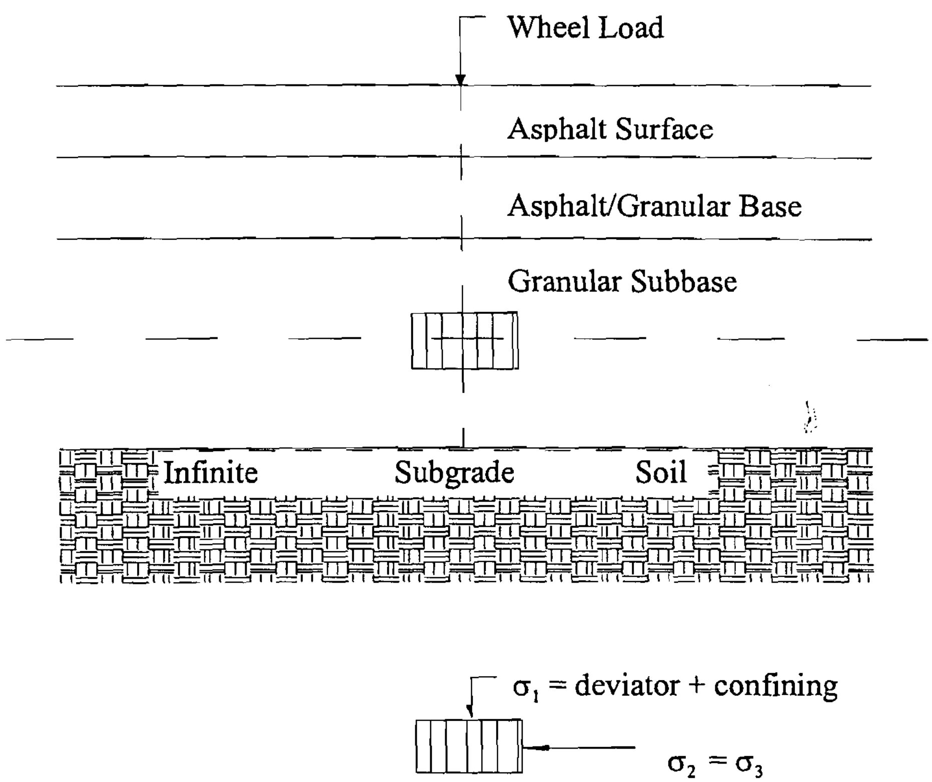


Figure 4.40 Experimental Bulk Stress and Field Condition

```
RI ELASTIC SYSTEM 22,400LB AXLE LOAD

          ELASTIC    POISSON'S
LAYER     MODULUS    RATIO        THICKNESS
  1       325000.      .350          1.500 IN
  2       540000.      .350          1.500 IN
  3       480000.      .350          5.000 IN
  4        20300.      .400         12.000 IN
  5         6500.      .400       SEMI-INFINITE

  TWO LOAD(S), EACH LOAD AS FOLLOWS

TOTAL LOAD.....     5600.00 LBS
LOAD STRESS....      100.00 PSI
LOAD RADIUS....        4.22 IN

       LOCATED AT
LOAD     X         Y
  1      .000      .000
  2    13.110      .000

RESULTS REQUESTED FOR SYSTEM LOCATION(S)
DEPTH(S)

Z=  14.00
X-Y POINT(S)
  X          Y
   6.56        .00

Z=  14.00 LAYER NO,  4
  X          Y
   6.56       .00

PRINCIPAL STRESSES
PS 1    .1746E+01 (tension)      σ3
PS 2    .1292E+01 (tension)      σ2
PS 3    .4952E+01 (compression)  σ1
```

TOTAL = 4.95 psi Compression

Figure 4.41 Deviator Stress Calculations by ELSYM5 for Rhode Island Structures

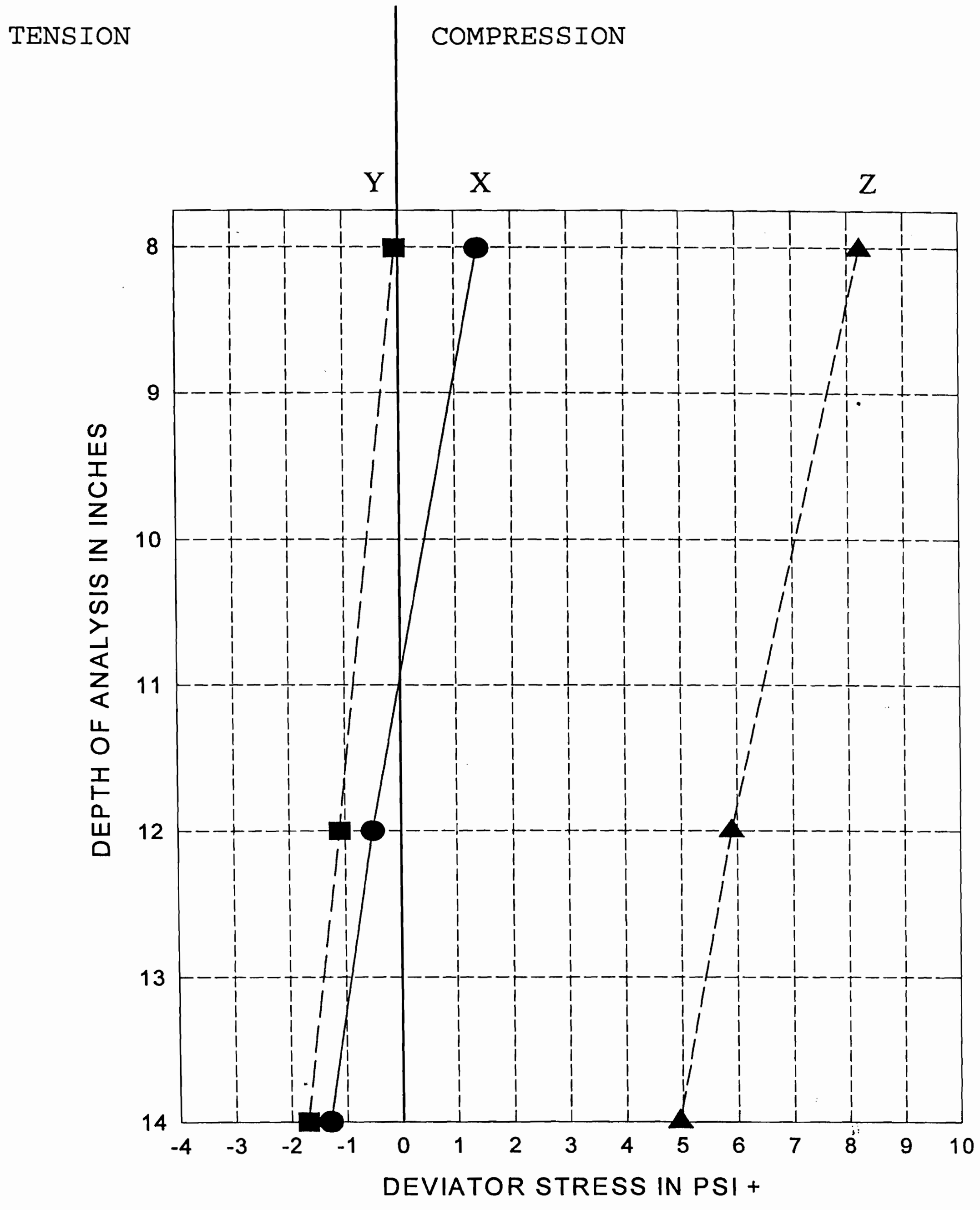


Figure 4.42 Deviator Stress in the X, Y and Z Planes as a Function of Depth for a Typical Pavement Cross Section on Rhode Island RT 2.

$\theta = \sigma_1 + \sigma_2 + \sigma_3$

$= (\sigma_{d1} + \sigma_{s1}) + \sigma_{d2} + \sigma_{s2} + \sigma_{d3} + \sigma_{s3}$

$= (\sigma_{d1} + \sigma_{s1}) + (\sigma_{d2} + \mathbf{k}_0\sigma_{s1}) + (\sigma_{d3} + \mathbf{k}_0\sigma_{s1})$

$= (\sigma_{d1} + \sigma_{d2} + \sigma_{d3}) + (\sigma_{s1} + K_0\sigma_{s1})$

where

$\mathbf{k}_0$ = coefficient of earth pressure at rest 1-sine ϕ for cohesionless soil and gravel

= 1-sine 40^0 = .3572

ϕ = angle of internal friction assumed @ 40^0 for subbase granular material (Lee, et al. 1994)

σ_{s1} = 8 in. asphalt + 6 in. subbase (stress calculation point)

= .67 ft.@(145pcf) + .5 ft.@(130pcf)

= (97.15+65)psf/144 in. = 1.12psi

$\sigma_{s1} + 2(k_0\sigma_{s1}) =$ 1.12 + 2(0.3572)(1.12) = 1.92psi

$\sigma_d = \sigma_{d1} + \sigma_{d2} + \sigma_{d3}$
(ELSYM5 analysis with 22.4Kip axle or (2) 5,600 lb. wheels)

= 4.95 psi (from Figure 4.41, tension = 0)

$\theta = \Sigma\sigma_d + \Sigma\sigma_s$ = 4.95 + 1.92 = 6.87psi

Sample Calculation of E_{sb}

$E_{sb} = 5808.7(\theta)^{0.37}$ (Lee et al. 1996) (AASHTO T292-91)

E_{sb} = 11,851 psi

Figure 4.43 Sample Calculation of Bulk Stress and Resilient Modulus (AASHTO T292-91)

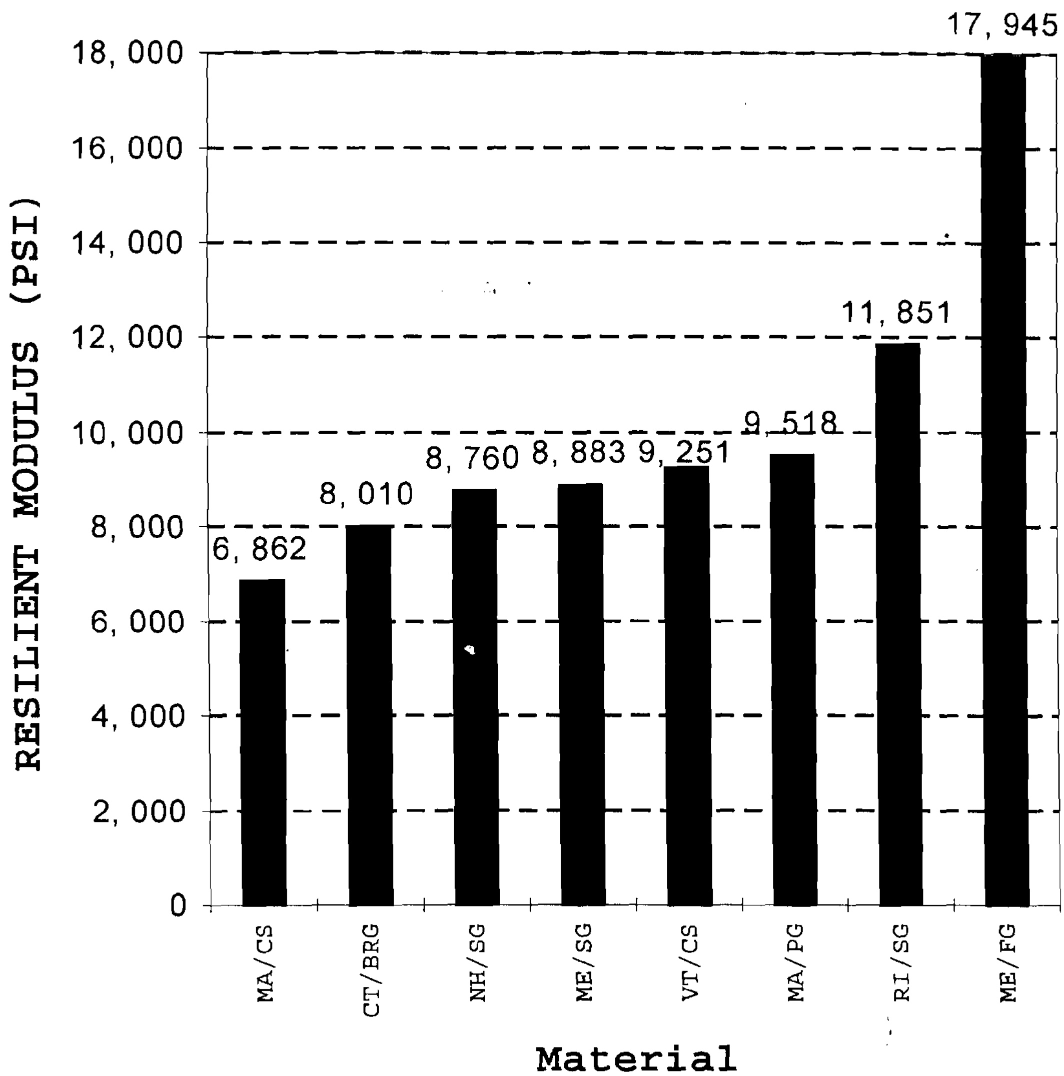


Figure 4.44 Comparison of Resilient Moduli Determined by the AASHTO T292-91 Procedure for Granular Subbase Materials in New England

4.5 Comparative Analysis of Resilient Modulus

The resilient moduli test determined in accordance with the procedures of AASHTO T292-91 and AASHTO TP46 were compiled in Table 4.9. A two sample t-test was performed to check whether the two results have statistically significant difference. In statistics, there are two types of hypothesis: (1) null hypothesis (H_0), and (2) alternative hypothesis (H_a). In this case, the hypothesis are as follows:

H_0: There is no significant difference in results between the two testing procedures (AASHTO T292-91 and AASHTO TP46)

H_a: There is significant difference in results between the two testing procedures (AASHTO T292-91 and AASHTO TP46)

If variances of two observations are assumed to be equal, the t-distribution is given by

$$t_0 = \frac{\overline{x_1} - \overline{x_2}}{s_p \sqrt{\frac{1}{n_1} + \frac{1}{n_2}}}$$

Where s_p is the pooled standard deviation of $x_1$bar – $x_2$bar which can is given by

$$s_p = \sqrt{\frac{(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2}{n_1 + n_2 - 2}}$$

where

$x_1$bar and $x_2$bar are the means of two observations

n_1 and n_2 are number of observations, and

s_1 and s_2 are standard deviations of the two data sets.

From the above two equations the t value was computed to be 3.41. But, since t(3.41) > $t_{0.025,\,14}$ (2.145), the H_0 was rejected. It was concluded that there is significant difference at 95% degree of confidence.

The correlation coefficient was also computed to be 0.44 which indicates a low correlation between the two results. In summary, it was found that there is a significant difference in results determined with the two AAHSTO T292-91 and AASHTO TP46 procedures. This difference in resilient modulus values can be explained by attributes listed below

1. Maximum Particle Size:

   AASHTO T292-91: The maximum particle size of the material was no larger than ¼ of the specimens diameter i.e., 4 in. (25 mm or 1 in.).

   AASHTO TP46: The maximum particle size exceeding 25 percent of the specimens diameter i.e., 6 in. were scalped. (37.5 mm or 1 ½ in.)

2. Compaction Effort:

   AASHTO T292-91: Specimens were compacted in 5 lifts with approximate equal layers in a 4 in. diameter split mold with a 5 lb. rammer.

   AASHTO TP46: Specimens were compacted in 6 lifts with precise amount of material for each layer using a vibratory compaction with a rated input of 750 to 1250 watts and capable of 1,800 to 3,000 blows per minute, thus giving higher compaction.

3. Loading Sequence:

   The specimen conditioning sequence aids in eliminating the effects of any specimen disturbance due to sampling, compaction, and specimen preparation procedures. However, it only results in partial elimination of initial disturbance, as the specimen gets more consolidated as the sequence progresses.

   AASHTO T292-91: Cyclic loads were applied on the sample in a descending order resulting in a quick consolidation of sample, which means that the deformations observed at the end of the sequence (lower cyclic loads) would be of lower value compared to what it would have been.

AASHTO TP46: Cyclic loads were applied on the sample in an ascending order, which would compact the soil at regular stages, thus resulting in more reasonable and reliable values of deformation for lower cyclic loads.

4.6 Permeability of Granular Subbase Materials

Drainage is generally treated by considering the effect of water on the properties of the pavement layers and the consequences to the structural capacity of the pavement. In the AASHTO Guide, the effect of drainage is considered by modifying the structural layer coefficient for flexible pavements as a function of

(1) the quality of drainage (e.g., the time required for the pavement to drain)

(2) the percent of time the pavement structure is exposed to moisture levels approaching saturation.

In the present study, the permeability was used to estimate the quality of drainage.

The coefficient of permeability was determined in accordance with AASHTO T215-70 Permeability of Granular Soils (Constant Head). It is a constant head method for the laminar flow of water through granular soils. It is intended to establish representative values of the coefficient of permeability for granular materials used as subbase courses. Materials should not have more than 10 percent passing the 0.075 mm (No. 200) sieve.

The procedure of AASHTO T215-70 requires a 150 mm (6 in.) mold when the maximum particle size lies between sieve openings 9.5 mm (3/8 in.) and 19 mm (3/4 in.) and more than 35 percent of the total soil is retained on the 9.5 mm (3/8 in.) sieve. The minimum diameter of permeameter should be 8 x maximum particle size (8 x .75 in. = 6 in.). A Soil Test Model K-612A compaction permeameter was selected in the present study. This mold and base permits mechanical compaction at OMC prior to testing. The same Soil Test mechanical compactor was used to prepare specimens as was used for the proctor testing.

The coefficient of permeability can be calculated as follows:

$$k = \frac{QL}{Ath} \quad \text{(4.9)}$$

where

k = coefficient of permeability (cm/sec.)
Q = discharge of water in t seconds cm^3
L = length of the specimen, cm
A = area of specimen cross section, cm^2
t = time to discharge Q water, seconds
h = pressure head (cm of water)

Table 4.10 presents the results of permeability test for New England subbase materials.

The coefficients of permeability fall within the range of 10^{-3} and 10^{-6}. Clean sand and sand and gravel mixtures are expected to have values from 1 to 10^{-3} and glacial tills would have lower values in the 10^{-3} to 10^{-5} range (Mcarthy 1977). The Maine Frenchville subbase material exhibited poor permeability, or practically impervious. When using this material in a pavement structure a drainage layer would be strongly recommended. Although this material has shown a good strength characteristic, it is composed of friable material. It may be noted that fines are generated during compaction and loading which seriously retard drainage.

The New Hampshire sandy gravel produced the highest coefficient of permeability. This material was so uniformly graded that it was difficult to compact and retain in the mold. It had the lowest dry density and a large void ratio. This is what would be expected of a clean coarse sand with gravel mixture.

4.6.1 Estimation of Drainage Coefficients

Appendix DD of AASHTO Guide describes the development of drainage coefficients used in pavement design procedures. A drainage coefficient, m_i is used to reduce or increase the layer coefficient of the subbase layer. Positive drainage within the pavement structure will have a beneficial effect on the life of the pavement structure.

The drainage conditions must be assessed in terms of good, fair, and poor drainage conditions. The method recommended by the FHWA Report TS-80-224 ("Highway" 1980) requires the calculation of the time required to drain the base layer to 50% saturation (T_{50}). It is determined for different combinations of permeability (k), length of drainage path (L), effective porosity (n), and slope (S). The results of these calculations are included in Table DD.1 of the AASHTO Guide. The present study estimated the drainage coefficient by correlating this table with the permeability of the materials tested in feet per day. The drainage coefficients, m_i were calculated in this fashion (Table 4.11). These m_i values could be refined using specific characteristics for future individual projects.

4.7 Demonstration of the Effectiveness of the Developed Parameter Values

The effectiveness of the developed parameter values for Rt. 2 subbase materials were demonstrated through a design example (structural analysis). The AASHTO Guide procedure was used for the Rhode Island pavement structure. It was chosen mainly due to the availability of the other necessary parameters values. These parameters were extracted from the data accumulated by the URI research team and/or provided by the RIDOT,e.g., effective soil resilient modulus, accumulated 18-kip ESAL, etc. (Kovacs, et al. 1991; Lee, et al. 1994a; Lee, et al. 1994b; Lee, et al. 1999).

4.7.1 Design Requirements

Time Constraints

The analysis period selected for this design example is 20 years. The maximum performance period or initial service life selected for the initial pavement structure is 15 years. Therefore, it will be necessary to consider stage construction (i.e., planned rehabilitation) alternatives to develop design strategies which will last the analysis period.

Traffic

Based on average daily traffic and axle weight data, the traffic during the first year (in the design lane) was estimated to be 399,540 18-kip ESAL applications. Figure 4.45 provides a plot of the cumulative 18-kip ESAL traffic over the 20-year analysis period.

Reliability

A 90% overall reliability level was selected for design. This means that for a two-stage strategy, the design reliability for each stage must be $(0.9)^{0.5}$ or 95%. An approximate value of 0.35 for the standard deviation (S_0) is used in this example.

Environmental Impacts

The site of this highway construction project in Rhode Island is in a location that can be classified as U.S. Climatic Region II, i.e., wet with freeze thaw cycling. It was assumed that good drainage will be provided for the excess moisture removal in less than 1 day to prevent frost heave problems. The subgrade soil is not expansive. The AASHTO Classification is A-1-b, and the USC is SM, i.e., silty sand. The plastic index value is less than 4. Using Figure G3 of the AASHTO Guide, the vertical rise is zero. Therefore it is assumed that there is no serviceability loss due to swelling.

Serviceability

A terminal serviceability (P_t) of 2.5 was selected. Past experience is that the initial serviceability (P_o) achieved for flexible pavement in Rhode Island is 4.6. The overall design serviceability loss for this example is 4.6 - 2.5 = 2.1.

Effective Roadbed Soil Resilient Modulus

Application of the effective roadbed soil resilient modulus estimation procedure resulted in a value of 6,120 psi (Lee, et al. 1999).

Pavement Layer Materials Characterization

Three types of pavement materials will constitute the individual layers of the structure. They may be characterized by the layer coefficients.

Layer Coefficients

The structural layer coefficients (a_i-values) are as follows:

Asphalt Concrete Surface: E_{AC} = 325,000 psi

Asphalt Concrete Binder: E_{BI} = 540,000 psi

Asphalt Concrete Base : E_{BS} = 480,000 psi

Asphalt Concrete: a_1 = 0.45 (weighted)

Granular Subbase : E_{SB} = 11,851 psi (AASHTO T292-91) $\Rightarrow a_3 = 0.09$

E_{SB} = 15,327 psi (AASHTO TP46-94) $\Rightarrow a_3 = 0.11$

Drainage Coefficient

The drainage coefficient which corresponds to the granular subbase materials with "good" drainage (i.e., water removed within one day) and a balanced wet dry climate is 0.7 (for greater than 25% moisture exposure time).

4.7.2 Development of a Design Alternative

The strategy with the maximum recommended initial structural number was determined using the effective roadbed soil resilient modulus of 6,120 psi, a reliability of 95%, an overall standard deviation of 0.35, a design serviceability loss of 2.1 and the cumulative traffic of 7.1×10^6 ESAL at the maximum performance period. The maximum initial structural number (SN) of 5.0 was determined as shown in Figure 4.46.

4.7.3 Determination of Structural Layer Thickness for the Initial Structure

The SN required above the subbase material was determined by applying Figure 3.1 of the AASHTO Guide using the resilient modulus of the subbase material. Values of E_{SB} used were 11,851 psi and 15,327 psi determined by procedures of AASHTO T292-91 and AASHTO TP46, respectively (Table 4.9). With the first stage reliability (R) of 95%, W_{18} of 7.1×10^6 and $\Delta PSI = 2.1$, it resulted in a structural numbers of 4.0 and 3.6 for subbase modulus values determined by AASHTO T292-91 and AASHTO TP46 respectively. Since the design nomograph cannot be used for layers having elastic modulus values higher than 40,000 psi, the asphalt concrete layers were considered as one layer. The asphalt layers and granular subbase thickness required were determined as follows:

For E_{SB} values determined by <u>AASHTO T292-91</u>

$SN_1 = a_1 D_1$

$4.0 = 0.45 D_1$

$D_1 = 8.9$

$D_1^* = 9$ in.

$SN_1^* = 0.45 \times 9 = 4.05$

$D_3 = (SN_3 - SN_1^*)/(a_3 m_3)$

$= (5.0 - 4.05)/(0.09 \times 0.7)$

$= 15.1$ in.

$D_3^* = 15.5$ in.

$SN_3^* = 5.03$

For E_{SB} values determined by <u>AASHTO TP46</u>

$SN_1 = a_1 D_1$

$3.6 = 0.45 D_1$

$D_1 = 8$ in.

$D_1^* = 8$ in.

$SN_1^* = 0.45 \times 8 = 3.6$

$D_3 = (SN_3 - SN_1^*)/(a_3 m_3)$

$= (5.0 - 3.6)/(0.11 \times 0.7)$

$= 18.2$ in. use 18.5 in.

$D_3^* = 18.5$ in.

$SN_3^* = 5.03$

The analysis with the subbase resilient modulus determined by AASHTO T292-91 procedure resulted in a structure of 2 in. asphalt surface, 2 in. asphalt binder and 5.0

in. asphalt base, or a total of 9.0 in. The subbase layer would have a thickness of 15.5 in, which is thicker than the Rhode Island standard minimum depth of 12 in.

The analysis with the subbase resilient modulus determined by AASHTO TP46-94 procedure resulted in a structure of 2 in. surface, 2 in. binder and 4.0 in. of modified asphalt base for a total of 8.0 in. The subbase layer would have a thickness of 18.5 in., which is thicker than the Rhode Island standard minimum depth of 12 in.

It may be noted that the asphalt base layer was thinner and that the granular subbase layer was thicker, if the subbase resilient modulus was determined in accordance with the procedure of AASHTO TP46-94.

The effectiveness of the developed parameter values has also been demonstrated for the same pavement structure in Rhode Island using the DARWin™ 2.01 as shown in next page . The same trend was observed as the nomograph design. However, the design using the DARWin™ 2.01 software provided a little thicker asphalt base and granular subbase layers.

1993 AASHTO Pavement Design

DARWin(tm) Pavement Design System

A Proprietary AASHTOWARE(tm)
Computer Software Product

Flexible Structural Design Module

SUBBASE RESILIENT MODULUS DETERMINED BY THE AASHTO T292-91 PROCEDURE

Flexible Structural Design Module Data

18-kip ESALs Over Initial Performance Period:	7,100,000
Initial Serviceability:	4.6
Terminal Serviceability:	2.5
Reliability Level (%):	95
Overall Standard Deviation:	.35
Roadbed Soil Resilient Modulus (PSI):	6,120
Stage Construction:	2

Calculated Design Structural Number: 4.93

Layered Thickness Design

Thickness precision: Round up to nearest 1/2 inch

Layer	Material Description	Struct. Coef. (Ai)	Drain. Coef. (Mi)	Spec. Thick. (Di) (in)	Min. Thick. (in)	Elastic Modulus	Width (ft)	Calculated Thickness (in)	Calculated SN
1	Type I-1	.42	1	2.00	-	325,000	12	2.00	.84
2	Bituminous Binder	.42	1	2.00	-	540,000	12	2.00	.84
3	Bituminous Base	.45	1	-	-	480,000	12	5.50	2.48
4	Granular Subbase	.09	.7	-	-	11,851	12	15.50	.98
Total	-	-	-	-	-	-	-	25.00	5.13

Table 4.9 Comparison of Resilient Modulus Test Results Determined by AASHTO T292-91 and AASHTO TP46-94 procedures

Material	AASHTO T292-91 X_1	AASHTO TP46 X_2
CT/BRG	8,010	14,198
ME/FG	17,945	16,142
ME/SG	8,883	14,784
MA/CS	6,862	11,995
MA/PG	9,518	13,786
NH/SG	8,760	17,932
RI/SG	11,851	15,327
VT/CS	9,251	14,469
S.D	3460	1742
Mean	10,135	14,829

Table 4.10 Permeability Test Results

Material	Maximum Particle size	Percent material removed	specific gravity	MDD pcf	*e*	K cm/sec (ft/day)
CT BRG	127mm (5 in.)	44	2.66	126.1	.32	$4.6x10^{-4}$ (1.30)
ME FG	102mm (4in.)	50	2.90	145.4	.24	$1.0x10^{-6}$ (0.03)
ME SG	127mm (5in.)	24	2.66	136.1	.22	$1.4x10^{-4}$ (0.40)
MA CS	51mm (2in.)	35	2.75	139.5	.23	$3.7x10^{-5}$ (0.11)
MA PG	38mm (1.5in.)	30	2.66	125.0	.33	$2.2x10^{-5}$ (0.06)
NH SG	76mm (3in.)	32	2.66	117.5	.41	$4.0x10^{-3}$ (11.34)
RI SG	12.7mm (0.5in.)	0	2.66	129.7	.28	$3.2x10^{-5}$ (0.09)
VT CS	76mm (3in.)	40	2.66	135.0	.23	$1.0x10^{-4}$ (0.28)

note: 1. volume of 6 in. (i.d.) x 7 in. (h) mold = .1145cf

2. *e* = void ratio

$$= \frac{(\text{specific gravity* x density of water})}{\text{dry density}} - 1$$

= volume of voids / volume of solids

* based on mineral composition

Table 4.11 Estimation of a Drainage Coefficient for Flexible Pavement Design

Material	Permeability cm/sec (ft/day)	Water Removed Within (days)[1]	Quality of Drainage	Est. Drainage Coefficient (m_i)[2]
CT BRG	4.6×10^{-4} 1.30	2-6	fair	1.0
MAINE FG	1.0×10^{-6} 0.03	15-36	poor	0.7
MAINE SG 631,209	1.4×10^{-4} 0.40	4-29	poor to fair	0.7 to 1.0
MASS CS	3.7×10^{-5} 0.11	10-36	poor	0.7
MASS PG	2.2×10^{-5} 0.06	15-36	poor	0.7
NH/SG	4.0×10^{-3} 11.34	0.2-1	good	1.2
RI/SG	3.2×10^{-5} 0.09	12-40	poor	0.7
VT/CS	1.0×10^{-4} 0.28	2-36	poor to fair	0.7 to 1.0

1 Table DD.1 in AASHTO Guide Appendix DD
2 Table DD.3 in AASHTO Guide Appendix DD

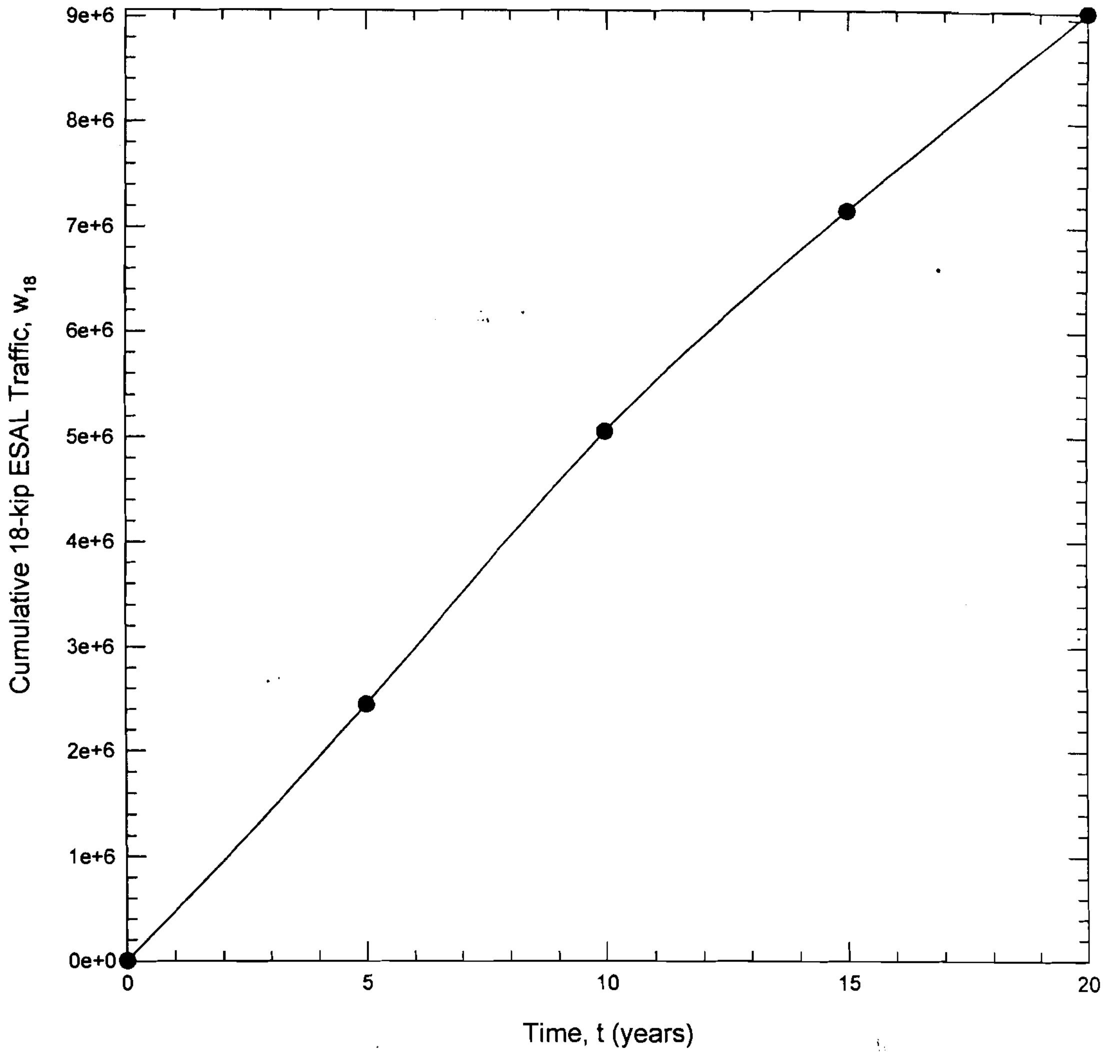


Figure 4.45 Plot of Cumulative 18-kip ESAL Traffic Versus Time for the Pavement Structure of Rt. 2, RI

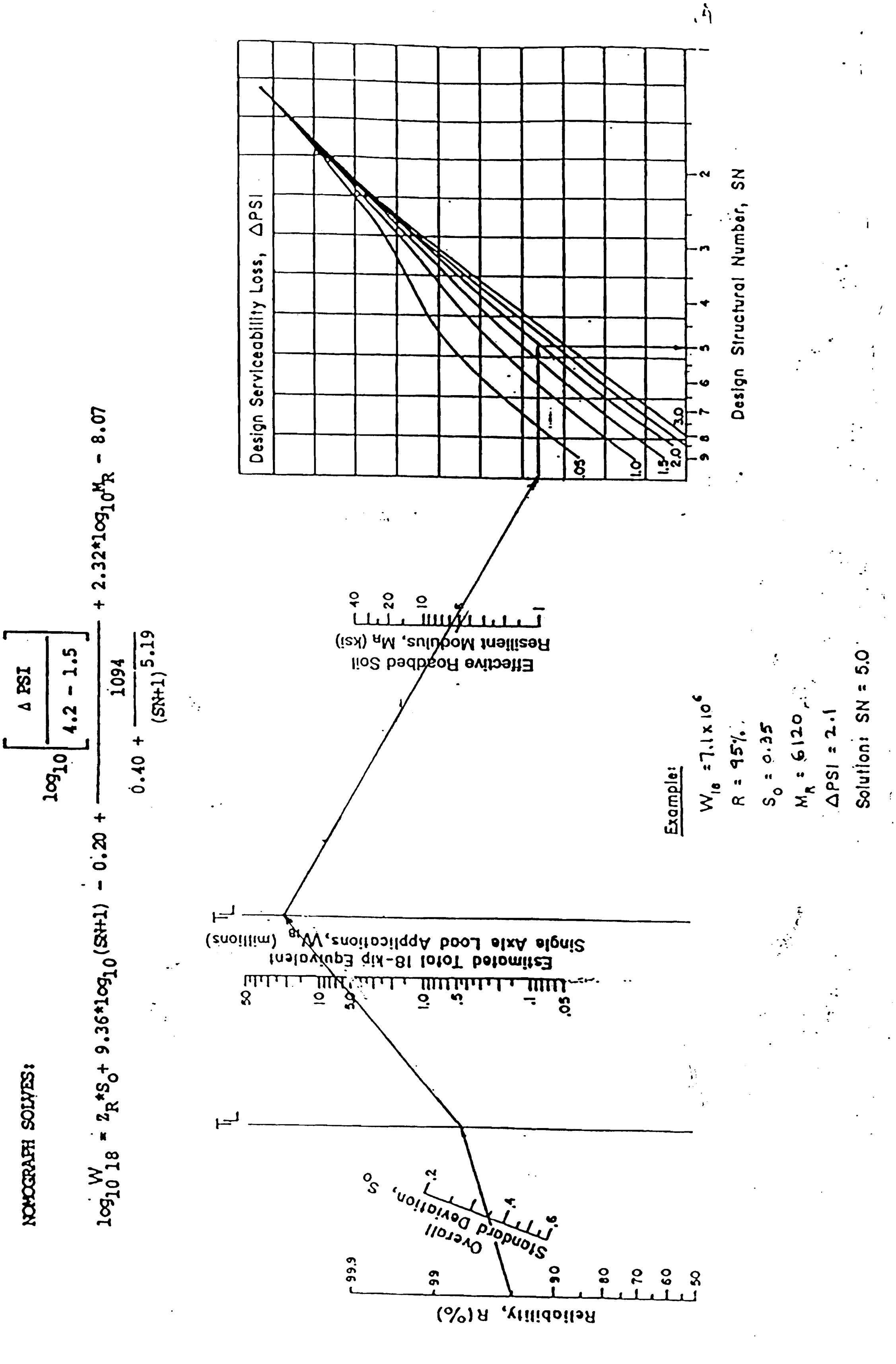


Figure 4.46 Determination of SN for Pavement Structure of RI State Rt. 2

CHAPTER 5 STRUCTURAL ANALYSIS OF SUBBASE MATERIALS WITH RECLAIMED MATERIALS

One of the objectives was to develop optimum performance characteristics for recycled material blends. This chapter discusses experimental program, fundamental test results, resilient modulus and permeability of subbase materials with reclaimed materials.

5.1 Experimental Program

The reclaimed asphalt pavement (RAP) was blended with processed gravel (PG) provided by the Massachusetts Highway Department (MAHWD) in the laboratory at six levels. The results of the PG material testing provided the benchmark. The blend levels were 0, 40, 50, 60, 70 and 100 percent milled RAP. The stockpiled RAP which passed a 19 mm(3/4 in.) sieve has been procured from the MAHWD ("Specifications," 1995). The literature review has indicated that the critical pavement performance characteristic is resilient modulus of subbase materials, and this characteristic was examined by blend to determine the optimum amount ("Guide" 1993).

The State of Massachusetts is currently reclaiming full depth for state routes. The State of Connecticut is also using a blend of gravel, RAP and recycled Portland cement concrete (PCC). This recycled material blend was also tested, and characteristics were compared to the laboratory blends.

5.2 Fundamental Test Results

Connecticut reclaimed mixture of gravel, RAP and PCC (GRC) and Massachusetts cold in place RAP were tested for the parameters listed in Table 4.1. The identical procedures in Chapter 4 were followed.

A letter provided by the MAHWD from a construction materials consultant (testing) of Canton, MA states that reclaimed base courses contain approximately 1% - 3% asphalt content which is well below the 6% in new pavements. It also reports that the asphalt cements in reclaimed base courses are old and oxidized; and any volatile (present in new asphalt cement) have evaporated. The state of Connecticut has limited the asphalt content of its GRC to 2% as discussed in Chapter 3. Asphalt content is beyond the scope of this project, but should be considered as a fundamental property in the future.

Connecticut GRC was found to contain 5% more 37.5mm (1.5 in.) grain size material than specified by the state as shown in Figure 5.1. Massachusetts RAP met the state grain size distribution envelope specified. Both materials had similar mid-range maximum dry density in comparison to the virgin granular materials tested (Table 5.1 and Figures 5.3 & 5.4). The GRC has a high optimum moisture content for a subbase material. This may be attributed to the crushed PCC. These reclaimed materials were classified A-1-a as good subbase material (Table 5.2). Maximum dry density and optimum moisture contents for MA/PG with different amount of RAP were also determined in order to pursue further testing and the results are shown in Table 5.3 and Figures 5.5 through 5.9.

5.3 Resilient Modulus of Granular Subbase Materials with Reclaimed Material

A detailed discussion of AASHTO T292-91 and AASHTO TP46 testing procedures were presented in Chapter 4. Connecticut GRC and Massachusetts RAP were tested in addition to laboratory reclaimed subbase material blends (5.1 Experimental Program). The bulk stress analysis for typical cross sections of Connecticut and

Massachusetts pavement structure was applied to estimate a layer coefficient for the reclaimed subbase material blends.

Results of resilient modulus tests by AASHTO T292-91 have been summarized in Table 5.4 and graphically illustrated in Figure 5.10 through 5.16. Figure 5.17 through 5.23 show results of resilient modulus test done in accordance to AASHTO TP46 and the results are summarized in Table 5.5.

During the testing of specimens using AASHTO T292-91 testing procedure it was difficult to achieve reliable R^2 values when testing reclaimed material. The 50%, 60% and 100% RAP blends were tested a minimum of twice, and test result with the higher R^2 is provided for this report. This consistent observation of unstable results may be caused by friability of the reclaimed materials.

E_{SB} results at mid-depth were found to increase with an increase in RAP content (Figure 5.24 and Figure 5.25). The 50% blend with an E_{SB} of 18,000 psi is at the highest value found in the natural aggregates tested in this study. Additionally, 18,000 psi is highest value provided by the AASHO Road Test subbase materials when wet ("Guide" 1993). Massachusetts gravel was the control virgin aggregate used for blending. May be noted that the MA/PG had lower modulus than any other blends. The pure milled asphalt RAP had the highest strength when tested. Resilient moduli determined by the AASHTO T292-91 procedure have been summarized in Table 5.6, and have been compared in Figure 5.24. The ones determined by the AASHTO TP46-94 have been summarized in Table 5.7, and has been compared in Table 5.25.

5.3.1 Comparative Analysis of Resilient Modulus.

A two sample t-test similar to Section 4.5 was performed to check whether the two results by AASHTO T292-91 and AASHTO TP46 procedures have statistically significant difference.

H_0: There is no significant difference in results between two testing procedures (AASHTO T292-91 and AASHTO TP46)

H_a: There is a significant difference in results between two testing procedures (AASHTO T292-91 and AASHTO TP46)

The t-value was computed as 1.0899. But, since $t(1.0899) < t_{0.025,\,12}$ (2.179), the alternate hypothesis was rejected. It was concluded that there is no significant difference at 95% degree of confidence. The correlation coefficient was computed to be 0.75, which indicates that there is a high correlation between two results.

5.4 Determination of Optimum RAP Amount

Initially, experiment was performed with six blends to determine the optimum RAP amount using the AASHTO T292-91 procedure. The pure RAP was by far the highest modulus value, but initially the 60% modulus was lower than the 50% blend. This may indicate that the E_{SB} is lower with more RAP, which made it necessary to test a 70% blend. Since an increasing trend was then observed, additional testing of the 60% blend revealed the final resulting upward trend. Figure 5.26 has two different test series moduli values for the 50% and 60% blends plotted on a second degree regression curve for the final optimization analysis.

A clear trend of increasing strength with increased RAP content has been shown. The 40, 50 and 60% blends provide a minimum increase of 50% higher E_{SB} value than the natural aggregate. The 60% and 70% blends have an E_{SB} greater than 26,000 psi above the increase in slope after 50%. Granular material with a modulus of 26,000 psi is stiff enough to use for a base layer. RAP with similar modulus values is currently used as a base layer in New England. It was found that the range of New England virgin subbase materials is between 6,862 and 17,945 psi (Table 4.7). Therefore, the blend amount at 17,945 was determined as the optimum RAP amount, i.e.; 50%.

Similar analysis was performed with resilient modulus values determined with the AASHTO TP46-94 procedure. It was found that the range of New England virgin subbase materials is between 11,995 and 17,932 (Table 4.8). Therefore, the blend amount at 17,932 was determined as the optimum RAP amount i.e.; 46% (Figure 5.27).

5.5 Permeability of Reclaimed Subbase Materials

The coefficient of permeability was determined in accordance with AASHTO T215-70 Permeability of Granular Soils (Constant Head). Reclaimed subbase materials were tested in the same manner as the virgin aggregate blends.

The Massachusetts RAP was field processed with a CMI RS-500 reclaimer stabilizer. It has a permeability in the range of glacial tills at 10^{-5}. This material will provide some poor drainage.

The Connecticut gravel RAP and concrete exhibited poor permeability, or practically impervious at 10^{-6} cm/sec (McCarthy 1977). When using this material in a pavement structure a drainage layer would be strongly recommended. Although this material has shown a good strength characteristic, it is composed of friable materials. It may be noted that fines are generated during compaction and loading which seriously retard drainage.

It should also be noted that the Connecticut material contains Portland cement rubble. This material is known to out-perform naturally occurring granular material, because it is lighter and carries loads just as well. No problems related to the subbase course have been observed on any highways constructed from this material in New York State, some of which are now more than 10 years old. The pH is however, usually above 11, a level that is corrosive to aluminum and the zinc galvanizing on pipes. A calcium-based solution has also been found by other states to leach from reclaimed Portland cement concrete, encrusting porous media and pipes, which prevents proper subsurface drainage. This can lead to early distress and failure of supported pavements. (Wheeler 1996). Table 5.8 presents the results of permeability test for New England Subbase materials with reclaimed materials.

5.5.1 Estimation of a Drainage Coefficient

Appendix DD of AASHTO Guide describes the development of drainage coefficients used in flexible and rigid pavement design procedures ("Guide" 1993). An "m-value" is used to reduce or increase the layer coefficient of the subbase layer. Positive drainage within the pavement structure will have a beneficial effect on the life of the pavement structure.

The drainage conditions must be assessed in terms of good, fair, and poor drainage conditions. The method recommended by the FHWA Report TS-80-224

("Highway" 1980) requires the calculation of the time required to drain the base layer to 50% saturation(T_{50}). T_{50} is determined for different combinations of permeability (k), length of drainage path (L), effective porosity (n), and slope (S). The results of these calculations are supplied in Table DD.1 of AASHTO Guide. This study estimated the drainage coefficient by correlating Table DD.1 with the permeability of the materials tested in feet per day (Table 5.9). An estimated range of m value has been calculated in this fashion. However, these m values should be recalculated using project specific information for all the parameters during the design period.

5.6 References

1. "Guide for Design of Pavement Structures." (1993). American Association of State Highway and Transportation Officials (AASHTO).
2. "Specifications for Subbase Materials", (1995). Massachusetts Highway Department.
3. McCarthy, F.D., "Essentials of Soil Mechanics and Foundations", 1977.
4. "Highway Subdrainage Design", (1980) U.S. Department of Transportation, Federal Highway Administration (FHWA).
5. Wheeler, Jr., John J., "Waste Materials in Highway Construction – Lessons from New York State", TR News No. 184 May – June 1996.

Table 5.1 Summary of Compaction Test Results for Reclaimed Materials

State Materials	Proctor Test AASHTO T 180-90	
	OMC %	γ_d Kg/m^3 (pcf)
CT GRC	8.8	2003.8 (125.1)
MA RAP(cold in place reclaimed)	7.3	2038.5 (127.2)

Table 5.2 Classification of Reclaimed Subbase Materials

State Materials	Soil Classification	Plasticity Index
CT GRC	A-1-a	Non-plastic
MA RAP(cold in place reclaimed)	A-1-a	Non-plastic

Table 5.3 Summary of Moisture Density Test Results for Different Blends of RAP with Subbase Materials

Materials	Proctor Test AASHTO T 180-90	
	OMC %	γ_d Kg/m^3 (pcf)
100% RAP	7.0	1789.3 (111.7)
70% RAP	7.4	1976.7 (123.4)
60% RAP	7.5	1965.5 (122.7)
50 % RAP	8.2	1999.1 (124.8)
40% RAP	6.6	2021.5 (126.2)

Table 5.4 Summary of Resilient Modulus Test Results (AASHTO T292-91)

Material	Source	K_1	K_2	R^2
CT GRC	F	5184	.54	.69
MA RAP	F	8390	.56	.70
MA PG	Virgin	3058	.58	.95
MA 40%RAP	Lab.	5672	.49	.91
MA 50%RAP	Lab.	4997	.66	.65
MA 60%RAP	Lab.	9021	.55	.66
MA 70%RAP	Lab.	12927	.44	.68
MA RAP	Milled	25469	.36	.61

Note: $E_{sb} = K_1 \, \theta^{K2}$
(L) = laboratory blend
(F) = field construction blend
(M) = pure milled asphalt from MAHWD inspected stockpile

Table 5.5 Summary of Recycled Material Resilient

Modulus Test Results (AASHTO TP46)

Material	Source	K_1	K_2	K_5	R^2
CT GRC	F	9688	0.19056	0.37299	0.86
MA RAP	F	13338	0.13115	0.39282	.083
MA PG	Virgin	8520	0.14347	0.37425	.083
MA 40RAP	Lab	10157	0.14882	0.39412	.085
MA 50RAP	Lab	8869	0.20697	0.55092	0.80
MA 60RAP	Lab	11155	0.16936	0.39555	0.84
MA 70RAP	Lab	13888	0.12505	0.37286	0.81
MA RAP	Milled	16019	0.10684	0.34094	.081

Note: $E_{sb} = K_1 (S_c)^{K2} (S_3)^{K5}$
(L) = laboratory blend
(F) = field construction blend
(M) = pure milled asphalt from MAHWD inspected stockpile

Table 5.6 Resilient Moduli of Subbase Materials with Reclaimed Materials for New England States Using the AASHTO T292-91 Procedure

Material	$\Sigma\sigma_s$ psi	$\Sigma\sigma_d$ psi	θ psi	$M_r=k_1\theta^{k2}$	E_{sb} (psi) (292-91)	a_3
MA RAP(F)	1.93	5.15	7.08	$8390.5\theta^{0.56}$	25,107	0.16
CT GRC(F)	1.99	4.48	6.47	$5184.4\theta^{0.54}$	14,204	0.10
MA PG(F)	1.93	5.15	7.08	$3058.8\theta^{0.58}$	9,518	0.06
MA 40RAP(L)	1.93	5.15	7.08	$5672.0\theta^{0.49}$	14,800	0.11
MA 50RAP(L)	1.93	5.15	7.08	$5042.0\theta^{0.66}$	18,349	0.13
MA 60RAP(L)	1.93	5.15	7.08	$9021.0\theta^{0.55}$	26,471	0.17
MA 70RAP(L)	1.93	5.15	7.08	$12927.6\theta^{0.44}$	30,586	0.18
MA RAP(M)	1.93	5.15	7.08	$25469\theta^{0.36}$	51,525	0.23

Note:

1. Bulk stress was analyzed at mid-depth

2. Resilient modulus tests were performed in accordance with AASHTO T292-91

(L) = laboratory blend
(F) = field construction blend
(M) = pure milled asphalt from MAHWD inspected stockpile

Table 5.7 Resilient Moduli of Subbase Materials with Reclaimed Materials for New England States Using the AASHTO TP46-94 Procedure

Material	S_3 psi	S_c psi	$M_r = K1(S_c)^{K2}(S_3)^{K5}$	E_{sb} (psi) (TP46)	a_3
CT GRC(F)	1.99	4.48	$9688(S_c)^{0.19056}(S_3)^{0.37299}$	19,324	0.13
MA RAP(F)	1.93	5.15	$13338(S_c)^{0.13115}(S_3)^{0.39282}$	21,410	0.14
MA PG(F)	1.93	5.15	$8520(S_c)^{0.14347}(S_3)^{0.37425}$	13,786	0.10
MA 40RAP(L)	1.93	5.15	$10157(S_c)^{0.14882}(S_3)^{0.39412}$	16,797	0.12
MA 50RAP(L)	1.93	5.15	$8869(S_c)^{0.20697}(S_3)^{0.55092}$	17,886	0.13
MA 60RAP(L)	1.93	5.15	$11155(S_c)^{0.16936}(S_3)^{0.39555}$	19,097	0.13
MA 70RAP(L)	1.93	5.15	$13888(S_c)^{0.12505}(S_3)^{0.37286}$	21,783	0.15
MA RAP(M)	1.93	5.15	$16019(S_c)^{0.10684}(S_3)^{0.34094}$	23,880	0.16

Note:

1. Bulk stress was analyzed at mid-depth

2. Resilient modulus tests were performed in accordance with AASHTO TP46

(L) = laboratory blend
(F) = field construction blend
(M) = pure milled asphalt from MAHWD inspected stockpile

Table 5.8 Permeability Test Results for Reclaimed Subbase Materials

Material	Max. Particle size	% material discarded	specific gravity	MDD pcf	*e*	k cm/sec (ft/day)
CT GRC(F)	38mm 1.5 in.	20	2.66	125.1	.33	$2.5x10^{-6}$ (0.01)
MA RAP(F)	51mm 2 in.	27	2.66	127.2	.30	$3.8x10^{-5}$ (0.11)

note: 1. volume of 6 in. (i.d.) x 7 in. (h) mold = .1145cf

e = void ratio

$$= \frac{\text{specific gravity* x density of water}}{\text{dry density}} - 1 \quad \text{.....(4.6)}$$

= volume of voids / volume of solids

* based on mineral composition

2. (F) = field construction blend

Table 5.9 An Estimation of a Drainage Coefficient for Flexible Pavement Design

Material	Permeability cm/sec (ft/day)	Water Removed Within (days)[1]	Quality of Drainage	Est. Drainage Coefficient(m)[2]
CT GRC(F)	$2.5x10^{-6}$ 0.01	12-40	poor	0.7
MA RAP(F)	$3.8x10^{-5}$ 0.11	10-36	poor	0.7

1 Table DD.1 in AASHTO Guide Appendix DD
2 TABLE DD.3 in AASHTO Guide Appendix DD

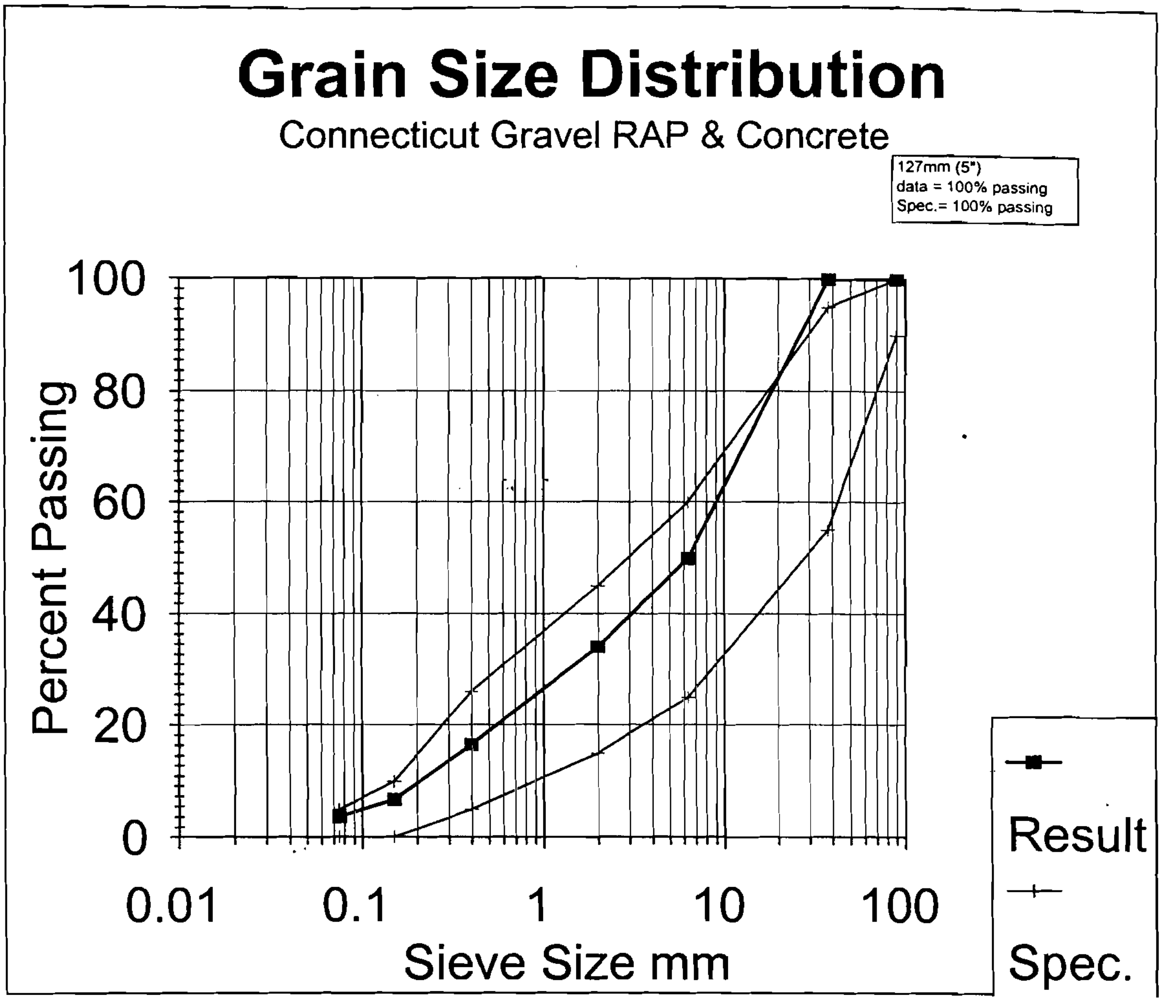


Figure 5.1 Connecticut GRC Grain Size Distribution

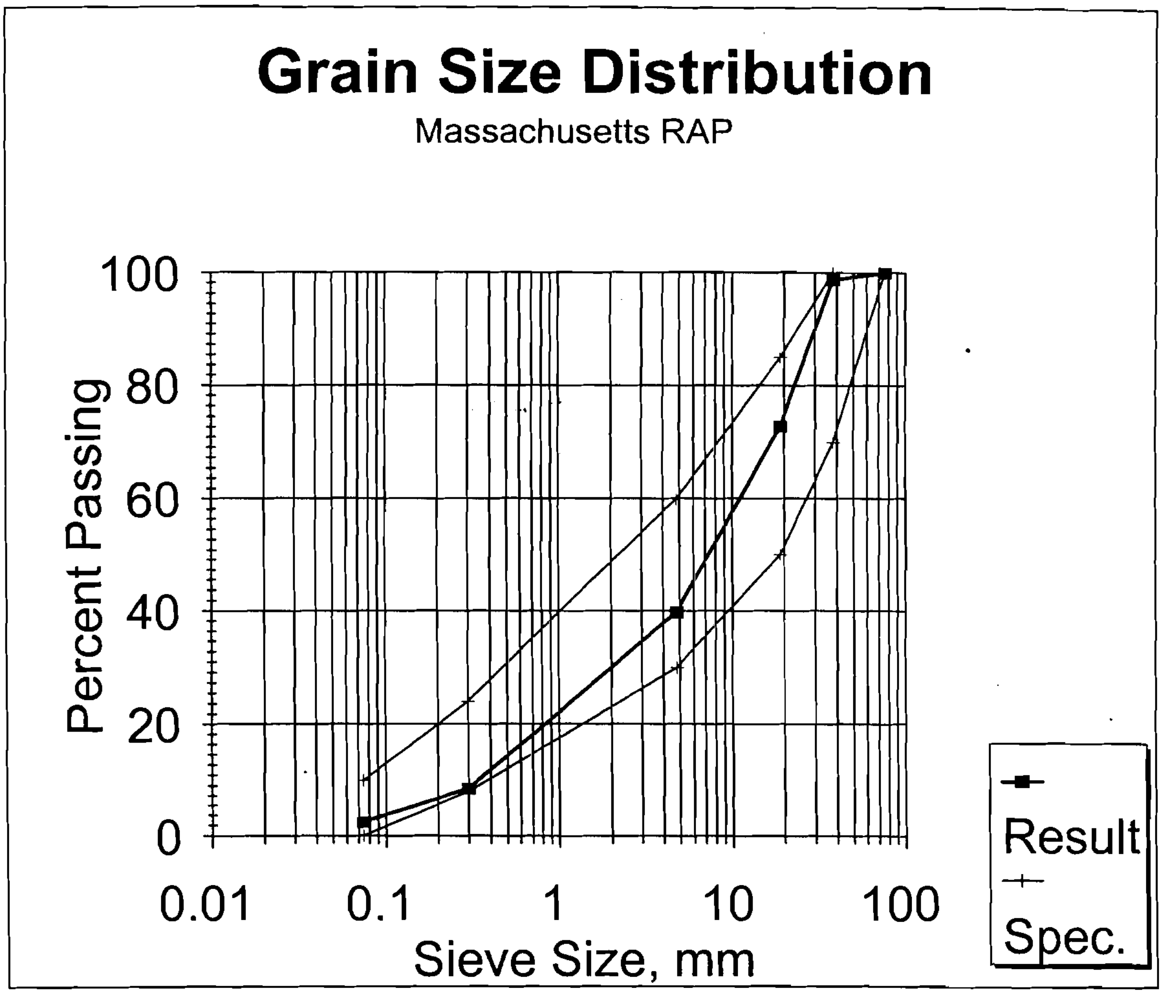


Figure 5.2 Massachusetts RAP Grain Size Distribution

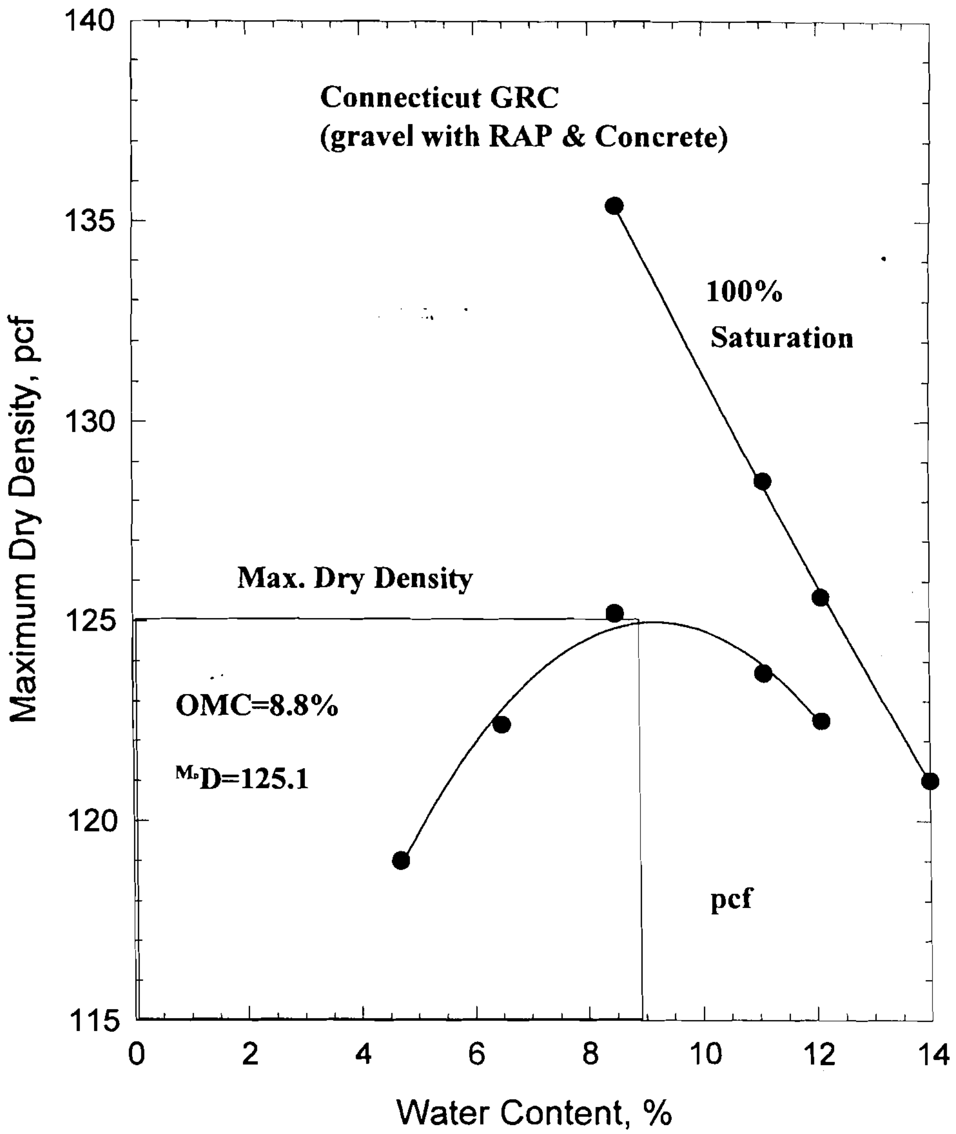


Figure 5.3 Proctor Test Results for Connecticut GRC

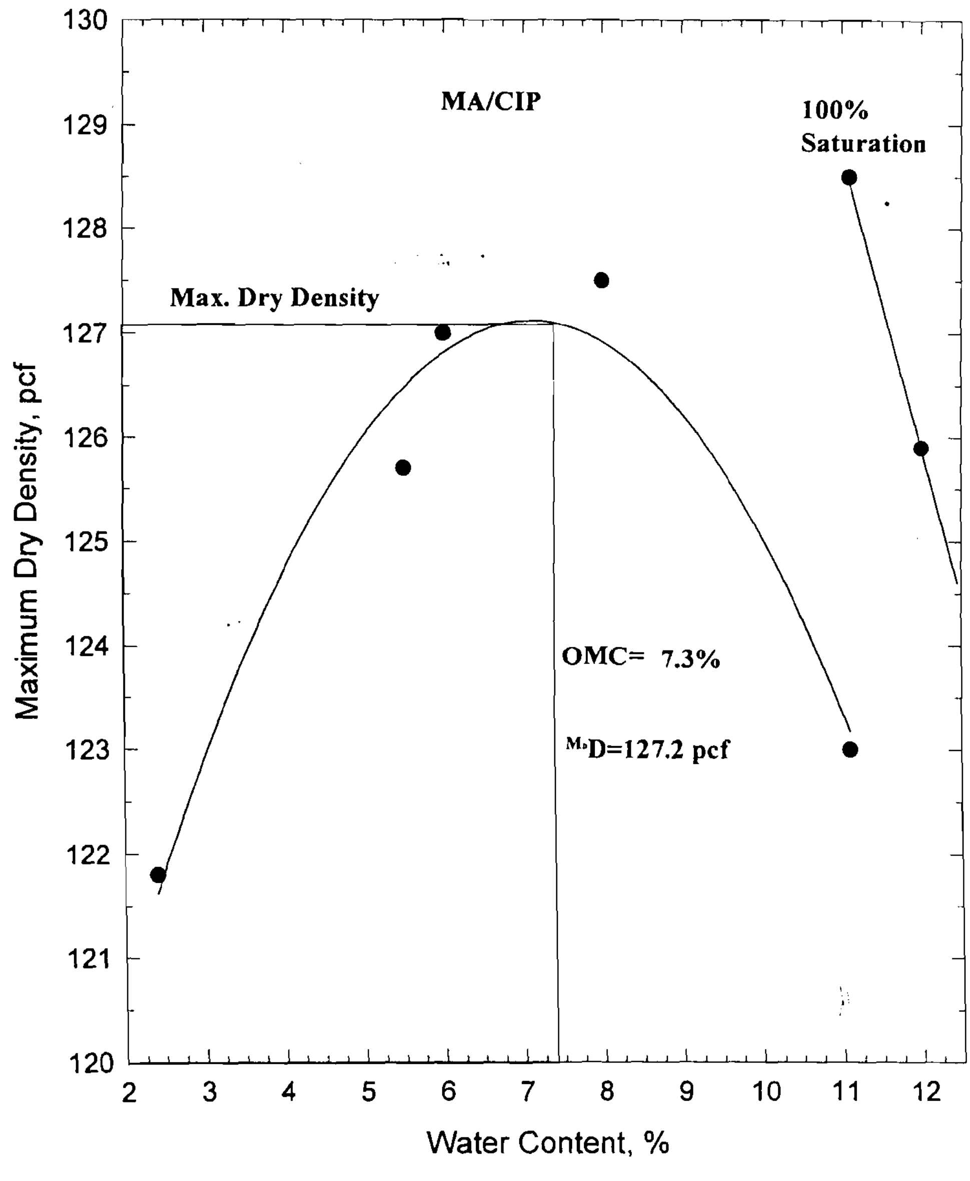


Figure 5.4 Proctor Test Results for Massachusetts RAP

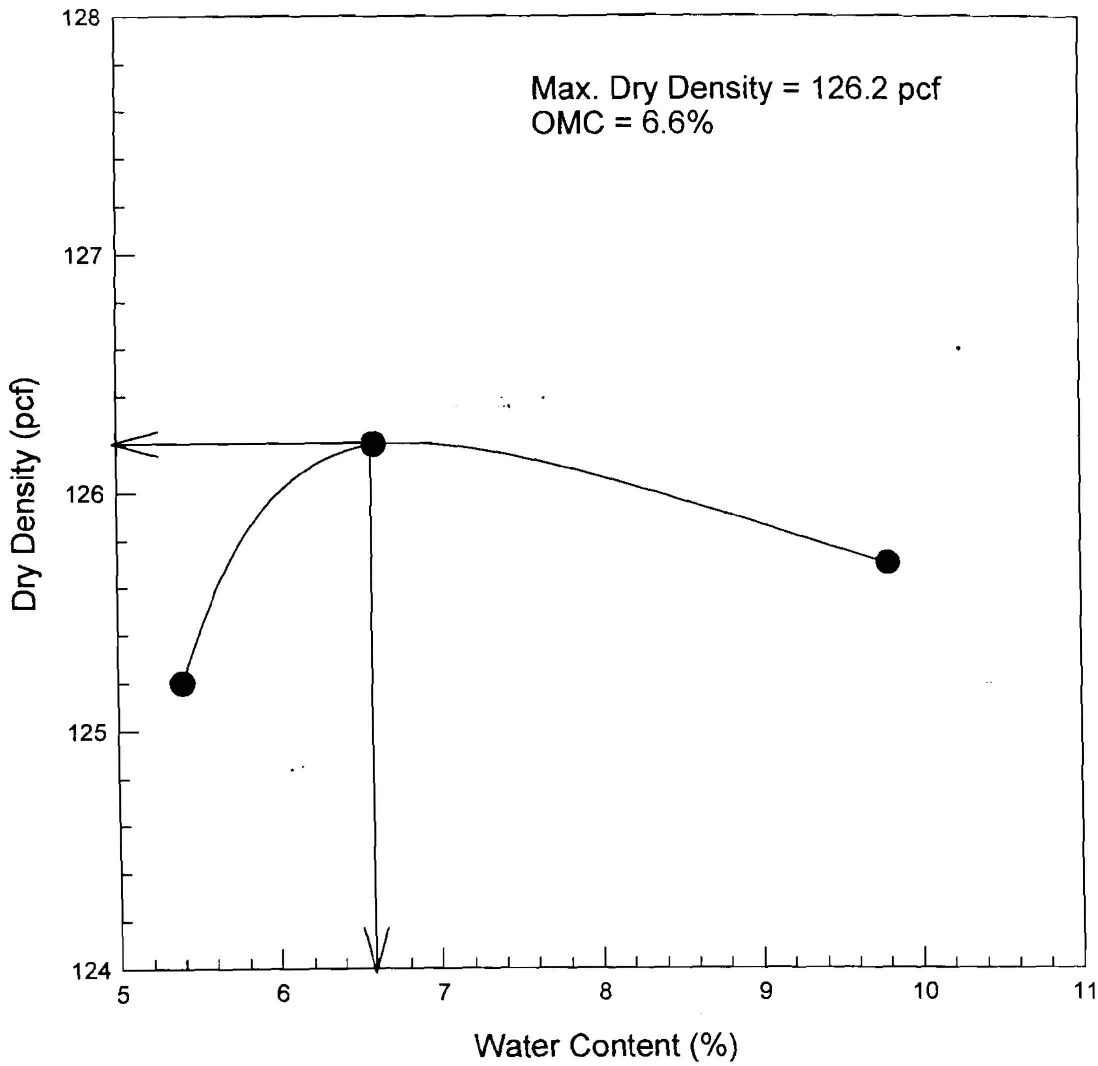
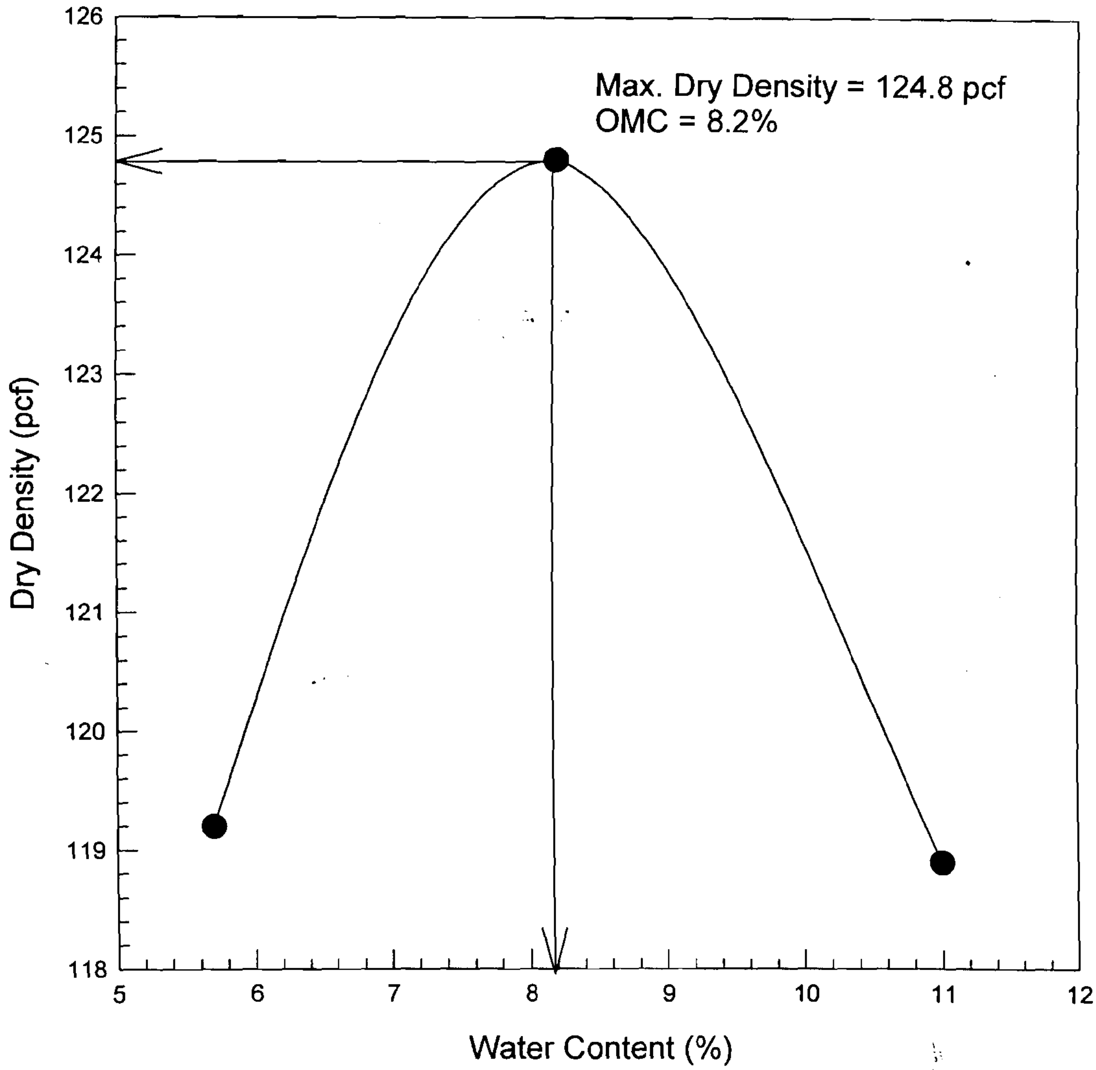


Figure 5.5 Moisture Density Relationship (40% RAP-60%PG)



Fiugre 5.6 Moisture Density Relationship (50% RAP-50%PG)

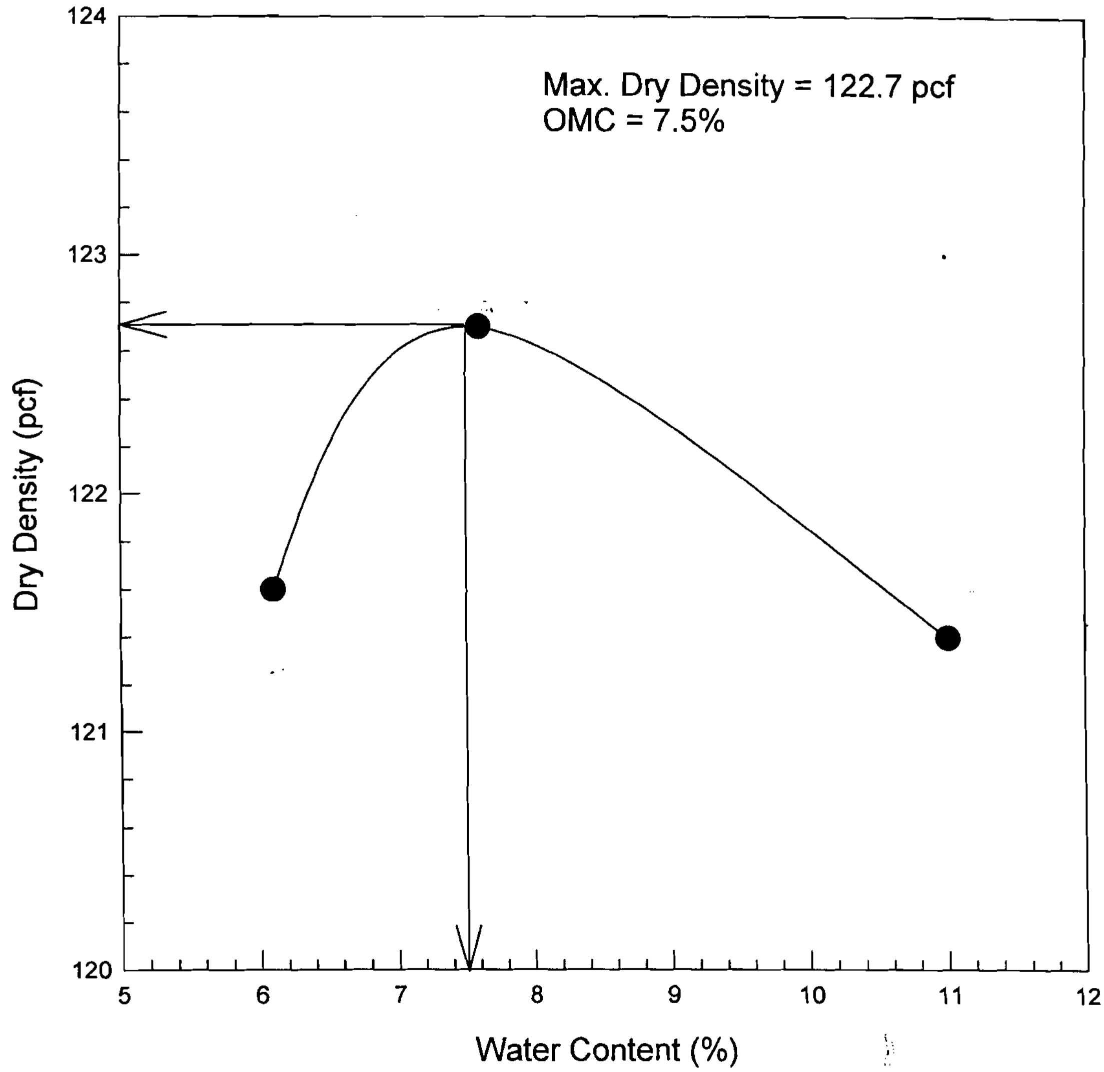


Figure 5.7 Moisture Density Relationship (60% RAP-40%PG)

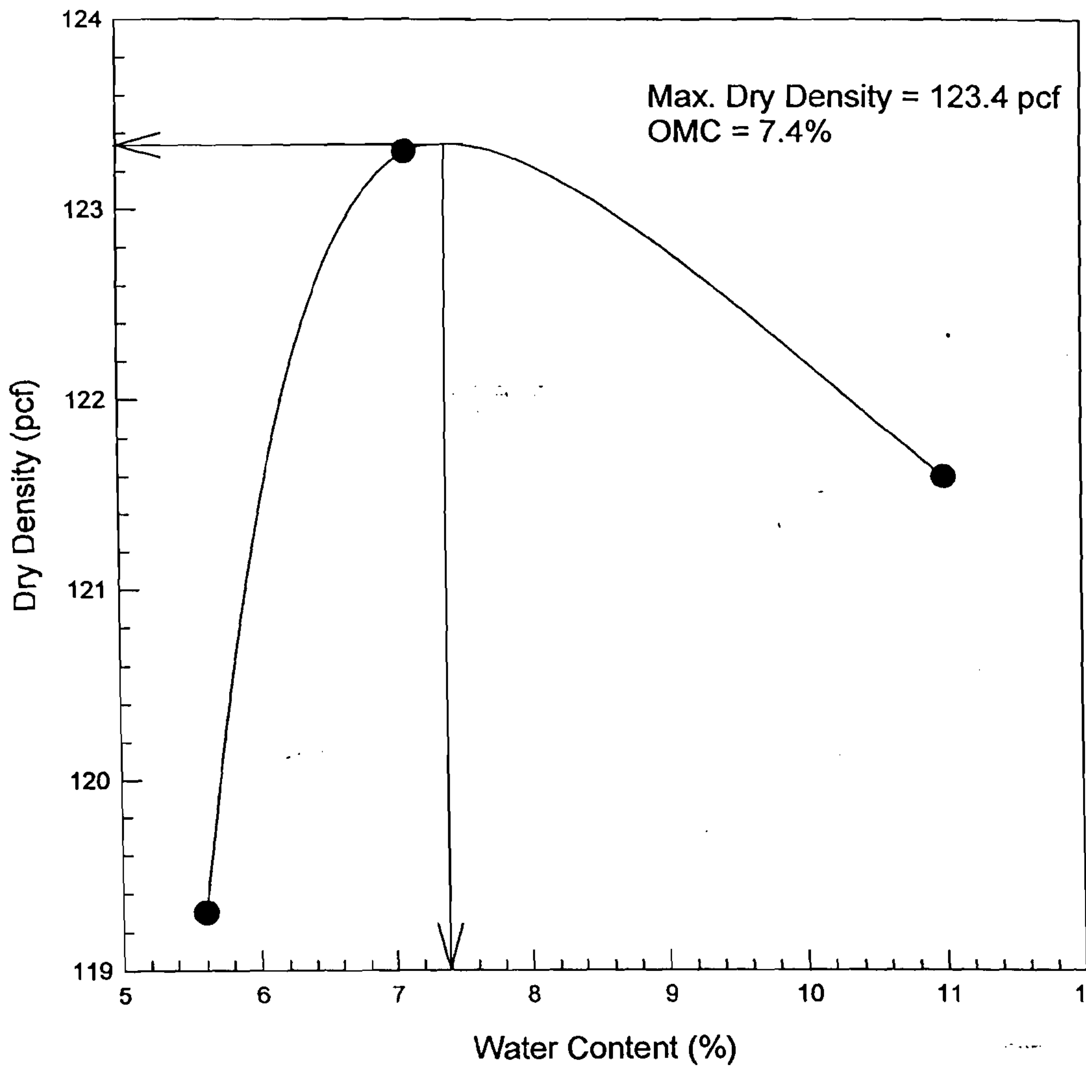


Figure 5.8 Moisture Density Relationship (70% RAP-30%PG)

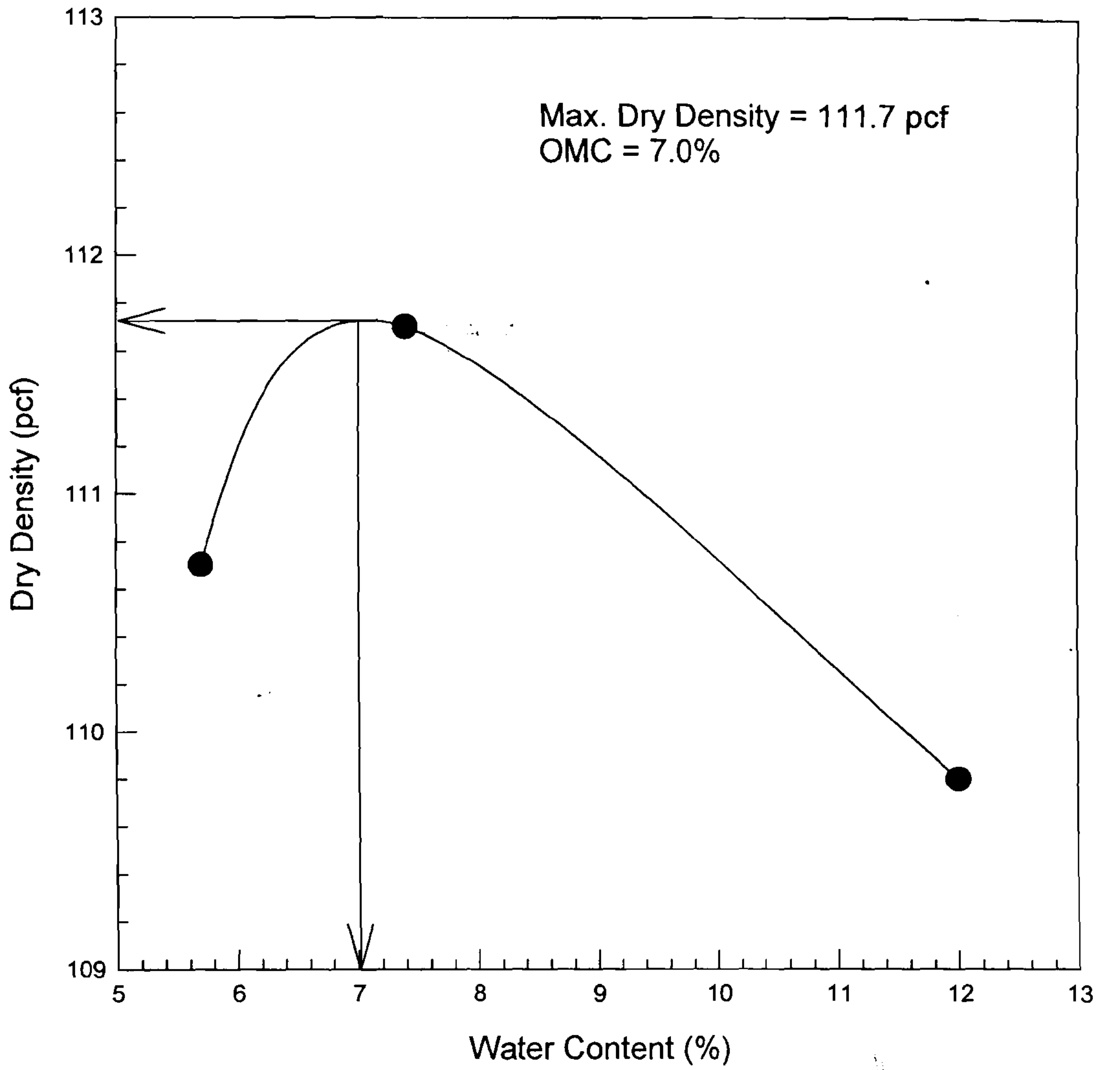


Figure 5.9 Moisture Density Relationship (100% RAP)

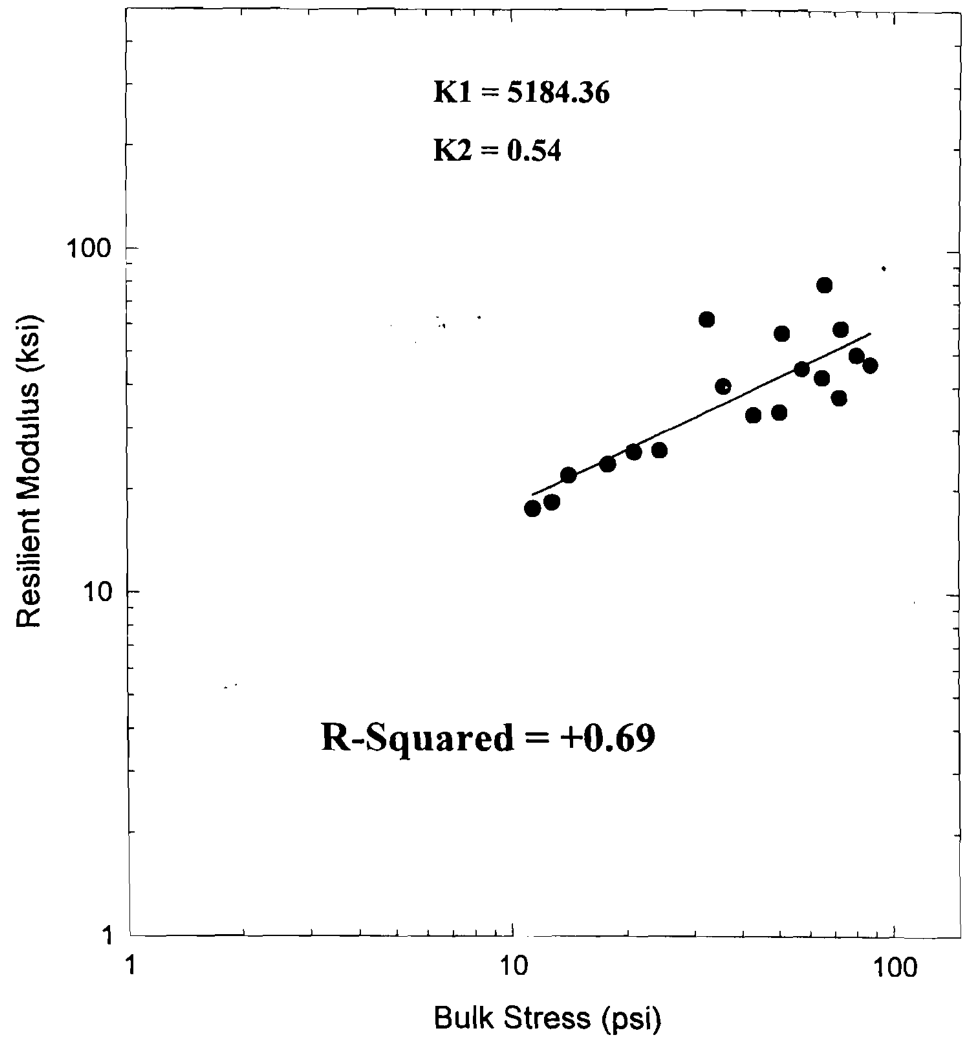


Figure 5.10 Resilient Modulus Test Results for Connecticut Gravel RAP & Portland Concrete

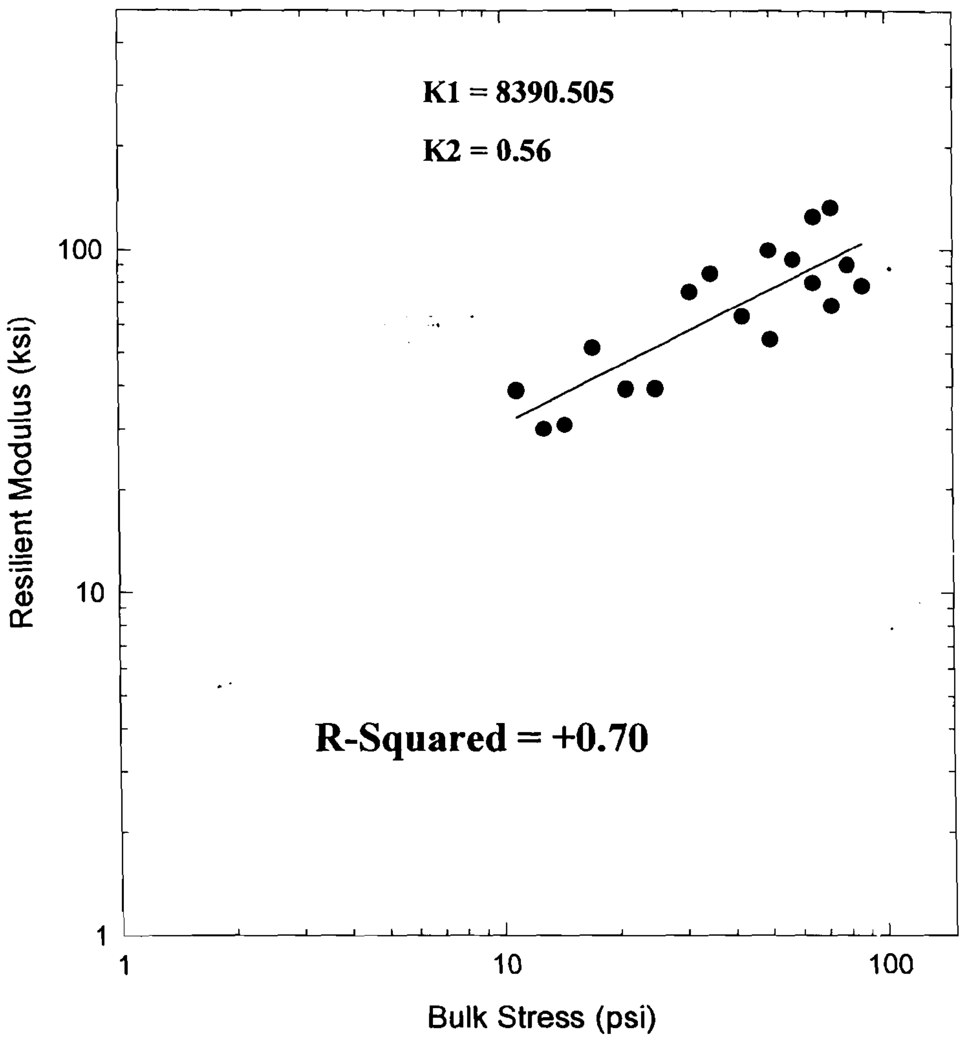


Figure 5.11 Resilient Modulus Test results for Massachusetts RAP (cold in place reclaimed)

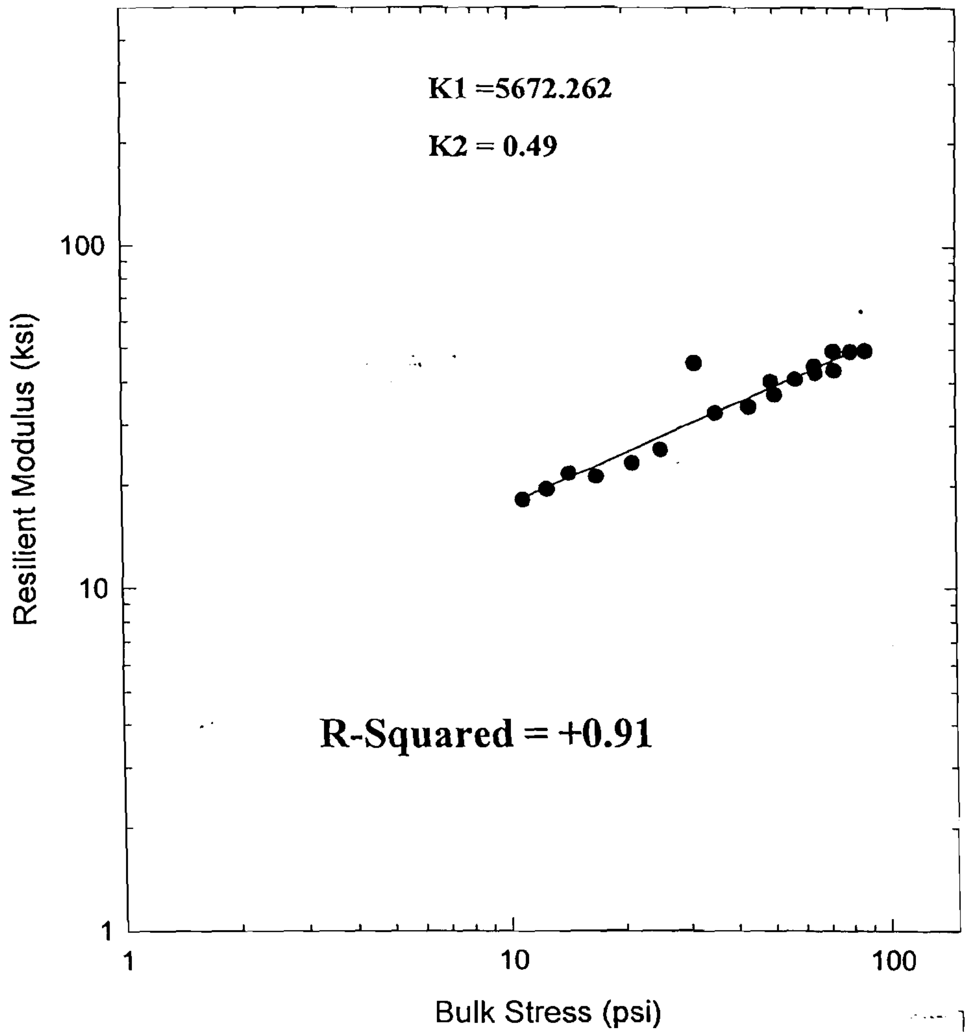


Figure 5.12 Resilient Modulus Test Results for Massachusetts Milled Asphalt(RAP)/Processed Gravel(PG) Optimization Blend: 40(RAP)/60(PG)

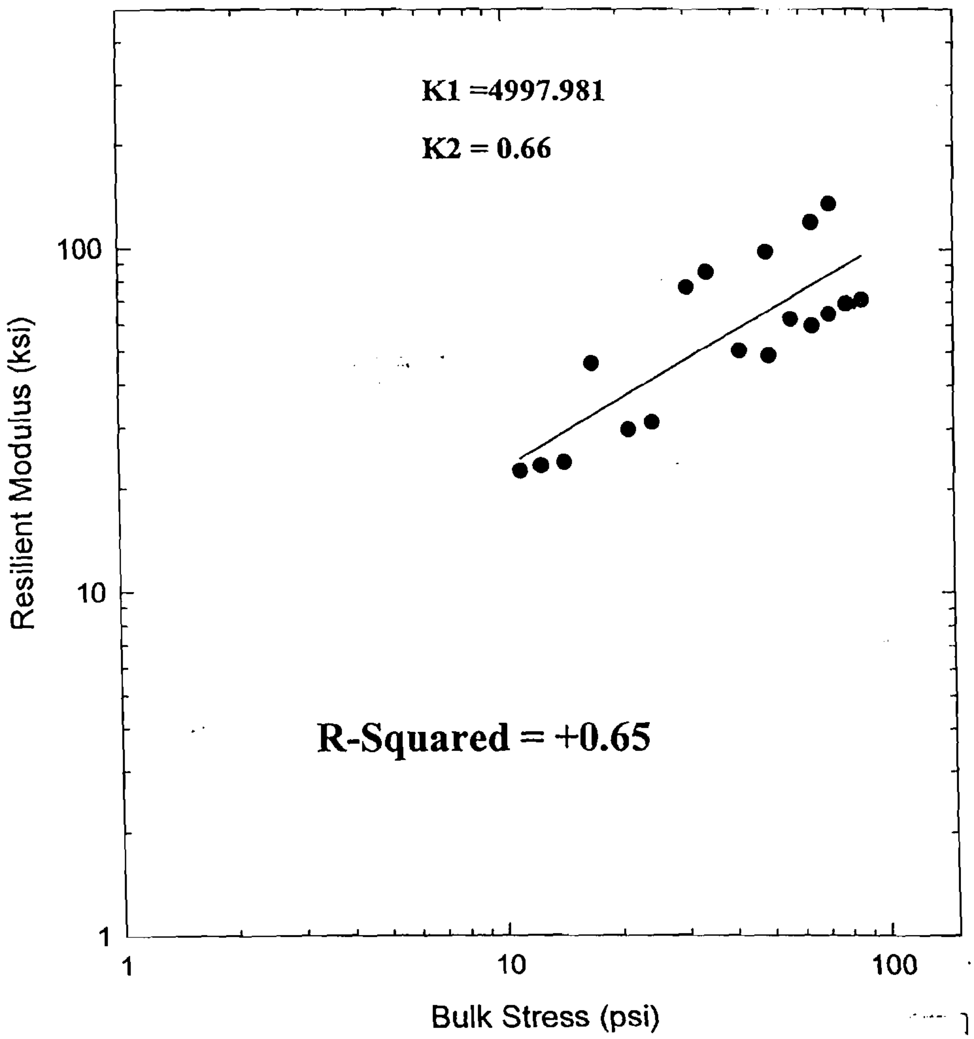


Figure 5.13 **Resilient Modulus Test Results for Massachusetts Milled Asphalt(RAP)/Processed Gravel(PG) Optimization Blend: 50(RAP)/50(PG)**

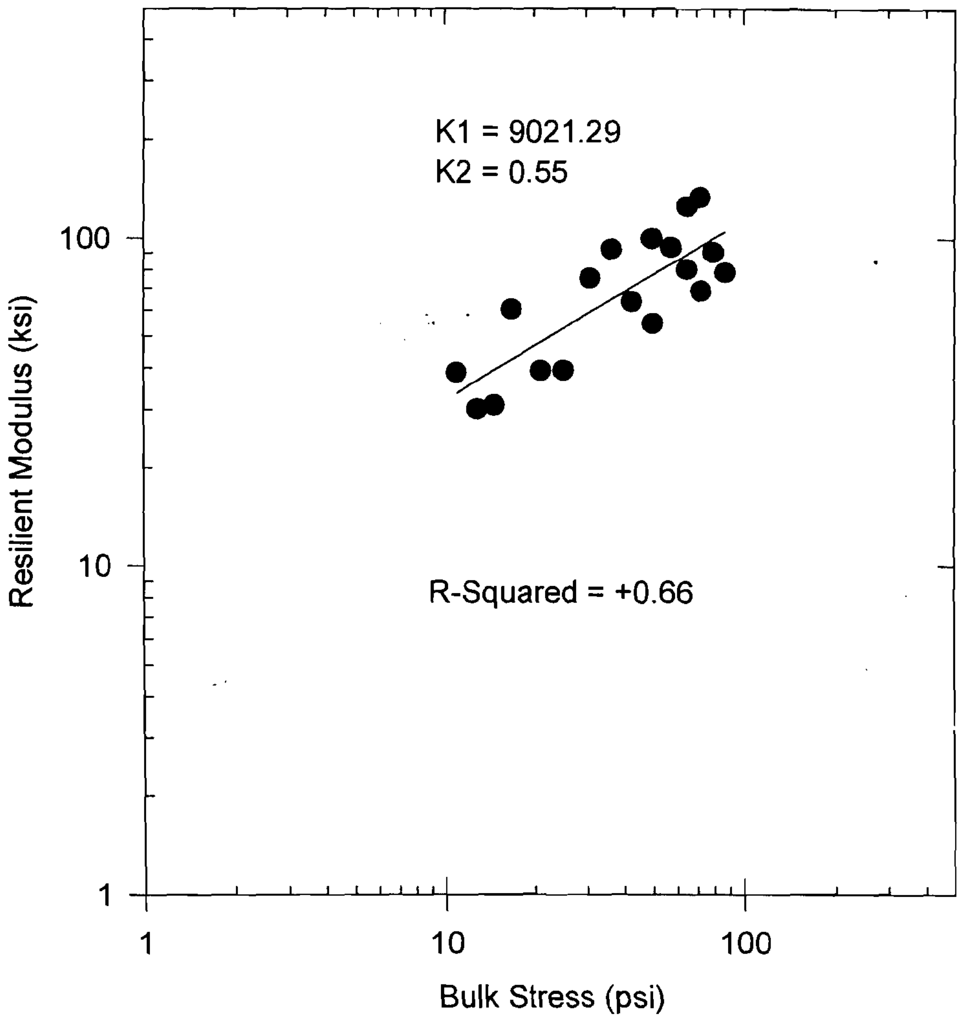


Figure 5.14 Resilient Modulus Test Results for Massachusetts Milled Asphalt(RAP)/Processed Gravel(PG) Optimization Blend: 60(RAP)/40(PG)

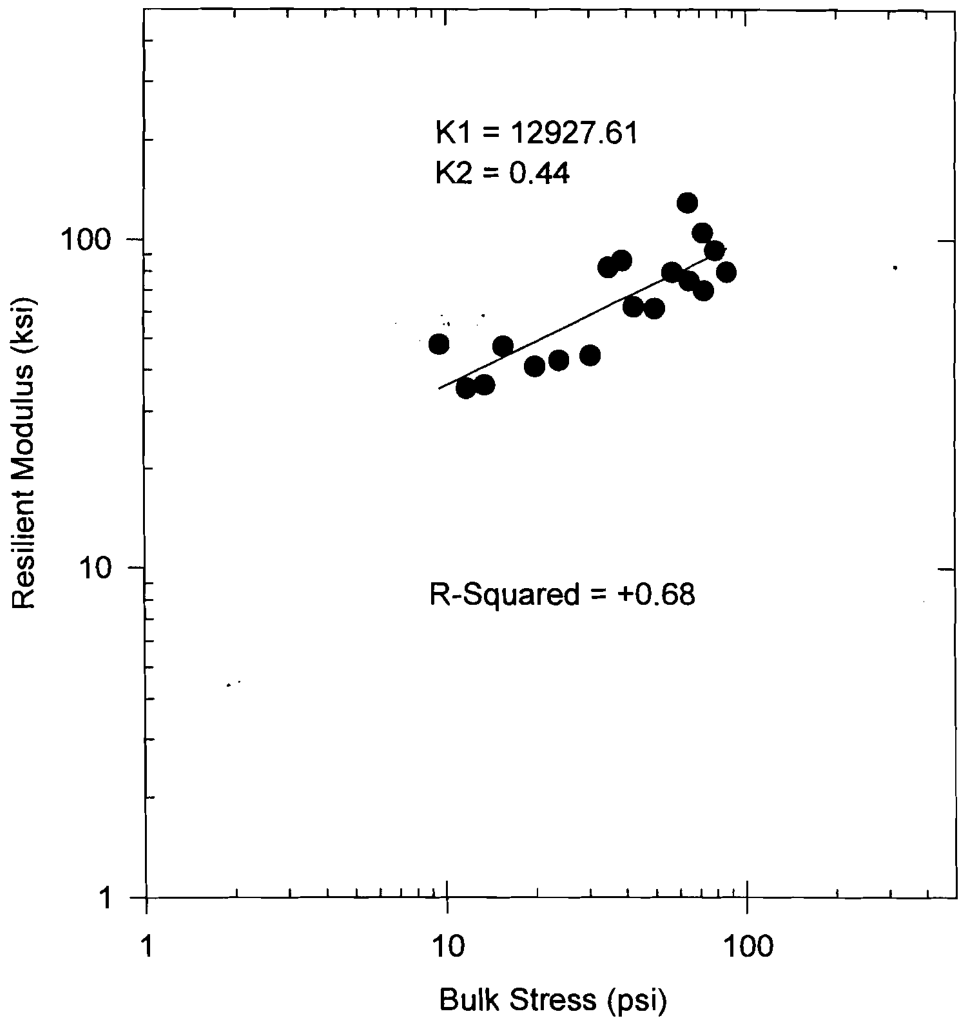


Figure 5.15 Resilient Modulus Test Results for Massachusetts Milled Asphalt(RAP)/Processed Gravel(PG) Optimization Blend: 70(RAP)/30(PG)

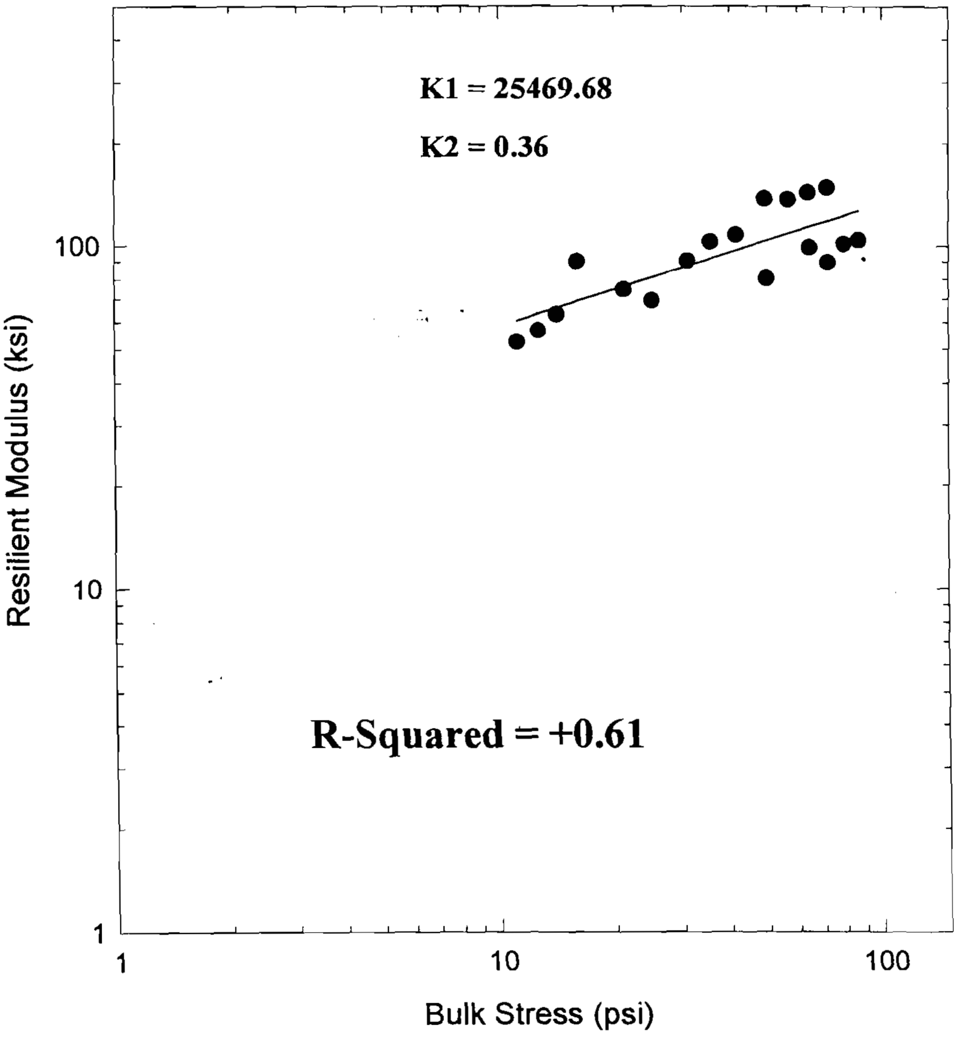


Figure 5.16 **Resilient Modulus Test Results for Massachusetts Milled Asphalt(RAP)/Processed Gravel(PG) Optimization Blend: 100(RAP)**

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Connecticut
2. MATERIAL DESCRIPTION: Gravel, RAP, and Concrete
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 10-25-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 =	9,688
K2 =	0.19056
K5 =	0.37299
R^2 =	0.86

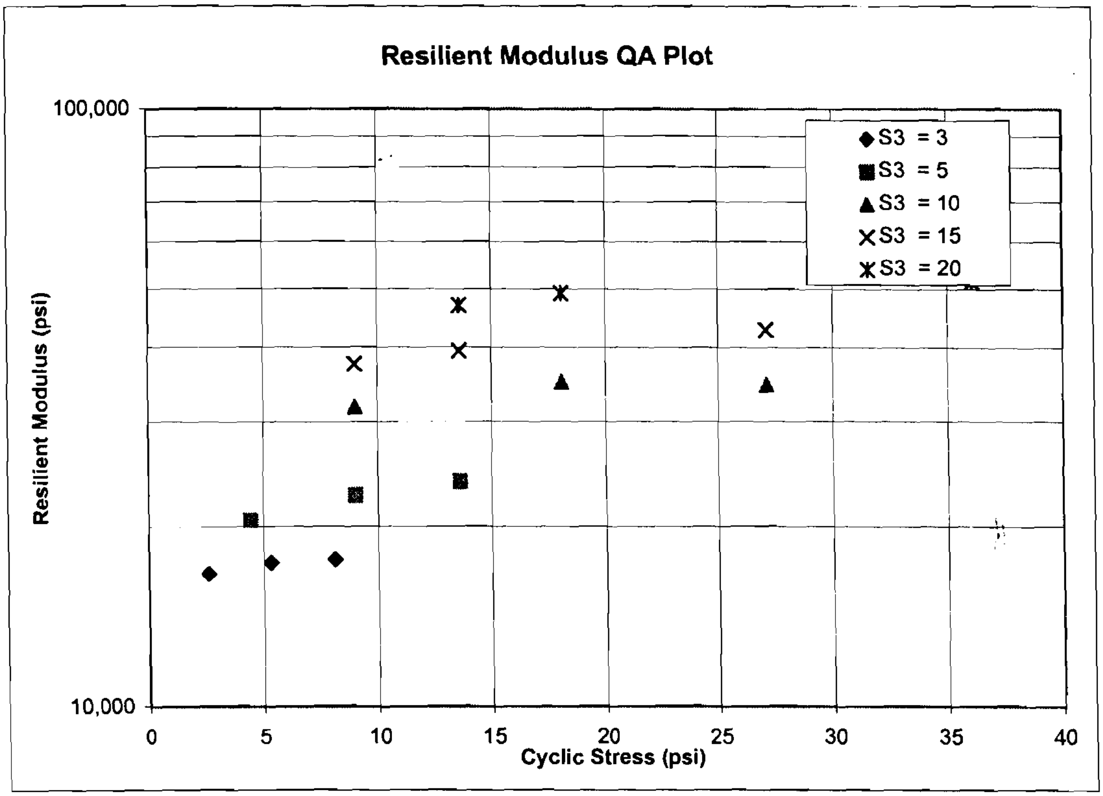


FIGURE 5.17 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Massachusetts
2. MATERIAL DESCRIPTION: Field RAP
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 10-25-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 =	13,338
K2 =	0.13115
K5 =	0.39282
R^2 =	0.83

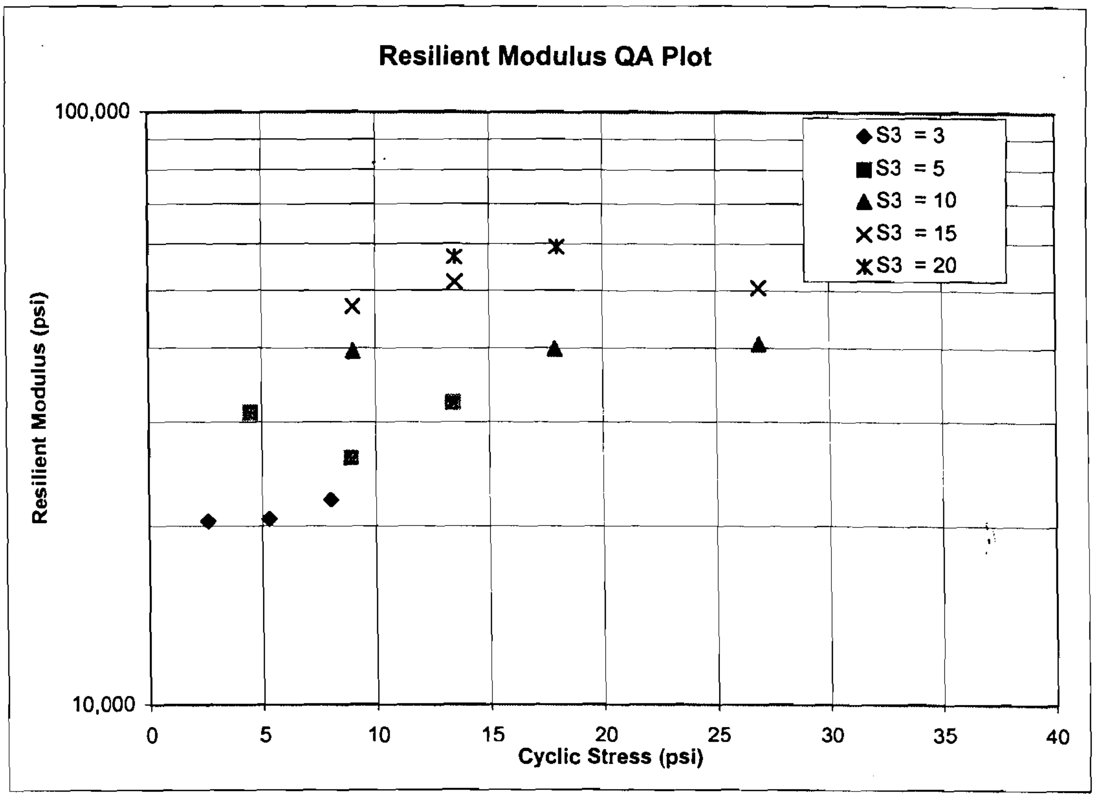


FIGURE 5.18 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC

PROJECT ID: 94-1

1. MATERIAL SOURCE: Reclaimed
2. MATERIAL DESCRIPTION: 40%RAP-60%PG
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 11-17-98

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 10,157

K2 = 0.14882

K5 = 0.39412

R^2 = 0.85

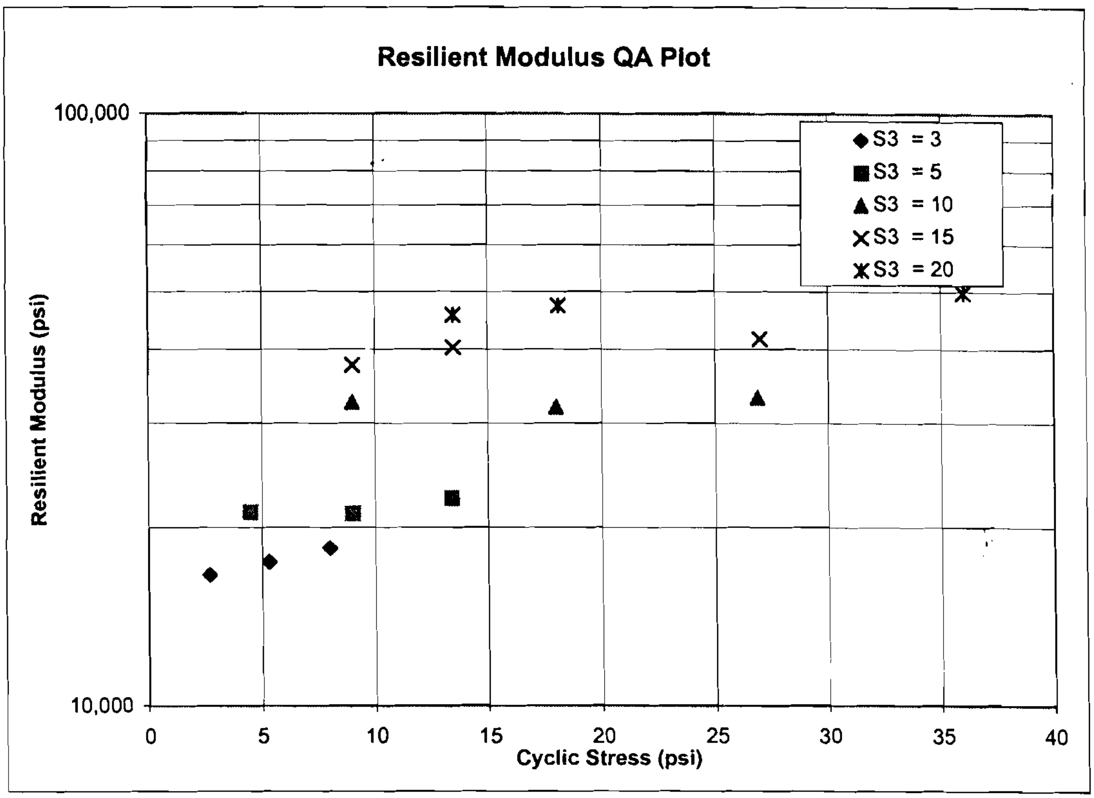


FIGURE 5.19 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Reclaimed
2. MATERIAL DESCRIPTION: 50%RAP-50%PG
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 11-18-99

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 =	5,469
K2 =	0.20697
K5 =	0.55092
R^2 =	0.80

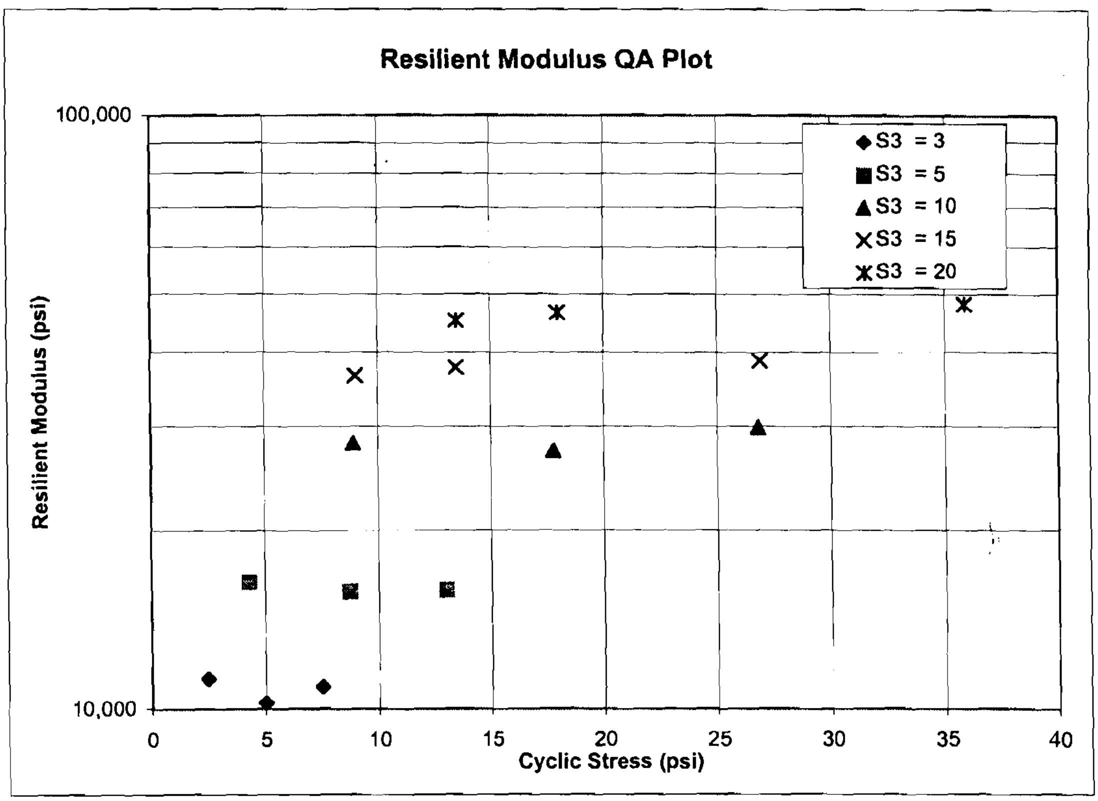


FIGURE 5.20 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Reclaimed
2. MATERIAL DESCRIPTION: 60%RAP-40%PG
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 11-18-99

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 =	11,155
K2 =	0.16936
K5 =	0.39555
R^2 =	0.84

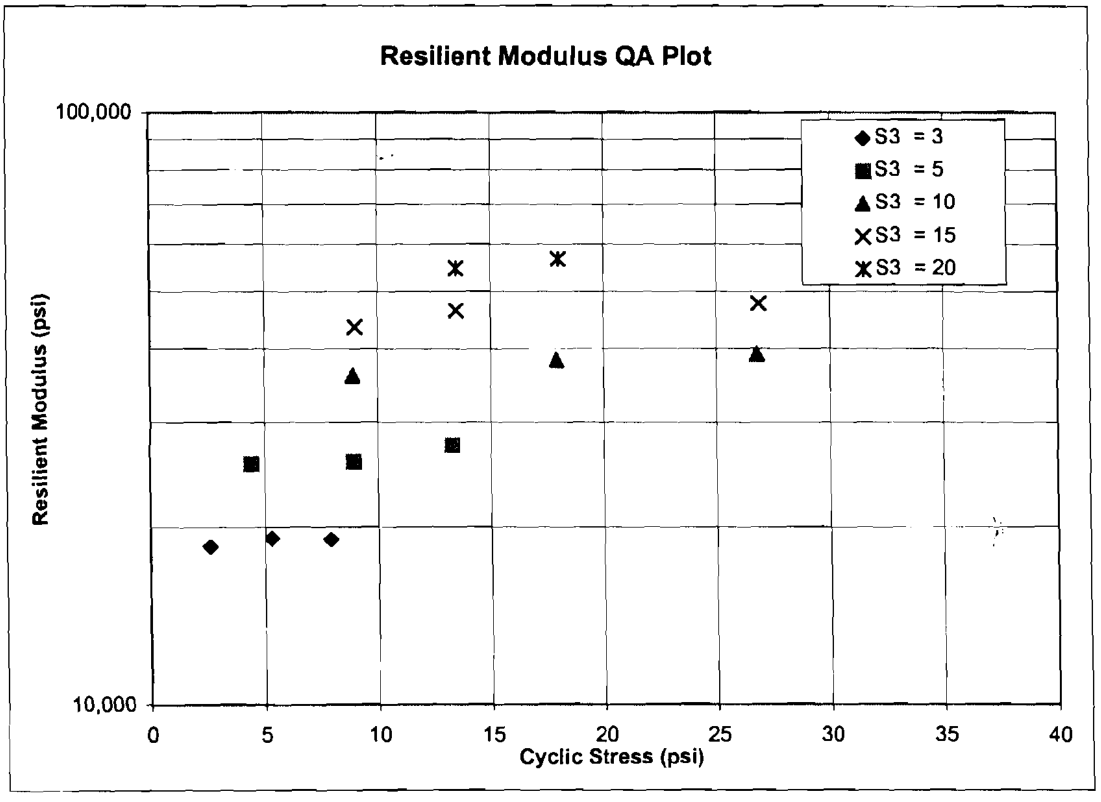


FIGURE 5.21 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME: NETC
PROJECT ID: 94-1
1. MATERIAL SOURCE Reclaimed
2. MATERIAL DESCRIPTION: 70%RAP-30%PG
3. REMOLDING TARGETS: 95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE: 1
5. TEST DATE: 11-19-99

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 =	13,888
K2 =	0.12505
K5 =	0.37286
R^2 =	0.81

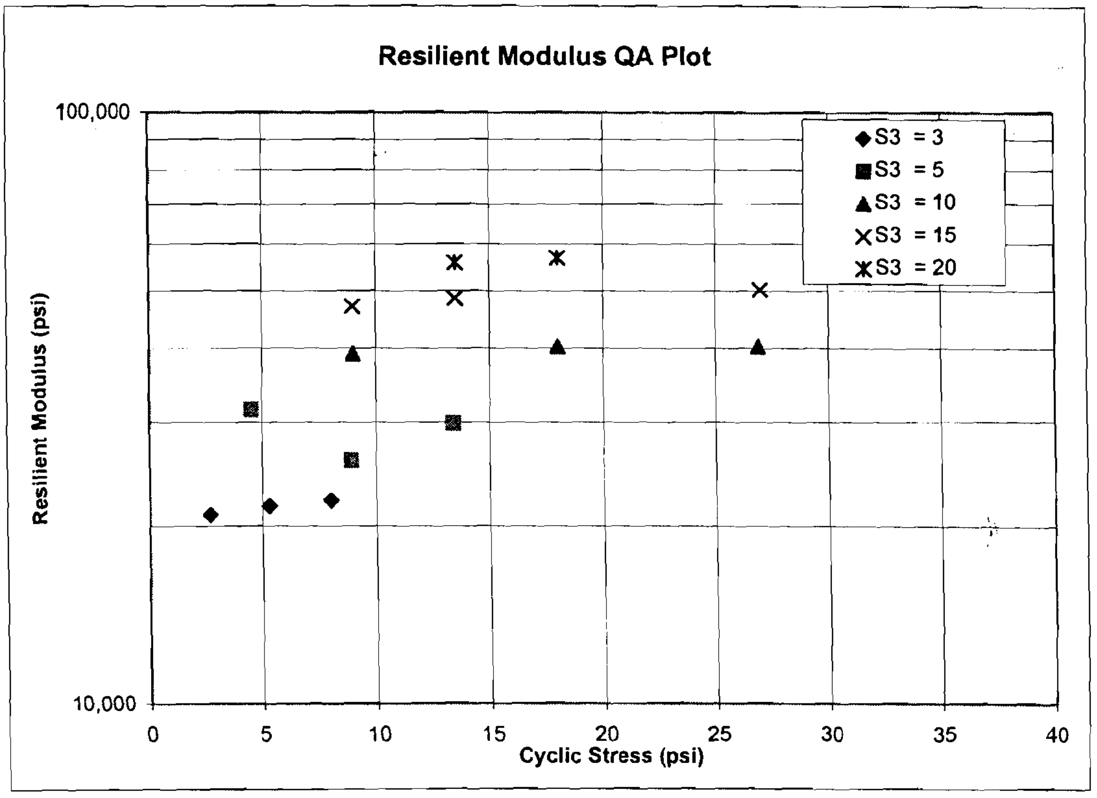


FIGURE 5.22 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

PROJECT NAME:	NETC
PROJECT ID:	94-1
1. MATERIAL SOURCE	Reclaimed
2. MATERIAL DESCRIPTION:	100%RAP
3. REMOLDING TARGETS:	95% modified Dry Density at Optimum Moisture Content
4. MATERIAL TYPE:	1
5. TEST DATE:	11-20-99

$$M_R = K1\,(S_C)^{K2}\,(S_3)^{K5}$$

K1 = 16,019
K2 = 0.10684
K5 = 0.34094
R^2 = 0.81

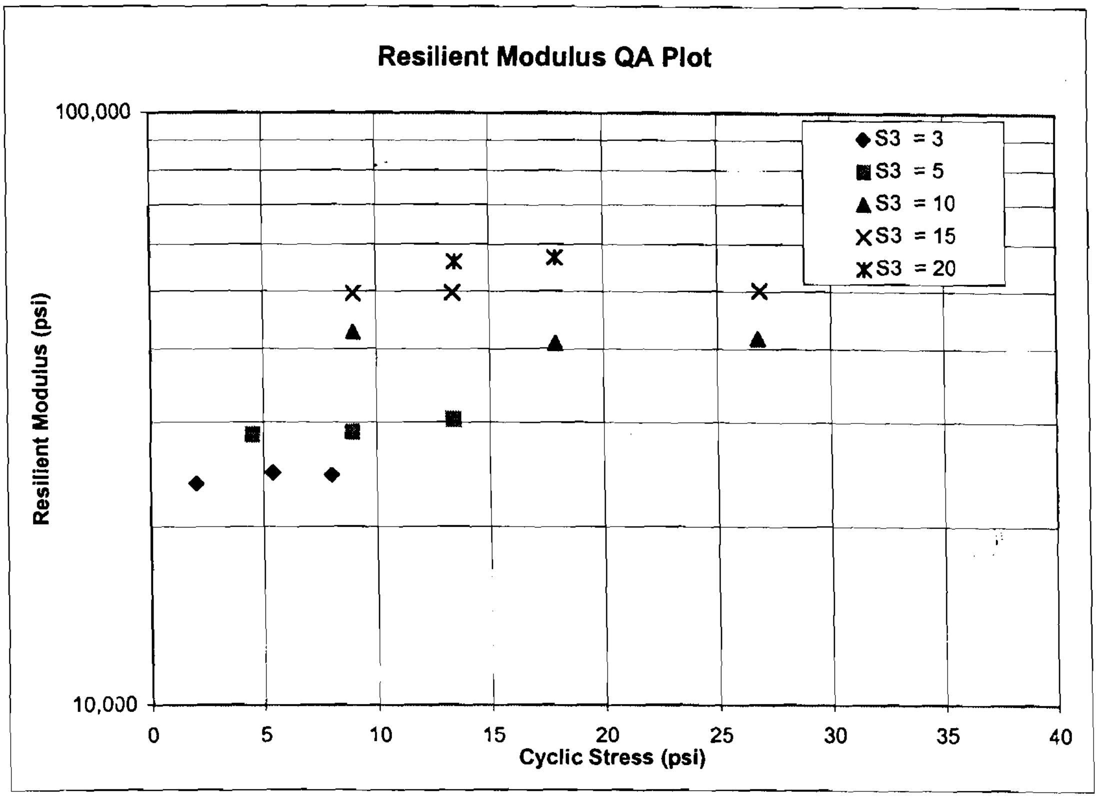


FIGURE 5.23 - Logarithmic Plot of Resilient Modulus (M_R) vs Cyclic Stress (S_C)

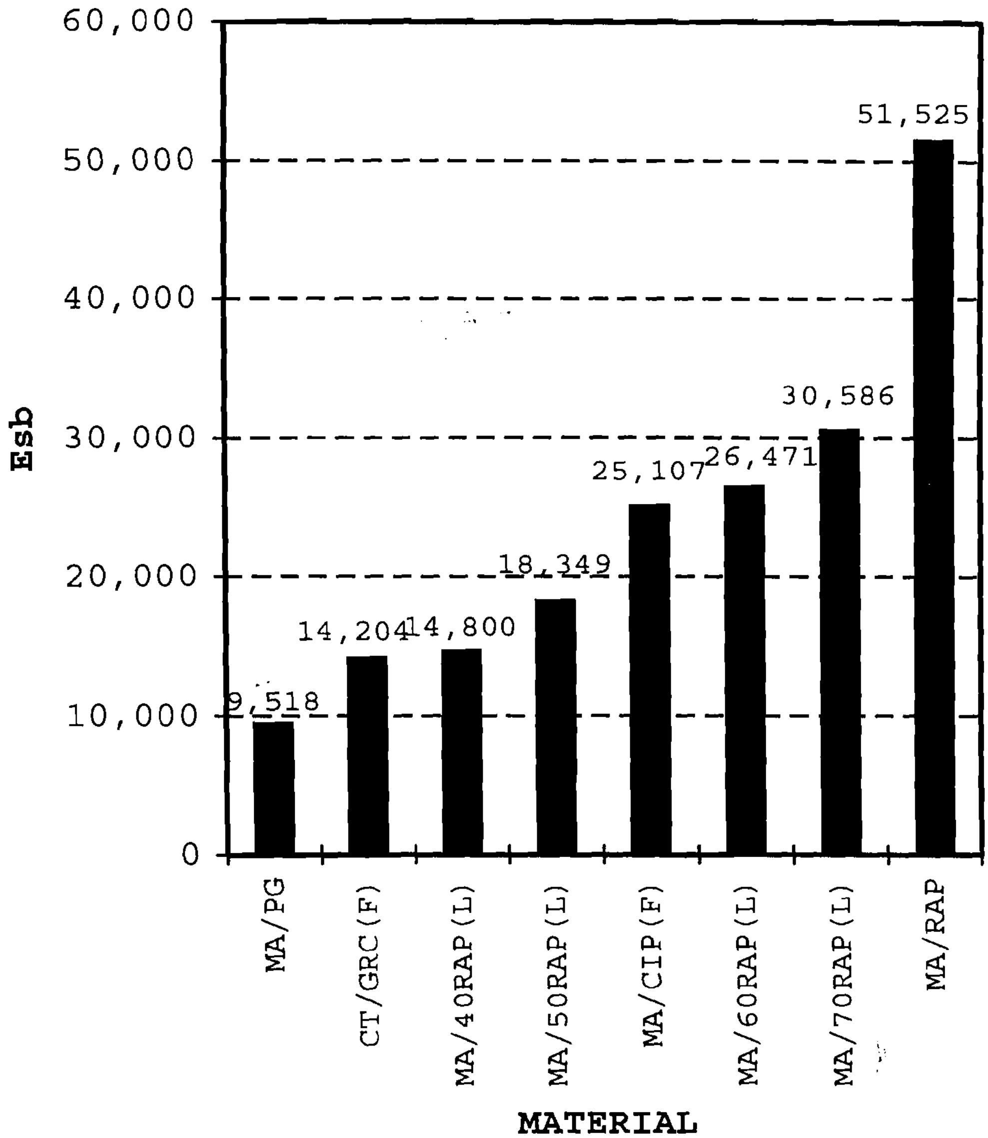


Figure 5.24 E_{sb} Comparison for Subbase Materials with Reclaimed Materials, Bulk Stress Analyzed at Mid-depth

(L) = laboratory blend

(F) = field construction blend

(M) = pure milled asphalt from MAHWD inspected stockpile

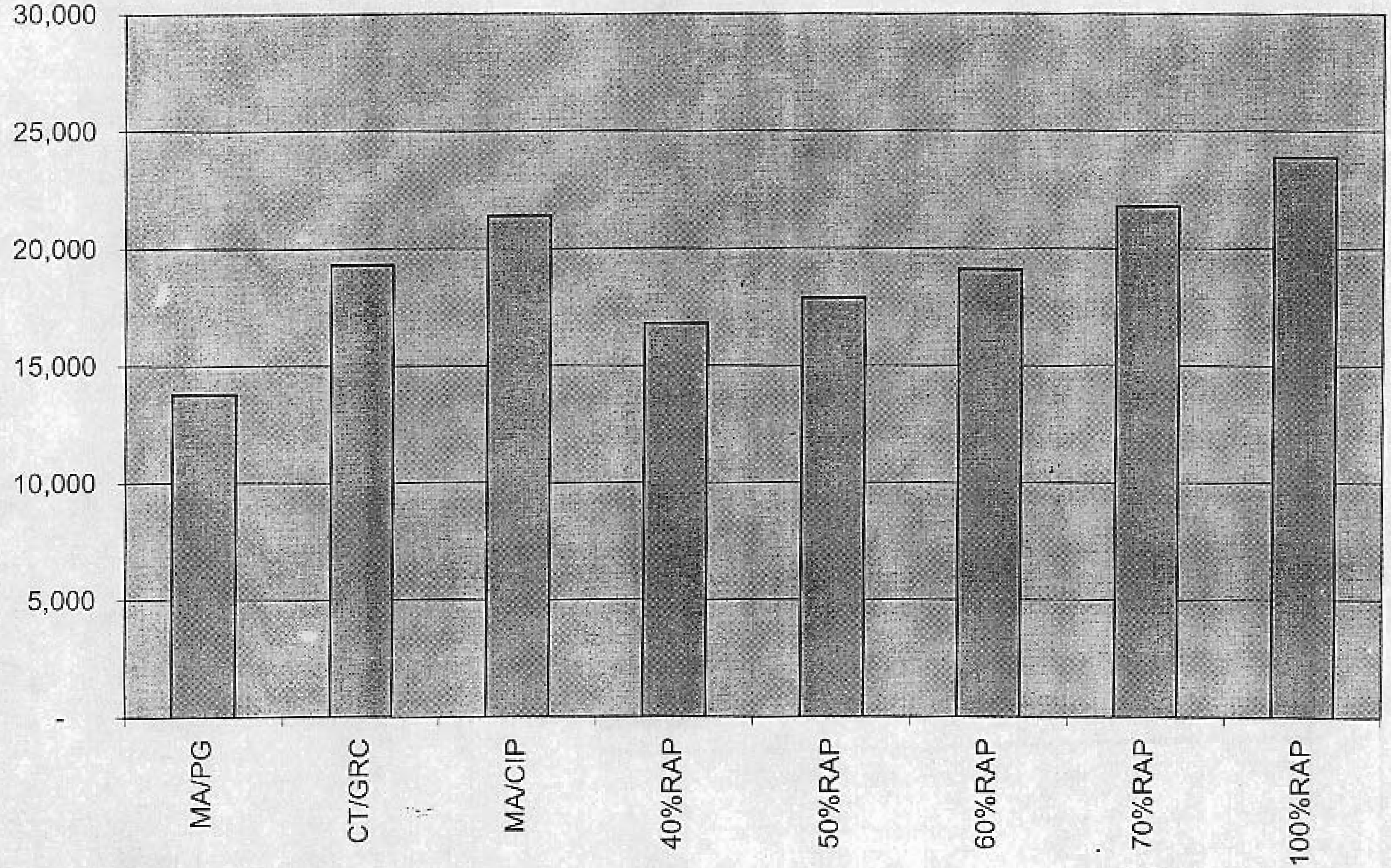


Figure 5.25 Esb Comparison for Subbase Materials with Reclaimed Materials, (Bulk Stress Analyzed at Mid-depth)

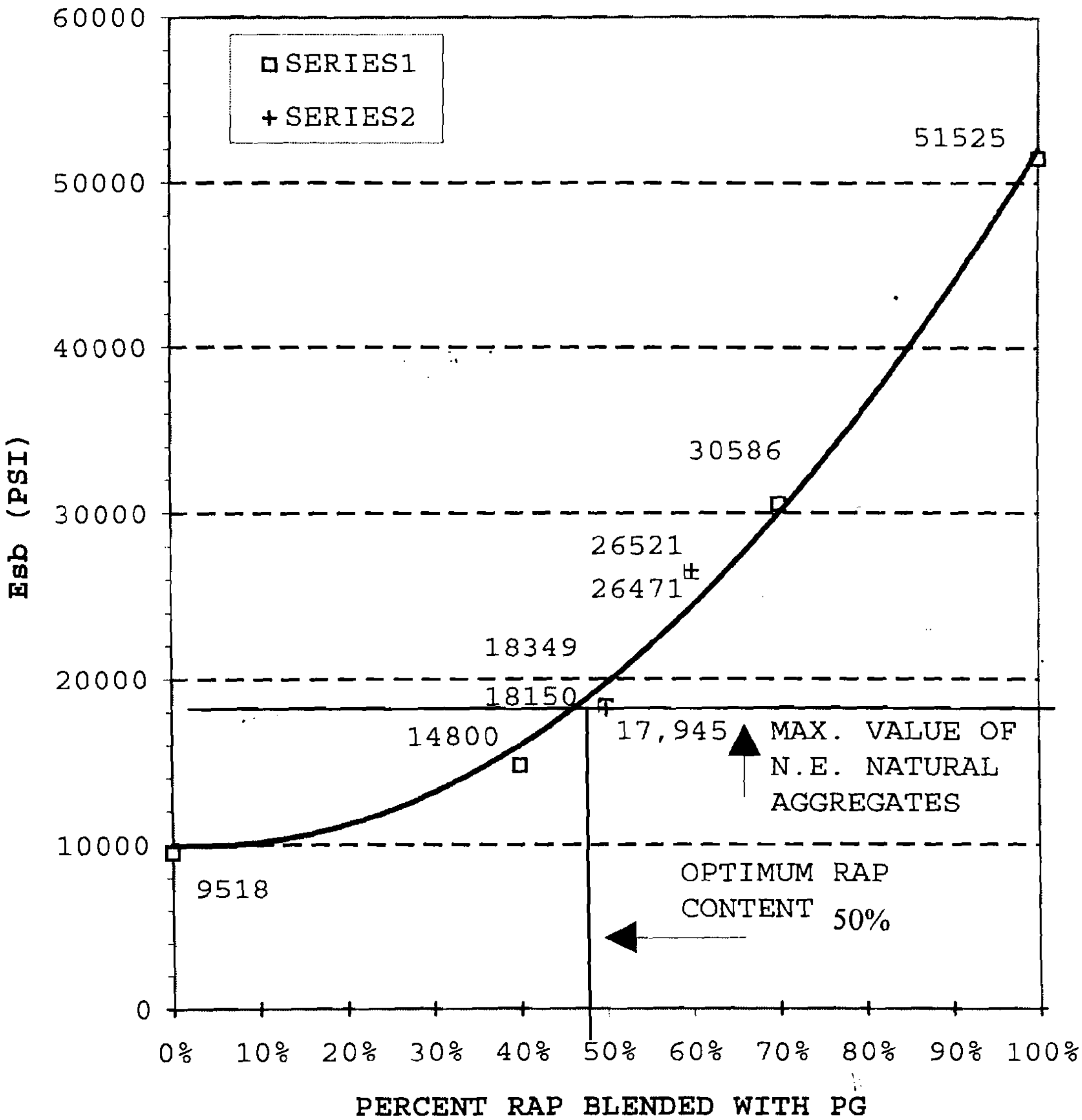


Figure 5.26 Comparison of Lab. RAP Blends Resilient Modulus

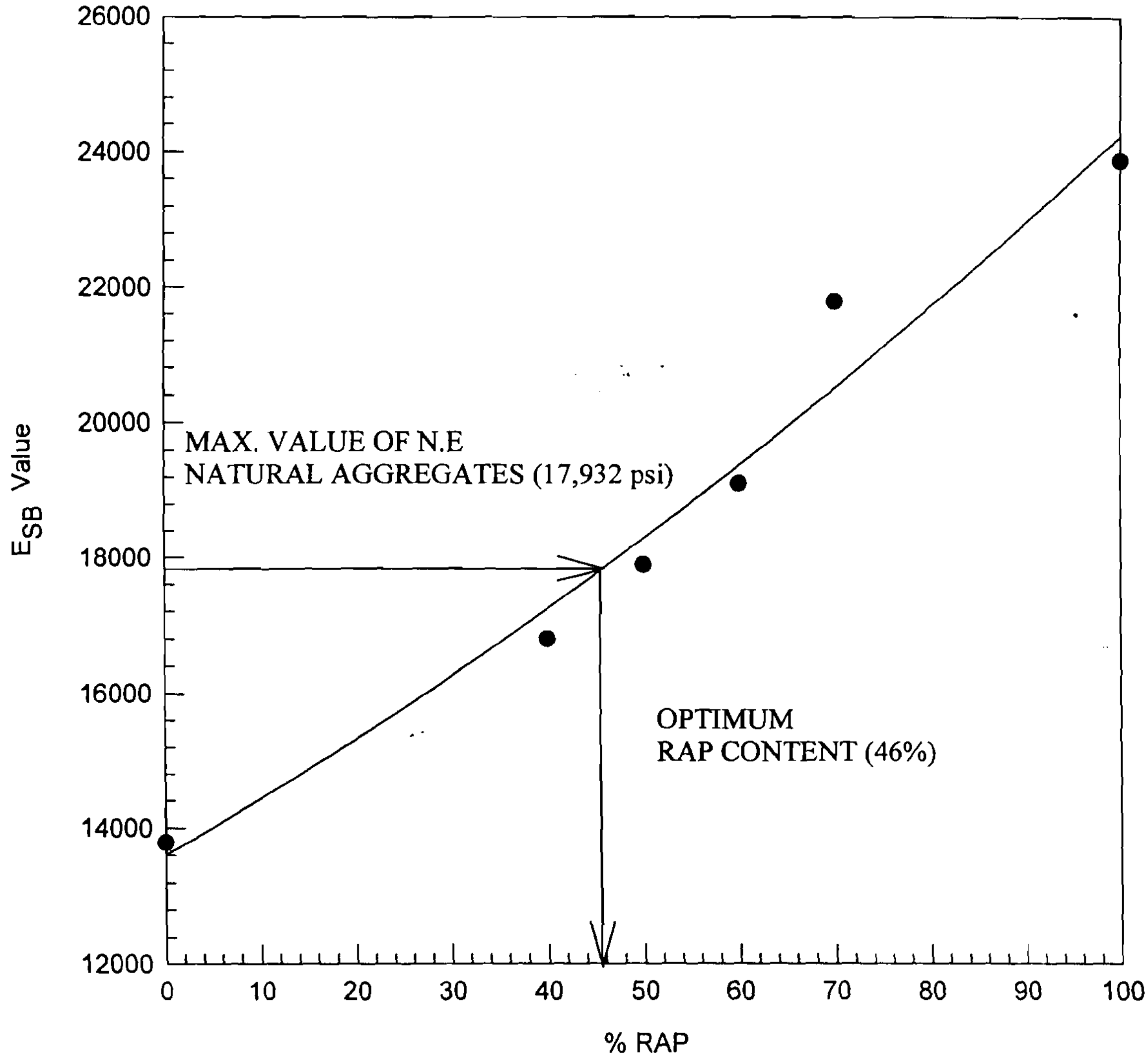


Figure 5.27 Comparison of Lab. RAP Blends Resilient Modulus (AASHTO TP46)

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

The conclusions and recommendations based on the findings and observations of this study have been summarized below.

6.1 Conclusions

(1) Typical aggregate types available for structural subbase material in New England have been identified as follows:

Connecticut:	Bank Run Gravel (CT/BRG)
Maine:	Frenchville Subbase (ME/FG)
	Sabbatus Subbase (ME/SG)
Massachusetts:	Crushed Stone (MA/CS)
	Processed Gravel (CS/CS)
New Hampshire:	Sandy Gravel (NH/SG)
Rhode Island:	Sandy Gravel (RI/SG)
Vermont:	Crushed Stone VT/CS)

(2) The resilient modulus of subbase materials (E_{SB}) was selected as the parameter for structural analysis, and AASHTO designation T292-91 was initially chosen to determine E_{SB} values for New England subbase materials. However, it was found that the AASHTO TP46-94 procedure was more appropriate for New England States agencies to determine resilient moduli of subbase materials with and without reclaimed asphalt pavement (RAP). Therefore, further testing was

performed in accordance with AASHTO TP 46-94 procedure with 150 mm x 300 mm specimens which better represent the field samples.

(3) The moduli determined in accordance with the AASHTO TP46-94 procedure have higher resilient modulus values than the ones by the AASHTO T292-91 procedure, because of the larger particle sizes and higher compaction rate.

(4) Asphalt pavement can be reclaimed in place, eliminating transportation and sorting costs. This saves energy and is usually economically feasible. All utility covers must be lowered below the level of grinding prior to full depth reclaiming. The layer coefficient for the cold in place reclaimed asphalt pavement (RAP) provided by the Massachusetts Highway Department were 0.16 and 0.14 according to AASHTO T292-91 and AASHTO TP46-94 respectively. It has a permeability in the range of a glacial till (10^{-3} to 10^{-5}). These are excellent characteristics for a subbase material used primarily for load bearing.

(5) It was observed that the mixture of gravel, RAP and Portland cement concrete (PCC) from Connecticut had higher moduli than most of the natural aggregates. The moduli of all the reclaimed blends tested were higher than the ones of virgin aggregates used in the blend. It was found that the layer coefficients of Connecticut PCC were 0.10 and 0.13 from results determined by AASHTO T292-91 and TP46-94 procedures, respectively.

(6) Layer coefficients for 40, 50 and 60 percent blended materials tested according to procedures of AASHTO T292-91 and AASHTO TP46-94 were ranged from 0.11 to 0.17 and 0.12 to 0.15, respectively. These are within ranges of 0.06 and 0.03, respectively. The layer coefficients of reclaimed material blend of 50 percent RAP were 0.07 and 0.03 higher than the ones of natural, respectively. This increase in support value from a site blended material should make reclaiming worth while.

(7) The effectiveness of the developed parameter values was demonstrated through a pavement strcture of Rt. 2 in Rhode Island using the 1993 AASHTO Guide and DARWin™ 2.01 software.

6.2 Recommendations

(1) In order to have a stable subgrade soils and avoid problems relating to placing and compacting subbase and base materials and providing adequate support for subsequent paving operations, it is recommended that the subgrade soils also be tested according to AASHTO TP46-94 procedure to establish appropriate resilient modulus inputs to design pavement structures.

(2) In place reclaiming has proven to be feasible when monitored properly. In order to maintain the specified grain size a sieve analysis must be performed on the RAP as it is being processed as well as proctor testing for OMC and MDD. In addition, testing for permeability and abrasion (wear) should be performed. Reclaimed materials have been found to degrade during the compaction process.

Additional sieve analysis after compaction is recommended. Soundness testing (sodium soundness) is recommended for PCC materials. The reclaimed material should be wind-rowed off the existing material to an optimum depth for 95 percent MDD compaction. Nuclear densometer testing may be used to check compaction. Prior to compaction all drainage considerations should be addressed based upon the particular site.

(3) Milled RAP blended with processed gravel at the URI laboratory may provide a good reference for material blending. Pure milled asphalt had the best modulus value. Pure RAP is not likely to be used for a subbase material. The pure RAP may be a better candidate for a base due to its modulus and location in the pavement structure. It may also be used for hot mix asphalt (HMA) recycling. A reclaimed aggregate blends containing 46% RAP is recommended for use as subbase material.

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