

# Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Design of Open Graded Friction Courses With Sulfur Extended Asphalt Binders		5. Report Date April, 1981	
		6. Performing Organization Code	
		8. Performing Organization Report No.	
7. Author(s) D. Saylak, K. K. Ho, Bob M. Gallaway, D. N. Little		10. Work Unit No. (TRAIS)	
9. Performing Organization Name and Address Texas Transportation Institute Texas A&M University College Station, Texas 77843		11. Contract or Grant No. DOT-FH-11-9588	
		13. Type of Report and Period Covered Final Report	
12. Sponsoring Agency Name and Address Office of Research and Development Federal Highway Administration U.S. Department of Transportation Washington, D. C. 20590		14. Sponsoring Agency Code	
15. Supplementary Notes FHWA Contract Manager - Edward Harrigan FHWA Implementation Manager - Charles Niessner			
16. Abstract <p>The combination of the anticipated shortage of asphalt cement and the projected abundance of sulfur has led to the investigation of the potential for substituting this element for the former in the paving industry.</p> <p>This research study was conducted to incorporate sulfur with asphalt to form Sulfur-Extended Asphalt (SEA) binders for use in Open Graded Friction Course (OGFC) mixtures.</p> <p>The experimental design variable included aggregate type, asphalt cement, level of sulfur contents in the binder and method of preparing SEA binders.</p> <p>The studies indicate that, with minor modifications, the existing FHWA (Federal Highway Administration) mix-design procedure may also be used for preparing SEA-OGFC mixtures. In an analysis of variance study, it was found that minimal differences existed between binders produced by direct-substitution method and emulsification. Test results also showed that SEA-OGFC system exhibited improved structural drainage and freeze-thaw properties over conventional OGFC systems.</p> <p>Based on the test results obtained in this study, a proposed SEA-OGFC mix-design procedure, patterned after the existing FHWA method, has been developed.</p>			
17. Key Words Sulfur, asphalt, sulfur-extended asphalt, open-graded, surface course, friction, design		18. Distribution Statement No restriction. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 211	22. Price

## TABLE OF CONTENTS

	Page
LIST OF TABLES . . . . .	v
LIST OF FIGURES . . . . .	vi
CHAPTER	
I. INTRODUCTION . . . . .	1
Background . . . . .	1
Objectives of the Study . . . . .	2
Scope of the Investigation . . . . .	2
II. LITERATURE SURVEY . . . . .	5
Development of OGFC's in United States . . . . .	5
Properties of Elemental Sulfur . . . . .	5
Sulfur-Asphalt Mixtures . . . . .	7
Development of SEA-OGFC's . . . . .	7
III. MIX DESIGN PROCEDURES . . . . .	11
Design Considerations . . . . .	11
Aggregate Grading and Characterization . . . . .	16
Asphalt Grades and Supplier . . . . .	21
Sulfur and Its Supplier . . . . .	21
Mixing Temperatures . . . . .	27
Mixing the SEA Binders . . . . .	27
Compaction . . . . .	28

	Page
IV. LABORATORY EVALUATION PROCEDURES . . . . .	31
Experimental Design . . . . .	31
Permeability . . . . .	31
Freeze-Thaw Durability . . . . .	35
V. DATA ANALYSIS . . . . .	37
Analysis of Variance Study . . . . .	37
Analysis of East Texas Sandstone SEA-OGFC Mixtures . . . . .	40
Analysis for Limestone SEA-OGFC Mixtures . . . . .	56
Analysis for Streetman SEA-OGFC Mixtures . . . . .	73
Selection of Optimum SEA-OGFC Mix-Design . . . . .	85
Evaluation of Low Temperature Mixes . . . . .	88
Loss of Permeability by Fouling . . . . .	89
Structural Considerations . . . . .	90
Test Sequence . . . . .	93
Results . . . . .	95
VI. CONCLUSIONS . . . . .	99
VII. RECOMMENDATIONS FOR FUTURE WORK . . . . .	101

	Page
APPENDIX A - Federal Highway Administration Design Method for Open-Graded Asphalt Friction Courses (Modified for use with Synthetic Aggregates) . . . . .	107
APPENDIX B - Safety Considerations for SEA-OGFC Mixtures . . . . .	120
APPENDIX C - Texas State Department of Highways and Public Transportation, Texas Method Designation TEX-206-F, Part II, Motorized Gyratory-Shear Molding Press Operating Procedure . . . . .	128
APPENDIX D - American Association of State Highway and Transportation Officials, Test Method T 167, Compressive Strength of Bituminous Mixtures . . . . .	133
APPENDIX E - Texas State Department of Highways and Public Transportation, Texas Method Designation TEX-126-E, Part II, Motorized Gyratory-Shear Molding Press Operating Procedure . . . . .	136
APPENDIX F - OGAF C Permeability Test Method and Calculations . . . . .	139
APPENDIX G - A Tentative Procedure for Determining Resistance of Open-Graded Friction Courses (OGFC's) to Rapidly Repeated Cycles of Freezing and Thawing in Water and Alternate Method for Test Samples, (Ref. AASHTO T 135) . . . . .	157
APPENDIX H - A Proposed Mix-Design Procedure and Laboratory Evaluation Test for Sulfur- Extended Asphalt Open-Graded Friction Courses . . . . .	163
APPENDIX I - Tabulated Test Results for SEA-OGFC Aggregate Systems . . . . .	183



## LIST OF TABLES

<u>TABLE</u>	Page
1. Survey of State Practices . . . . .	6
2. Preliminary Gradation for Limestone, East Texas Sandstone and Streetman Aggregate . . . . .	18
3. Characterization Data for Selected Program Aggregate Systems . . . . .	25
4. Asphalt Cement Properties . . . . .	26
5. Summary of Results of Factorial Analysis . . . . .	39
6. Preliminary Screening Test Results for East Texas Sandstone SEA-OGFC Mixtures . . . . .	53
7. Permeability Test Results for East Texas Sandstone SEA-OGFC Mixtures . . . . .	54
8. Freeze-Thaw Test Results for East Texas Sandstone SEA-OGFC Mixtures . . . . .	55
9. Preliminary Screening Test Results for Limestone SEA-OGFC Mixtures . . . . .	67
10. Permeability Test Results for Limestone Mixes . . . . .	68
11. Freeze-Thaw Results for Limestone Mixes . . . . .	70
12. Preliminary Screening Test Results for Streetman SEA-OGFC Mixtures . . . . .	82
13. Permeability Test Results for Streetman SEA-OGFC Mixtures . . . . .	83
14. Freeze-Thaw Test Results for Streetman SEA-OGFC Mixtures . . . . .	84
15. Properties of Optimum SEA-OGFC Mix-Design for Each Aggregate System . . . . .	86
16. Comparison Between Normal and Low Temperature Mixes .	89

	Page
17. Change in Permeability on Nacogdoches SEA-OGFC pavement after Nine Months Service Life . . . . .	90
18. Structural Test Sequence . . . . .	94
19. Approximate Slopes of Log $d_a/d_n$ versus Log $\Delta K$ curves for Open Graded Mixtures . . . . .	96

## LIST OF FIGURES

<u>FIGURE</u>	Page
1. Viscosity - Temperature Curve for Liquid Sulfur . . . . .	8
2. General Trend of Physical Properties of Sulfur-Asphalt Emulsions . . . . .	9
3. Average of Typical Gradation in Use by States for Open-Graded Asphalt Friction Courses . . . . .	14
4. Aggregate Gradation for Limestone . . . . .	22
5. Aggregate Gradation for East Texas Sandstone . . . . .	23
6. Aggregate Gradation for Streetman . . . . .	24
7. Experimental Design Variable for Evaluation of SEA-OGFC's . . . . .	32
8a. Preliminary Laboratory Evaluation Procedure for Each Aggregate . . . . .	33
8b. Procedure for Laboratory Evaluation of Structural and Performance Characteristics for Each Aggregate Mixture . . . . .	34
9. Four-Factor Factorial Analysis . . . . .	38
10. Marshall Stability vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	41
11. Marshall Stability vs. AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures . . . . .	41
12. Marshall Flow vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	42
13. Hveem Stability vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	42
14. Hveem Stability vs. AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures . . . . .	43
15. Resilient Modulus at 34°F (1°C) vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	43

16. Resilient Modulus at 34°F (1°C) vs. AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures . . . . .	44
17. Resilient Modulus at 68°F (20°C) vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	44
18. Resilient Modulus at 68°F (20°C) vs. AC Grade with Corresponding Sulfur Content East Texas Sandstone SEA-OGFC Mixtures . . . . .	45
19. Resilient Modulus at 77°F (25°C) vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	45
20. Resilient Modulus at 77°F (25°C) vs. AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures . . . . .	46
21. Air Voids vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures. . . . .	46
22. Voids in Mineral Aggregate vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	47
23. Index of Retained Strength vs. Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures . . . . .	47
24. Marshall Stability vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . . . . .	57
25. Marshall Stability vs. AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures . . . . .	57
26. Marshall Flow vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . . . . .	58
27. Hveem Stability vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . . . . .	58
28. Hveem Stability vs. AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures . . . . .	59
29. Resilient Modulus at 34°F (1°C) vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . .	59

30. Resilient Modulus at 34°F (1°C) vs. AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures . . . . .	60
31. Resilient Modulus at 68°F (20°C) vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . . . . .	60
32. Resilient Modulus at 68°F (20°C) vs. AC Grade with Corresponding Sulfur Content for SEA-OGFC Mixtures . . . . .	61
33. Resilient Modulus at 77°F (25°C) vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . . . . .	61
34. Resilient Modulus at 77°F (25°C) vs. AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures . . . . .	62
35. Air Voids vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . . . . .	62
36. Voids in Mineral Aggregates vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures . . . . .	63
37. Index of Retained Strength vs. Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixture . . . . .	63
38. Marshall Stability vs. Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures. . . . .	74
39. Marshall Stability vs. AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures . . . . .	74
40. Marshall Flow vs. Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures . . . . .	75
41. Hveem Stability vs. Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures . . . . .	75
42. Hveem Stability vs. AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures . . . . .	76
43. Resilient Modulus at 34°F (1°C) vs. Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures. . .	76

	Page
44. Resilient Modulus at 34°F (1°C) vs. AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures . . . . .	77
45. Resilient Modulus at 68°F (20°C) vs. Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures . . . . .	77
46. Resilient Modulus at 68°F (20°C) vs. AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures . . . . .	78
47. Resilient Modulus at 77°F (25°C) vs. Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures . . . . .	78
48. Resilient Modulus at 77°F (25°C) vs. AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures . . . . .	79
49. Air Voids vs. Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures . . . . .	79
50. Typical Layered Elastic Pavement Section Used to Evaluate Tensile Strains in OFGC . . . . .	92
51. Permanent Deformation Versus Load Cycles . . . . .	98

## CHAPTER I

### INTRODUCTION

#### Background

With the rising number of vehicles on the road, it is not surprising that there is a corresponding rise in the number of vehicular accidents. One of the major factors contributing to these accidents is skidding on a wet pavement surface. During periods of rain, a film of water develops on the surface of a conventional pavement. This film of water can, at high vehicle speeds, create a loss of contact between the tire and the pavement surface, which adversely affects skid resistance and thus adds to the hazards of wet weather driving. In a special study conducted by the National Transportation Safety Board, an independent investigative agency, it was found that 13.5% of all fatal highway accidents during 1976 and 1977 occurred on wet and thus skid susceptible pavements, while precipitation occurred only about three percent of the year throughout the nation. The National Transportation Safety Board report concluded that fatal accidents on wet pavements occur 3.9 to 4.5 times more often than might be expected (1). Therefore, it is in the best interest of the pavement industry to devise a pavement surface that can eliminate the surface water during periods of rain.

To this end, porous asphalt pavements have been designed and are currently in use which allow rainfall to flow through the top layer to the roadway shoulder rather than remain on the pavement surface. Among the names given to such surfaces are "plant-mix seal", "popcorn mix" and "open-graded friction course". In this report, we will use the term "open-graded friction course" (OGFC) as has been proposed by the Federal Highway Administration (2) to describe such pavements. Forty-nine states in the United States, as well as several foreign countries, have constructed such surface courses, either on an experimental basis or as a standard practice, (3, 4, 5, 6, 7, 8). In particular, Texas has experimented with OGFC's for a number of years in several districts (9) and their construction methodology is now operational. A number of desirable benefits were recognized through these studies and they include (10):

1. Reduction in quantities of materials needed to provide desirable skid-resistance characteristics.
2. Facilitation of the repair of minor surface irregularities.
3. Reduction or elimination of tire splash and spray.

4. Improved visibility of painted markings during wet weather.
5. Production of quieter riding surfaces.
6. Improved drainage of surface water to roadway shoulder.

This type of pavement surface has the potential to significantly reduce wet weather skid accidents on highway pavements. Therefore, OGFC's will continue to play an important role in the construction of skid-resistant pavements. Construction of OGFC's to date have only used conventional asphalt binders. The dependence on foreign sources for the supply of asphalt materials, coupled with the significant increases in the price of asphalt during the last few years, have led to the investigation of alternate, domestic materials to reduce the demand for asphalt cement. Research sponsorship in this area has been shared by The Sulphur Institute, Societe Nationale Elf Aquitaine (SNEA), Shell Canada Limited, Gulf Oil Limited of Canada, the Federal Highway Administration, the U. S. Bureau of Mines and the Texas Transportation Institute (TTI). Research studies conducted by these agencies have led to the recognition of sulfur as having an excellent potential for use in road-way construction. In 1978 world stockpiles of sulfur were estimated to be 26 million metric tons and were increasing due, primarily, to the recovery of sulfur from pollution-abatement processes in power plants and sour gas wells (11). It is expected that the supply should exceed the demand around the year 1985. The combination of the anticipated shortage of asphalt cement and the projected abundance of sulfur have helped to justify the practicality of substituting the latter product for the former.

#### Objectives of the Study

The major objectives of this research study were to develop criteria for material selection, establish mixture design guidelines, and to develop evaluation criteria for sulfur-extended asphalt-open-graded friction courses (SEA-OGFC).

#### Scope of the Investigation

The objectives of this study were accomplished by means of the five major tasks listed below:

- Task 1. Survey of the literature for existing design methods.
- Task 2. Selection of a reliable and logical mixture design method to form the basis of proposed work.



Task 3. Establish optimum mixture design procedures for SEA-OGFC materials.

Task 4. Establish evaluation criteria using appropriate tests on compacted mixes.

Task 5. Recommend safety considerations.

Task 1 was a literature survey of existing design and construction methods that indicated the unique properties of sulfur-modified paving mixtures compared to conventional mixtures, and provided some basic design criteria to support the selection of the design and evaluation methods used in this study.

Task 2 was designed to establish, based on laboratory tests and use considerations, a reasonable mix design rationale using, where appropriate, the existing methods generated in Task 1.

Task 3 provided an outline of the specific methodology required to characterize the suitability of materials for use in SEA-OGFC paving mixtures. Characterization test methods for these materials were also established. Three types of aggregates and three different grades of asphalt cements were used in this program.

A statistical analysis of variance of the various test results was carried out in conjunction with the experimental design phase. This was done to evaluate the relative significance of each primary variable in the mix design method and thus provide a basis for the elimination of any redundancy within the experiments.

Also included in this task was the establishment of a method for finding the optimum mixing and compaction temperatures to avoid drainage of the lower-viscosity sulfur-asphalt binders, settlement of undissolved sulfur particles, incomplete coating of aggregates, crystallization of sulfur during mixing and compaction and excessive evolution of hydrogen sulfide and/or sulfur dioxide.

In this particular task, two methods of incorporating sulfur with asphalt cements were considered, namely, direct substitution and emulsification. In direct substitution, sulfur and asphalt is added to the aggregate separately during mixing. In the case of the emulsified binder, sulfur and asphalts are pre-blended using a mechanical homogenizer and then added to the aggregates.

A crushed limestone, quartzite and a synthetic aggregate were used in this program. The resistance to crushing of synthetic (lightweight) aggregates is lower than for conventional gravels and limestones. Therefore, modifications had to be made to the

conventional compaction procedures to insure proper densification and structural integrity of the test samples.

Task 4 was designed to develop mix design rationale to optimize SEA-OGFC mixtures. A set of preliminary screening tests was conducted on a wide variety of prepared mixes to eliminate any that did not show adequate physical or mechanical behavioral characteristics required for use in OGFC highway pavements. The particular tests used in this screening program included:

1. Marshall Stability and Flow (ASTM D 1559)
2. Hveem Stability (ASTM D 1560)
3. Resilient Modulus (12)
4. Compressive Strength (Index of retained strength)  
(AASHTO T 167)

After the preliminary screening was accomplished, additional tests were introduced to further narrow the number of mix designs to those which exhibited high permeability to water and good freeze-thaw durability. These additional tests included:

1. Permeability (13)
2. Freeze-Thaw Durability (9)
3. Air Voids (14)

Task 5 dealt primarily with safety considerations associated with the preparation and construction of SEA-OGFC mixtures. Exposure to hydrogen sulfide, sulfur dioxide and particulate sulfur is hazardous to human health. The evolution of critical concentrations of these gases during construction can be hazardous to personnel as well as cause pollution of the environment. Therefore, recommendations for the sufficient control of the evolution of these two gases are presented under this task.

## CHAPTER II

### LITERATURE SURVEY

#### Development of OGFC in United States

The use of bituminous plant mix seal coats in the United States began in four Western states; Oregon, California, Arizona and Nevada (15). In the 1930's, Oregon began experimenting with open-type asphaltic concrete wearing surfaces (5). The top size of the aggregates used was 1/2-in. (1.27 cm) and the thickness of the mat laid was as thin as 3/4 in. (1.9 cm). Test results showed high skid-resistance, less glare from headlights of oncoming vehicles and better visibility of the centerline stripe. According to Eager (15), California used this type of construction as early as 1944. He reported that Arizona and Nevada began using OGFC's in 1954 and that FHWA began its use in federal parks in 1961. In these seal coats, aggregates were mixed with a relatively high percentage of paving-grade asphalt in a hot-mix plant and placed with a paver at a thickness of 5/8 to 3/4 in. (1.6 to 1.9 cm). Eager also reported that Indiana used a "retard" construction (7) similar in composition to plant mix seals as early as the 1930's.

Since the 1950's forty-nine states plus the District of Columbia and Puerto Rico have constructed open-graded asphalt surfaces (5, 11), and about fifteen states use open-graded construction as a part of their standard construction practices. For states involved in the early use of plant mix seals, the design of the mixtures evolved primarily from experience. These states still rely, to a major extent, on their own design procedures. More recently, however, some states have constructed sections based on the experimental procedures recommended by the FHWA (2). Approximately 35 different specifications are in use. A limited summary of specifications used by several states and other agencies is given in Table 1 (9). A more complete summary of state practices is also given by FHWA (7).

#### Properties of Elemental Sulfur

At normal room temperature and pressure, ordinary sulfur is an odorless and tasteless yellow solid (16). Sulfur is a non-metallic element with an atomic number of sixteen and an atomic weight of 32.06. It has a specific gravity of 2.0 and its melting point is at 240°F (116°C).

The working range of sulfur/asphalt blends is about 255°F to 300°F (124°C to 149°C), which corresponds quite well to the highway con-

Table 1. Survey of State Practices. (From Reference 9)

Item	North Carolina	FHWA Region 9 (Colorado, Wyoming, Utah and New Mexico)	FHWA Region 7 (California, Arizona, Nevada, Hawaii)	Franklin Institute	Louisiana																																																																				
Mix Designation	Bituminous Seal Coat	Open Graded Plant Mix Seal	Plant Mix Seal Coats	Open Graded Asphalt Concrete	Plant Mix Seal																																																																				
Aggregate Type	95% of material retained on No. 4 have at least one fractured face. Percent wear 45% or less (AASHTO T96)	Hard, durable, resistance to abrasion and stripping, sharp angular and polish resistant. Minimum 75% crushed.	Broken stone or crushed gravel with 90% by weight having at least one fractured face.	Same as California specifications.	Crushed gravel, slag or expanded clay. Maximum abrasion loss 45% (by LDI Designation TR11) for expanded clay.																																																																				
Aggregate Gradation	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>25-45</td></tr><tr><td>#10</td><td>0-10</td></tr><tr><td>#200</td><td>0-2</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	25-45	#10	0-10	#200	0-2	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>95-100</td></tr><tr><td>#4</td><td>30-50</td></tr><tr><td>#8</td><td>10-25</td></tr><tr><td>#16</td><td>0-18</td></tr><tr><td>#200</td><td>0-5</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	95-100	#4	30-50	#8	10-25	#16	0-18	#200	0-5	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>30-50</td></tr><tr><td>#8</td><td>15-32</td></tr><tr><td>#16</td><td>0-15</td></tr><tr><td>#200</td><td>0-3</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	30-50	#8	15-32	#16	0-15	#200	0-3	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>35-50</td></tr><tr><td>#8</td><td>15-32</td></tr><tr><td>#16</td><td>0-15</td></tr><tr><td>#200</td><td>0-3</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	35-50	#8	15-32	#16	0-15	#200	0-3	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>95-100</td></tr><tr><td>#4</td><td>30-55</td></tr><tr><td>#10</td><td>0-20</td></tr><tr><td>#40</td><td>0-12</td></tr><tr><td>#200</td><td>0-6</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	95-100	#4	30-55	#10	0-20	#40	0-12	#200	0-6
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	25-45																																																																								
#10	0-10																																																																								
#200	0-2																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	95-100																																																																								
#4	30-50																																																																								
#8	10-25																																																																								
#16	0-18																																																																								
#200	0-5																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	30-50																																																																								
#8	15-32																																																																								
#16	0-15																																																																								
#200	0-3																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	35-50																																																																								
#8	15-32																																																																								
#16	0-15																																																																								
#200	0-3																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	95-100																																																																								
#4	30-55																																																																								
#10	0-20																																																																								
#40	0-12																																																																								
#200	0-6																																																																								
Approximate * Cost per Ton	\$8.00 to \$15.00	\$7.00 to \$10.00	\$8.00 to \$12.00	None specified	None specified																																																																				
Asphalt Cement	60-70 penetration	60-70 penetration or 85-100 penetration	85-100 penetration	85-100 penetration	AC-40 (with 0.5% anti-stripping additive)																																																																				
Asphalt Content (percent)	6 to 10 actual value fixed by engineer	6 to 7	5 to 7	4.0 to 5.5	Crushed 4.0-10.0 Slag 6.0-12.0 Expanded clay 10.0-17.0																																																																				
Aggregate and Asphalt Temp. (°F) at Mixing	250 Maximum 300 maximum	260 to 300 260 to 300	290 290	--- ---	Mixing temperature of mix = 260° maximum.																																																																				
Stability, Flow and Voids	None specified	Retained stability 50% minimum by AASHTO T165	None specified	Exceed criterion for medium traffic uses (10-100 DTN)	None specified																																																																				
Remarks	Asphalt cement to contain 0.3% of: No Strip concentrate 380, Kling XX, Pave-bond 206, Kling HS-BETA-1000-3 or approved equal. No. 10 size aggregate limited to 10% to prevent bleeding.	Asphalt should be 200-300 centistokes at mixing temperature and ductility of 50 cm at 39°F. Stripping to be 95% retained coating (AASHTO T182).	Specifications are for California, Arizona and Nevada have similar specifications. No plant mix seal coat placed in Hawaii. Require film stripping test - 95% retained coating.	Mix with 5.5% asphalt most durable. Resistance to stripping by water (ASTM D1664) to be more than 95%. If less, add anti-stripping agent.	Placement temperature 180°F minimum. Hauling in excess of 20 miles may cause separation. A maximum of 260°F most critical for good mix.																																																																				

\*Cost estimates based on 1974 prices.

struction working range for asphalt cements. Between 240°F (116°C) and 320°F (160°C), the viscosity of liquid sulfur decreases gradually as temperature rises. At 320°F (160°C), there is an abrupt change in the viscosity trend and viscosity increases dramatically as temperature rises from 320°F (160°C) to 370°F (188°C). At a temperature of 370°F (188°C), the viscosity reaches a maximum and the liquid is almost opaque. Above 370°F (188°C), viscosity again decreases at a moderate rate with increasing temperature. The viscosity-temperature relationship is presented in Figure 1.

The occurrence of sulfur deposits in nature is often associated with the formation of salt domes. Commercially pure sulfur is non-toxic and is produced in the United States by the Frasch process and as a by-product of pollution abatement processes in power plants and sour natural gas wells. It is one of the purest raw materials known. The majority of the commercially produced sulfur used in the United States is shipped in the liquid state.

#### Sulfur-Asphalt Mixtures

When hot asphalt and molten sulfur are blended, three different physico-chemical activities can result (17): (a) dehydrogenation as a result of the chemical reaction between the sulfur and the asphalt, (b) dissolution of the sulfur in the asphalt, and (c) suspension of some undissolved sulfur in the asphalt. The solubility of sulfur in asphalt is a function of the type and source of the asphalt cement (17), but is independent of the total amount of sulfur added to the emulsion. The relative amounts of dissolved and dispersed sulfur influence the properties of the blend, as shown in Figure 2.

Sulfur's unique properties permit it to be used as a structuring agent (i. e., playing the role of the aggregate), as an integral part of the binder or as both. As a result there has been an increase in research activities directed toward the use of sulfur in the development of pavement materials which permit the use of lower quality, locally available aggregates (18-22) along with concepts which permit the partial or complete replacement of asphalt and portland cement as the binder in concrete pavements and structures (23-30). Research efforts have also been directed toward using sulfur in the recycling of old bituminous pavement materials (31).

#### Development of SEA-OGFC

A review of the literature indicates that experience is very limited in the design of SEA open-graded mixes. No developmental

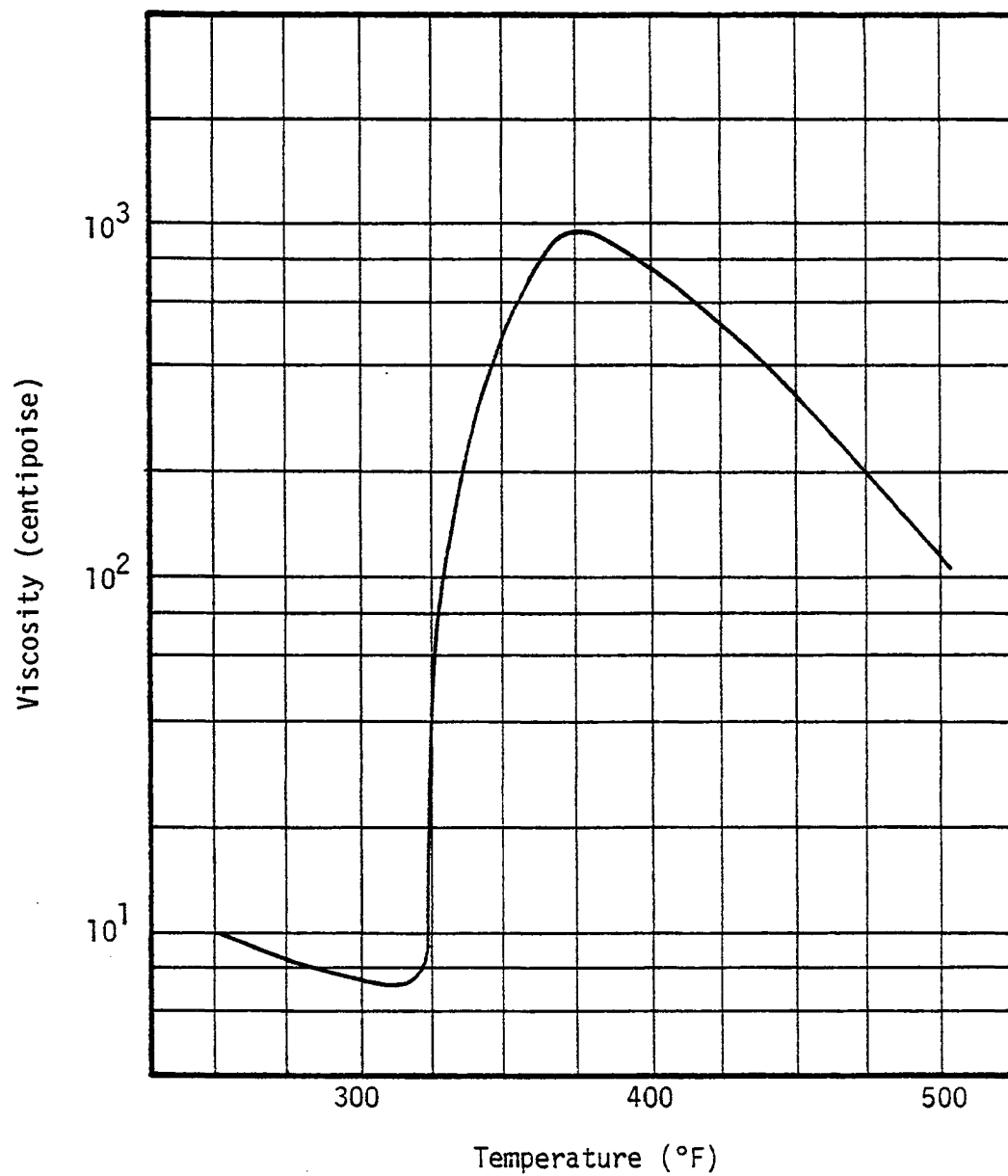


Figure 1 . Viscosity-Temperature Curve for Liquid Sulphur

$$^{\circ}\text{C} = (^{\circ}\text{F} - 32) \times 5/9$$

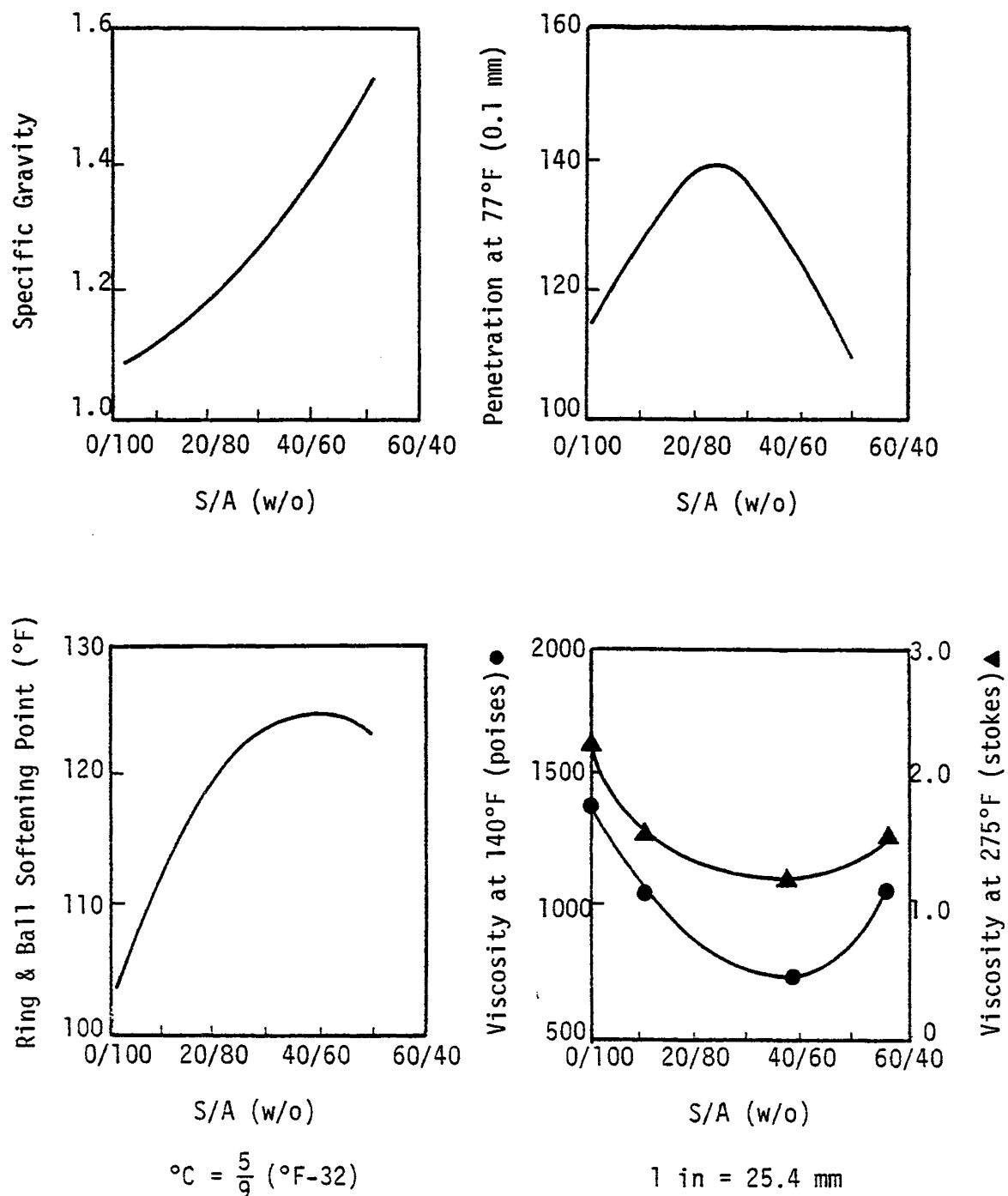


Figure 2 . General Trend of Physical Properties of Sulphur-Asphalt Emulsions.

work has been done to date in which sulfur was incorporated into the design of open-graded friction courses.

An SEA-OGFC field trial was conducted on Loop 495 in Nacogdoches, Texas, in August, 1980 (32). The sponsors and participants included the Texas State Department of Highways and Public Transportation, Texasgulf, Incorporated, Federal Highway Administration Implementation Division, East Texas Asphalt Company, The Sulfur Institute and the Texas Transportation Institute. The mixture had an S/A weight ratio of 35/65 in the SEA binder. The completed section is currently under evaluation.



## CHAPTER III

### MIX DESIGN PROCEDURES

Based on the review of the literature given in Chapter II, there were several items which have to be adequately dealt with in the successful placement of open-graded friction courses. The factors include polish susceptibility, surface texture, particle shape, unit weight, adequate coating of aggregates, method of compaction, asphalt drain-down, binder absorption, degradation and densification, and particularly temperature considerations (1,9,33). Review of several design specifications indicated that the design procedure described by Smith, et al., (2), is probably the most detailed method available among the contemporaries. When this procedure was first published, the determination of optimum asphalt content for lightweight aggregate mixture was not included. In a later publication (7), an alternative procedure for determining the optimum asphalt content for lightweight aggregate was added to the original procedure. A copy of the subsequent publication which includes the modification to the original procedure is included in Appendix A. A summary of the design consideration suggested by Gallaway, et al., (9) is given below.

#### Design Considerations

Aggregate requirements. The physical properties of the coarse aggregate in OGFC's are the controlling factors in providing adequate skid-resistance on the pavement surface. Both the macrotexture and the microtexture of aggregate surfaces contribute to the attainment of skid-resistance characteristics. Unlike the case with dense-graded mixtures, the coarse aggregate fraction in the OGFC must provide, be itself, the degree of macrotexture and microtexture necessary in the maintenance of a high skid-number. Therefore, aggregates with a high susceptibility to polish should not be used in the coarse aggregate fraction (i.e., material retained on No. 8 (2.38 mm sieve) of OGFC.

Gallaway and Epps (9) recommended that a specific minimum polish value for all aggregates, as determined by Texas Method Tex-438-A, of not less than 35 be required for traffic volumes less than or equal to 4,000 vehicles per day per lane. For heavier traffic volume, a minimum polish value of 40 should be specified.

Another important physical property of the aggregates used in an OGFC is the particle shape. Its major impact lies in the degree of ultimate densification of OGFC under traffic. Use of crushed

aggregates containing flat and elongated particles tends to result in the excessive densification of the OGFC in service. In the areas where heavy traffic and hot water prevail, block, subrounded and round particles, which have a higher resistance to densification, must be used in order for the OGFC to remain open and to permit surface water drainage.

If the coarse and fine aggregate fractions are separated using a No. 10 sieve, the recommended grading should be as follows:

<u>Sieve Size</u>	<u>Percent Passing</u>
1/2-inch (12.7 mm)	100
3/8-inch (9.52 mm)	90-100
No. 4 (4.76 mm)	40-60
No. 10 (2.00 mm)	8-12
No. 200 (0.074 mm)	2-5

Asphalt requirement. The amount of asphalt affects the ultimate utility and durability of the pavement mixture. If the mix contains too much asphalt flushing may result. On the other hand, too little asphalt in the mix may cause ravelling. Therefore, the selection of proper asphalt content is a critical element in the design of the optimum mix.

FHWA provides a two-step method of selecting the optimum asphalt content (see Appendix A). First, the surface capacity, surface roughness and absorption of aggregate sizes above the No. 4 (4.76 mm) sieve are determined by using a modified oil equivalent test developed in California. The surface capacity is represented by a surface constant  $K_c$ . Using this surface constant  $K_c$ , together with an empirical formula (given in Appendix A) based on field experience with similarly graded mixtures, the optimum asphalt content can be calculated. For aggregates that have a specific gravity less than 2.60 or greater than 2.70, FHWA provides a correction factor for the selected asphalt content.

Sulfur requirement in SEA/OGFC binders. The introduction of sulfur to replace part of the asphalt binder in the OGFC mixture will lead to an increase in the stiffness of the OGFC (see Chapter V). But the maximum amount of sulfur introduced into the SEA binder must be carefully controlled, since an increase in sulfur content leads to a decrease in moisture resistance as reflected in the Immersion-Compression test (see Chapter V).

To assure an equivalent aggregate coating capability it is recommended that an equal volume of SEA binder be used to replace the asphalt cement binder. However, construction experience using on conventional A/C mix designs placed in the United States and Canada

indicates that due to the lower viscosity of SGA binders less than equal volume substitution may be feasible (34). Since binder content is usually expressed in units of percent by dry weight of total aggregate, and the specific gravities of sulfur and asphalt are different, it is suggested that the following formula as developed by McBee (22) be used in this determination.

$$\text{Percent SEA binder} = A \times \frac{100R}{[100R - S(R - G)]}$$

where

- A = percent asphalt by dry weight of aggregate before substitution,
- R = ratio of specific gravities of sulfur to asphalt; 2 to 1
- S = percent sulfur by weight in the SEA binder, and
- G = specific gravity of the asphalt.

This calculation can be shown in the following example. A pavement mixture has an asphalt content of 6 percent by dry weight of the aggregates, and as S/A ratio of 40/60 SEA binder is used to replace the asphalt binder, hence

$$\text{Percent SEA binder} = (6) \frac{100(2)}{[100(2) - 40(2 - 1)]} = 7.5$$

i.e., 7.5 percent SEA binder by dry weight of aggregate is required to replace the asphalt binder in the mixture.

Aggregate gradation. Aggregate gradation is a major factor in determining the macrotexture and internal drainage characteristics of OGFC's. Thirty-five different grading specifications have been used in the United States, but most of them fall within the ranges recommended by FHWA (Figure 3). Consequently, adoption of these ranges is also recommended for SEA-OGFC systems.

In specifying the aggregate gradation, there is no significant difference in using percent by weight or percent by volume so long as all fractions of the aggregate or aggregates have the same or close specific gravities. However, if two or more aggregates having significantly different specific gravities are being blended, adjustment to proper volume percentages may be required.

Time and temperature considerations. Time and temperature controls are two of the more important factors which determine the successful placement of the SEA-OGFC materials. The mixing temperature must be carefully monitored to prevent drainage of liquid sulfur through the porous mixture. The time interval between the preparation and place-

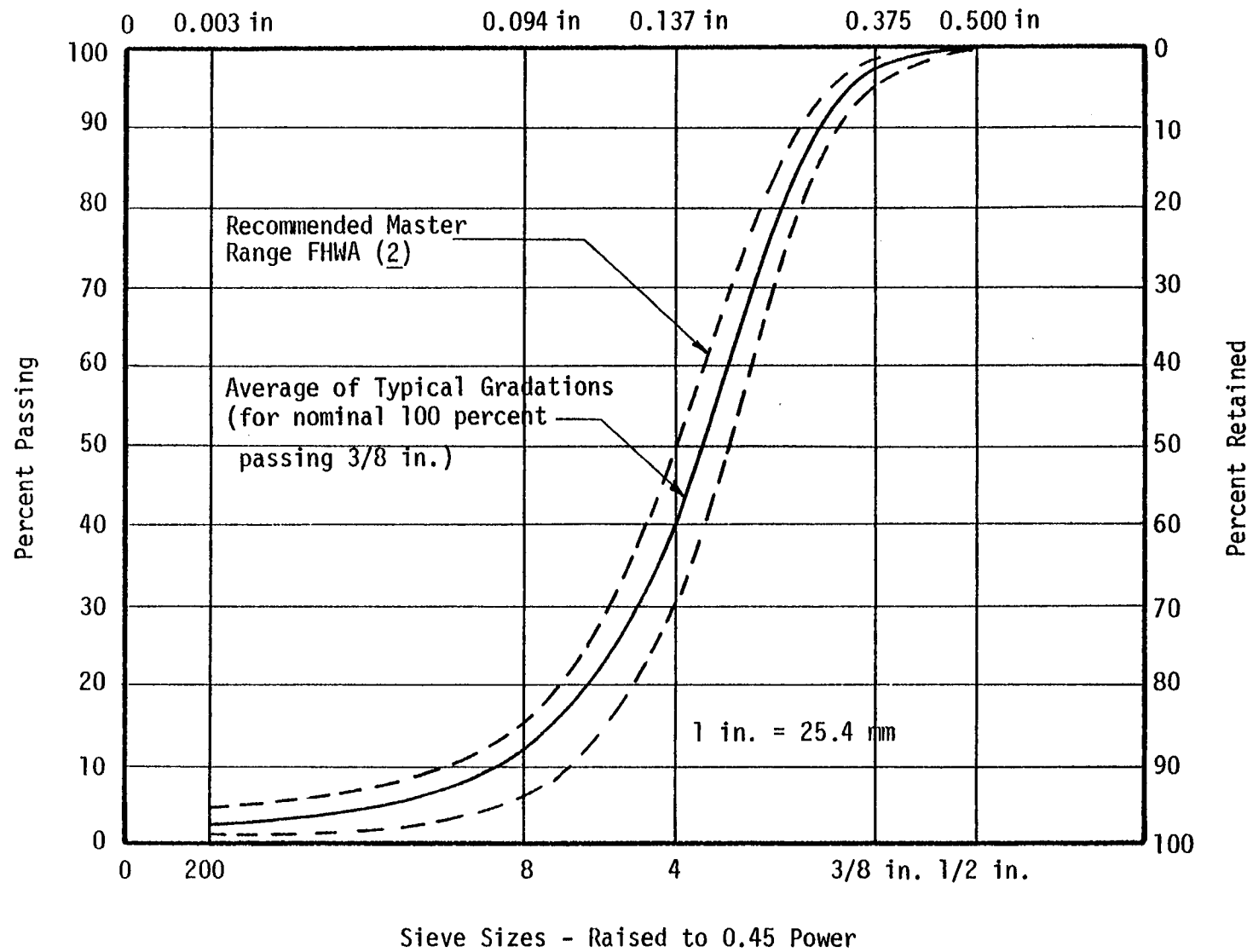


Figure 3 . Average of Typical Gradations in Use by States for Open-Graded Asphalt Friction Courses

ment of SEA-OGFC materials with high sulfur content binders is very significant, because the undissolved sulfur which has a higher specific gravity, has a tendency to settle with time and create a non-homogeneous mixture.

Degradation. It is generally believed that aggregate crushing does occur during construction and continues on a smaller scale during service. For a given material and aggregate gradation, the amount of contact area between the tire and road surface plays an important role in aggregate degradation, because the interaction of the tire and the pavement surface is an area-to-area phenomenon. Therefore, the stress levels acting on the aggregates will be very high if the points of contacts are few and aggregates will tend to be crushed more readily. One way to alleviate this problem is to increase the number of contact points, so that the loss of adequate voids and possible flushing could be kept at a minimum. This can be done by: (a) using an aggregate gradation with a small top-size, (b) using blocky, rounded or subrounded particle shapes and (c) adding some intermediate sized material to create a "chocking" action.

Densification. Excessive densification can significantly reduce the permeability of OGFC materials. Over-densification can also lead to flushing of the binder when the pavement is under traffic during hot weather conditions. This type of phenomenon may also be due to aggregate gradation and particle shape. Therefore, flat and elongated aggregates are not recommended for use in the construction of OGFC's. On the other hand, blocky, subrounded and rounded particles, which have higher resistance to densification under traffic and good surface texture, are highly recommended for use in OGFC mixtures.

Absorption and moisture problems. Normal weight aggregates do not present a great problem in this area, because the absorption capacity of such aggregates is relatively low. Recently, lightweight, synthetic aggregates have been considered and some have been used on several OGFC trial sections. These materials usually have water absorption capacity ranging from 10 to 30 percent by weight (9). If these types of aggregates have absorbed a large quantity of water before they enter the dryer, one can reasonably suspect that some water may still be present in the larger particles when they reach the road surface. The presence of this moisture may result in early distress, since moisture is trapped inside the larger particles. It can also lead to freeze-thaw durability problems. Laboratory data and discussions of freeze-thaw tests on these particles types of aggregate mixtures are presented in Chapter V.

Binder absorption. As discussed above, lightweight aggregates tend to have a higher absorption capacity (which may include water and asphalt) than normal weight aggregates. This absorption capacity, in effect, may require a comparatively higher binder content in the mixture. Gandhi made some studies (unpublished) on the asphalt absorption of lightweight synthetic aggregates which were obtained from seven sources (35). Only materials passing 3/8-in. (9.5 mm) and retained on No. 12 (2 mm) sieve were tested and a single source AC-10 asphalt cement was used. The amount of asphalt absorbed was found to be in the range of two to three percent by dry weight of the aggregate for all the aggregates tested. Since all the aggregates were oven-dried to constant weight before the test, a lower asphalt absorption capacity would be expected if some moisture were trapped in the aggregate. However, in the field situation, moisture is continuously released from the aggregates during mixing, hauling and placing of the mixture and this action hinders the binder absorption (9). Consequently, it can reasonably be expected that a lower binder absorption capacity than those indicated in Gandhi's report will be experienced under field conditions.

#### Aggregate Grading and Characterization

Aggregates and their suppliers. There were three different types of aggregates used in this program. The first aggregate was a standard laboratory coarse-grained crushed limestone, which had a polish value of about 28-32 and was supplied by White's Mines of Brownwood, Texas. The second aggregate, an East-Texas sandstone, was used as a quartzite type aggregate in this program and was supplied by East Texas Stone Company located in Oakwood, Texas. This aggregate had a polish value between 39 and 45. The third was a lightweight, synthetic aggregate, which was labeled Streetman in this report and was supplied by Texas Industries Company of Streetman, Texas. It had a comparatively high polish value of 47 to 50.

At least 75 percent (by weight) of the coarse aggregate fraction of all the above aggregate types had at least two fractured faces and 90 percent had one or more fractured faces. Thus, they were in compliance with the FHWA specifications and recommendations (7).

Aggregate grading. The coarse and fine aggregate fractions were separated by the percent retained and passing the No. 8 (2.30 mm) sieve, respectively. The former was called coarse aggregate and all aggregates that passed the No. 8 (2.38 mm) were considered to be in the fine aggregate fraction. It should be noted that sharp-crushed material passing the No. 8 (2.38 mm) sieve has a better ability to stabilize the mat which, thus, reduces mixture tenderness and the tendency to ravel. This phenomenon is sometimes called the "chocking effect". Limestone

was chosen to be used as the fine aggregate fraction material for all three types of aggregates in order to eliminate a design variable which the fines fraction would impose on the behavior of the mix if different materials were used.

All the aggregates were blended using the Percent-by-volume Method. This was done to minimize the effects of the difference in unit weights of the aggregates, particularly in the case of blending Streetman (lightweight) and limestone systems into one gradation.

The preliminary gradations used with the three types of aggregates are given in Table 2. The FHWA recommended gradation for OGFC's is also shown.

Bulk and apparent specific gravities. Bulk specific gravity tests were performed on coarse aggregate fractions (retained on the No. 8 (2.38 mm) sieve) for all three types of aggregates according to the procedures set forth in AASHTO T85 (36). Since limestone was used for the fines fraction in all three types of aggregate, bulk specific gravity tests were done on limestone fine fraction aggregates using the procedure published under AASHTO designation T84 (37).

Apparent specific gravity tests were performed on material passing the 3/8 in. (9.52 mm) and retained on the No. 4 (4.76 mm) sieve on all three types of aggregates. The procedure employed is specified in AASHTO designation T85. The determination of water absorption capacity for all three types of aggregates was also included in the test.

Abrasion loss. All three types of aggregates were subjected to the Los Angeles Abrasion Loss Test according to the procedure outlined in AASHTO designation T96 (38). The maximum allowable loss was set at 40 percent.

Selection of optimum asphalt content. The surface capacity and absorption capacity of the aggregates were determined using the procedure outlined in Section 3 of Appendix A. The method described in Appendix A also includes an updated procedure suitable for use with highly absorptive aggregates.

An example of the procedure used for determining the optimum asphalt content is given below using data obtained on limestone.

	Sample A	Sample B
Sample Weight (before test), gm	100.0	100.0
Sample Plus Oil Weight, gm	102.6	102.0
Percent Oil Retained	2.6	2.0
Average Percent Oil Retained		2.3

Table 2 . Preliminary Gradation for Limestone, East Texas Sandstone and Streetman Aggregate.

Sieve Size	Aggregate	Percent Passing by Volume			
		Limestone	East Texas Sandstone	Streetman	FHWA Specification
1/2"		100	100	100	100
3/8"		97	95	98	95-100
#4		41	32	30.3	30-15
#8		10	7	7	5-15
#16		7	4.9	4.9	
#30		6	4.2	4.2	
#50		5	3.5	3.5	
#100		4	2.8	2.8	
#200		3	2.1	2.1	2-5

1 in. = 25.4 mm



Since the apparent specific gravity of limestone is 2.71 (which is outside the given range of 2.60 to 2.70) a correction on the percent oil retained is required.

where 
$$\text{Corrected Average Percent Oil Retained} = \frac{O_R \times (SG)_{ca}}{2.65}$$
  
 $O_R$  = Average Percent Oil Retained  
 $(SG)_{ca}$  = Apparent specific gravity of coarse aggregate

Hence

$$\text{Corrected Percent Oil Retained} = 2.3 \times \frac{2.71}{2.65} = 2.35$$

Using a corrected percent oil retained value of 2.34 and going to Figure A-1 in Appendix A, the surface constant,  $K_c$ , was found to be 1.06.

The formula for calculating percent asphalt is as follows (7):

$$\text{Percent Asphalt} = (2.0 K_c + 4.0) \times \frac{2.65}{(SG)_{ca}}$$

Where

$K_c$  = surface constant

$(SG)_{ca}$  = apparent specific gravity of coarse aggregate (passing 3/8 in. (9.52 mm) and retained on the No. 4 (4.76 mm) sieve).

Therefore

$$\text{Percent Asphalt} = [2.0 (1.06) + 4.0] \times \frac{2.650}{2.707} = 6$$

Voids in mineral aggregate (VMA). The voids in mineral aggregate (VMA) are important because they indicate the relative amount of space available between the individual particles in the aggregate mass. One of the primary functions of OGFC's is to drain surface water laterally and internally during a rainfall. Such drainage channels must possess adequate capacity to handle normal rainfall without flooding. The void capacity of the mixture is primarily controlled by the compactability of the coarse aggregate. Therefore the coarse aggregate mass must provide the amount of interstitial voids which can accommodate the asphalt and fine aggregate, and simultaneously leave enough voids for internal drainage in the compacted mixture. Using

the test procedure described in Section 4 of Appendix A, one may determine, assuming no aggregate degradation, the minimum level of interstitial voids that could exist in the friction course after long-term densification under high traffic volume.

Optimum fines content. The concept of selecting the optimum fine aggregate content lies in the fact that adding a fine aggregate fraction can provide a "choking effect" to the mixture to prevent ravelling, while limiting the maximum large voids in the system for water drainage purposes. A design air voids content of 15 percent maximum and 15 percent of fine aggregate was allowed in the total aggregate.

The following equation was used for finding the optimum fines content used in this project (as described in Appendix A):

$$Y = \frac{[\%VMA-V] - [\%AC] (X)/U_a}{[(\%VMA-V)/100] + [(X)/U_f]}$$

Where:

Y = percent fines passing the No. 8 sieve, by weight,

X = actual vibrated unit weight of coarse aggregate  
(retained on the No. 8 (2.38 mm) sieve, lb/ft<sup>3</sup>,

U<sub>f</sub> = theoretical bulk dry soil unit weight of fine  
aggregate (passing the No. 8 (2.38 mm) sieve, lb/ft<sup>3</sup>,

U<sub>a</sub> = unit weight of asphalt cement, lb/ft<sup>3</sup>,

%AC = percent asphalt by total weight of aggregate  
 $(2.0 K_c + 4.0) \frac{2.65}{(SG)_{ca}}$

V = design percent air voids (15.0 percent)

U<sub>c</sub> = theoretical bulk dry solid unit weight of coarse  
aggregate (retained on the No. 8 (2.38 mm) sieve)

%VMA = percent voids in mineral aggregate of the coarse  
aggregate (retained on the No. 8 (2.38 mm) sieve).

which is  $[100 - 100 \frac{X}{U_c}]$

Using the data obtained on Limestone aggregate, an example for the calculation of the optimum design content of fine aggregate is as follows:

$$\begin{aligned} X &= 114.0 \text{ lb/ft}^3 \\ U_f &= 158.5 \text{ lb/ft}^3 \\ U_a &= 62.4 \text{ lb/ft}^3 \end{aligned}$$

$$\begin{aligned}\%AC &= 6.0 \\ V &= 15.0 \text{ percent} \\ \%VMA &= 31.5\end{aligned}$$

Hence

$$\begin{aligned}Y &= \frac{[31.5-15] - [(6.0) (114.0/62.4)]}{[(31.5-15)/100] + [(114.0/158.5)]} \\ &= 6.26 \text{ percent} \\ &= 6.3 \text{ percent (approximately)}\end{aligned}$$

Therefore the optimum fines content is found to be 6.3 percent.

Since the fines content was proposed at 10 percent in the preliminary gradation, an adjustment of the limestone gradation was necessary.

The same method was applied to find the optimum fines content for the other two types of aggregates, East Texas Sandstone and Streetman and their gradations were adjusted accordingly. The final gradations on all three types of aggregates are given in Figures 4, 5, and 6. Data on aggregate characterization tests are given in Table 3.

### Asphalt Grades and Suppliers

Three different grades of asphalt cements were used in this project. They were AC-10, AC-20, and a special residue which was used as an AC-40 in this program.

The AC-10 and AC-20 were supplied as standard laboratory asphalt cement by the American Petrofina Refining Company Located in Mt. Pleasant, Texas. A special residue, which was used as an AC-40 here, was manufactured by Dorchester Refining Company, Mount Pleasant, Texas. The higher viscosity asphalt was used to offset the fluxing affect sulfur has when blended with asphalt.

Each of the asphalt cements was subjected to a series of standard laboratory tests to determine their physical properties. The test data are given in Table 4 and were in compliance with the suggested specifications.

### Sulfur and Its Supplier

Elemental sulfur used in this project was of commercial grade and supplied by Stauffer Chemical Company. The sulfur was purchased in the solid (powder) form and shipped in 50 lb. (22.7 kg) bags.

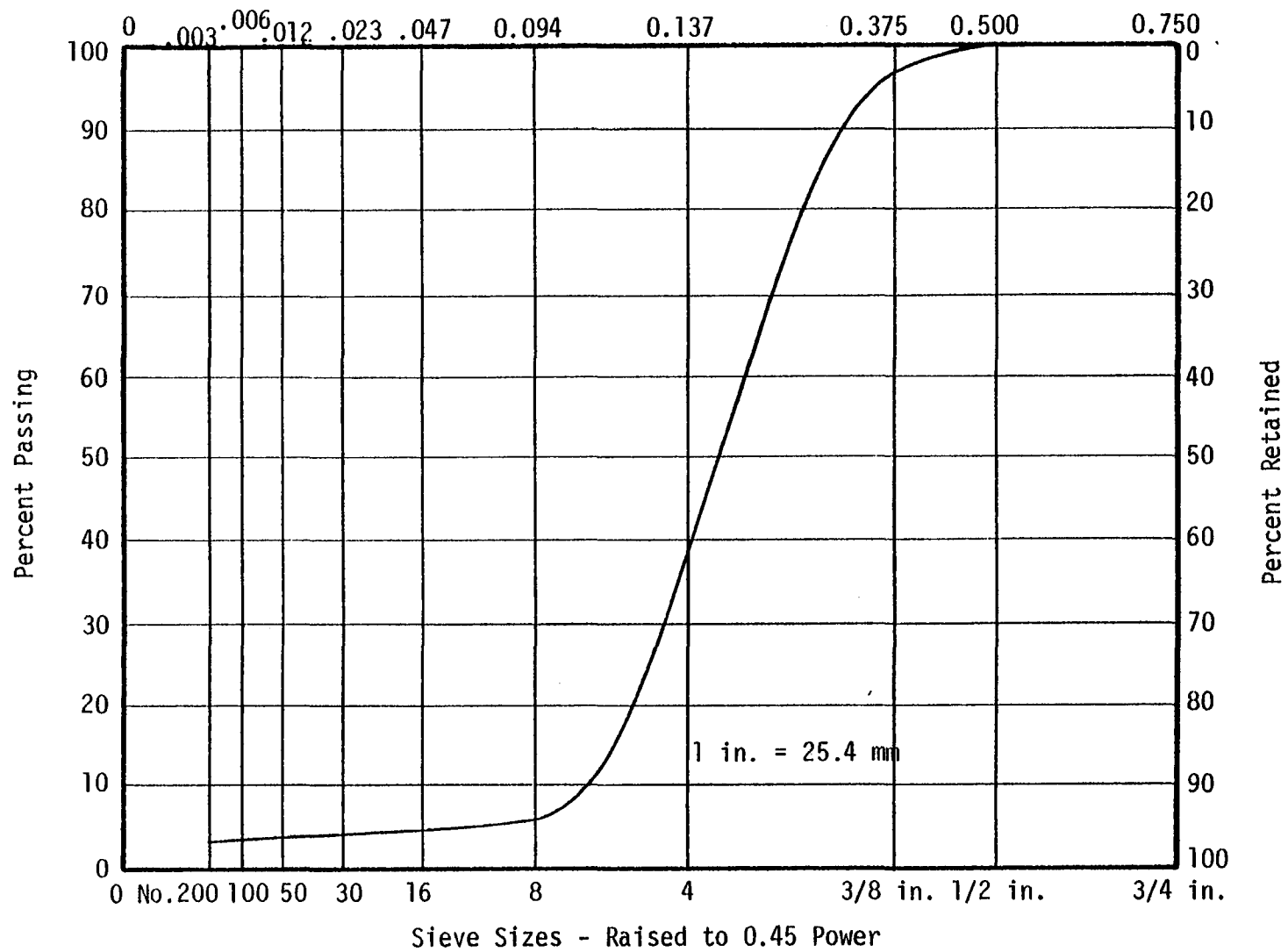


Figure 4 . Aggregate Gradation for Limestone

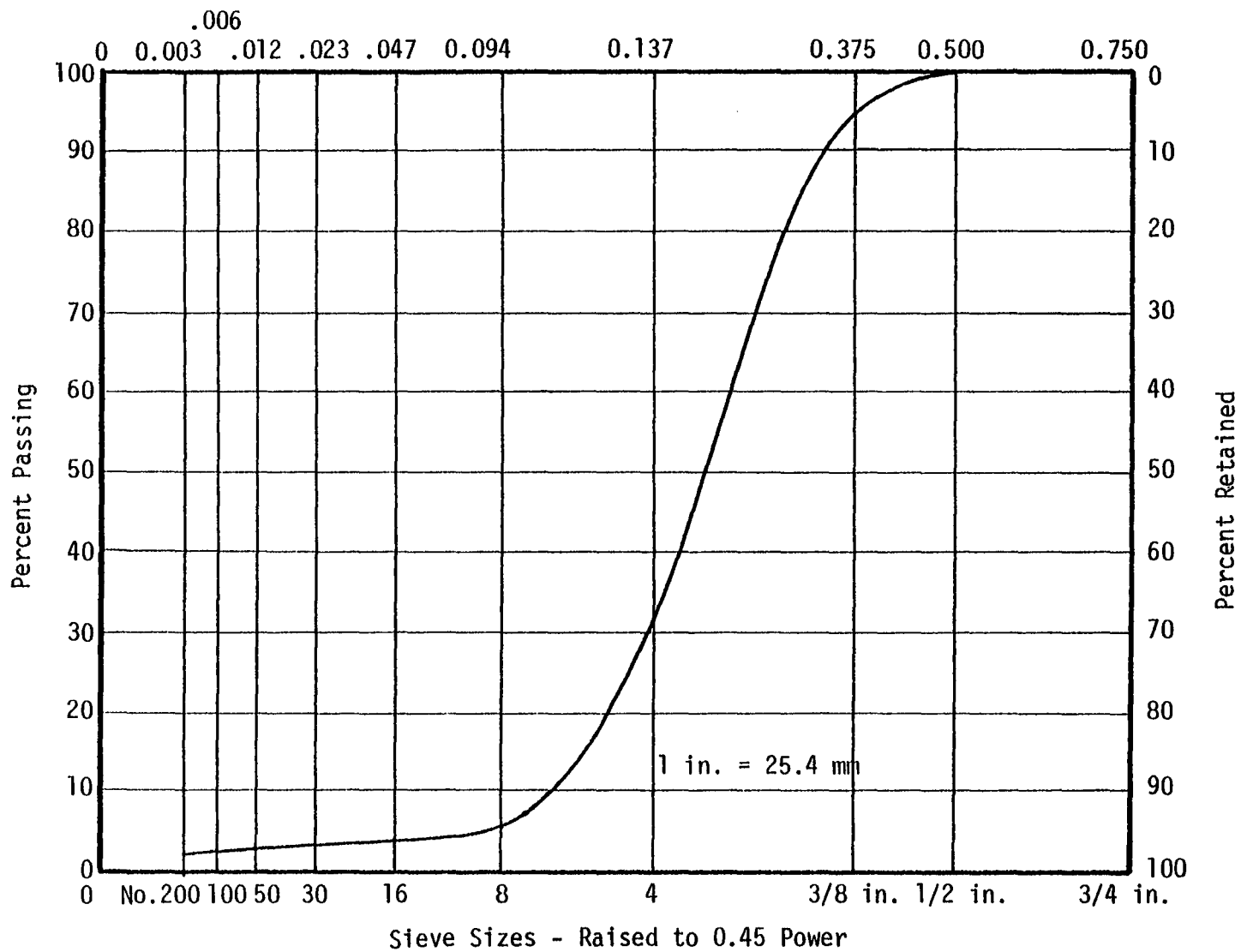


Figure 5. Aggregate Gradation for East Texas Sandstone

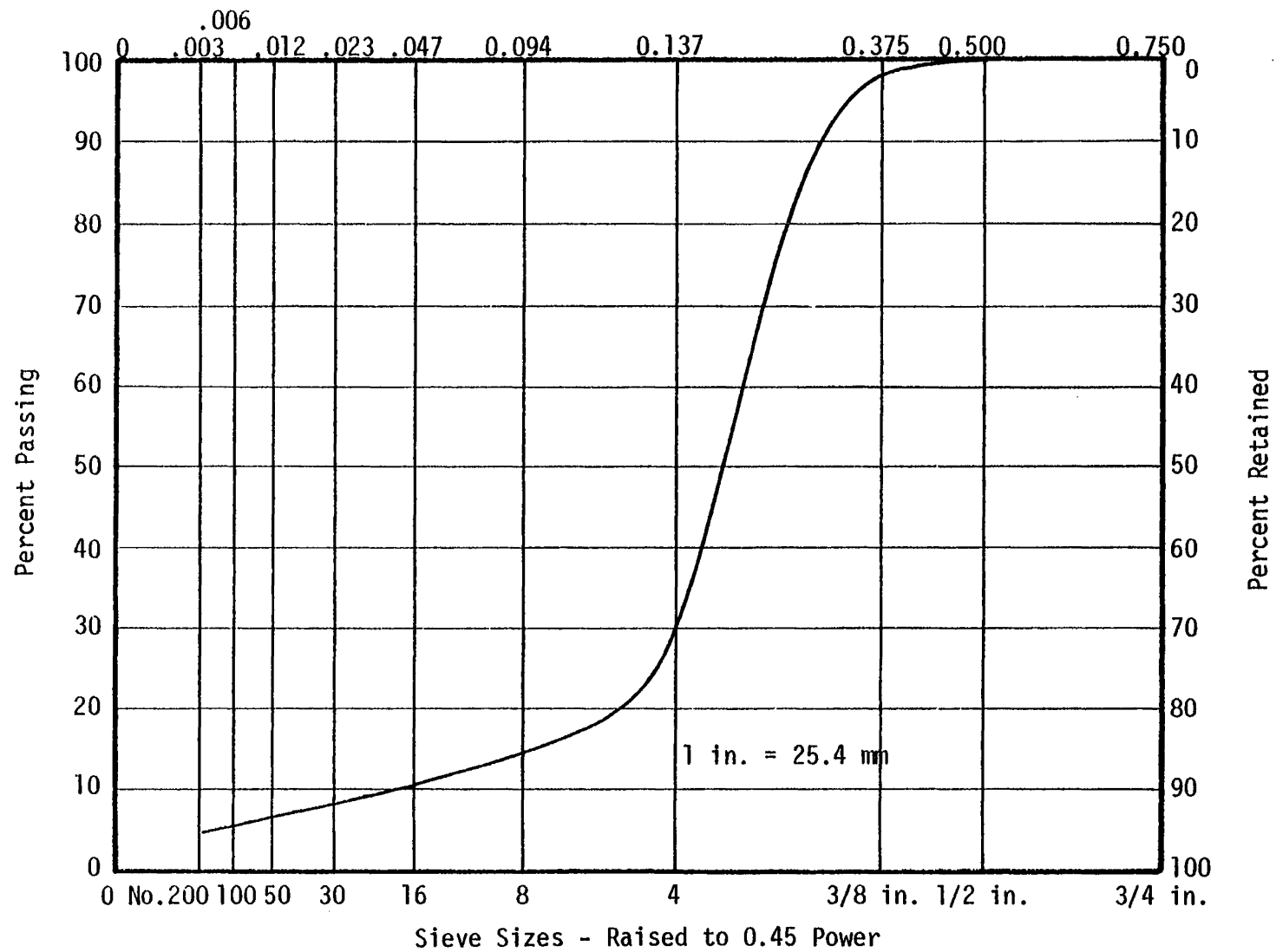


Figure 6 . Aggregate Gradation for Streetman

Table 3. Characterization Data for Selected Program Aggregate Systems

Test	Test Method	Aggregate		
		Limestone	East TX Sandstone	Streetman (Lt. Wt.)
(1) Bulk Sp. Gr. (Coarse)	AASHTO T85	2.66	2.74	1.25
(2) Bulk Sp. Gr. (Fine)	AASHTO T84	2.54	2.54	2.54
(3) Asphalt Content, %	FHWA Design [1]	6	6	11
(4) L.A. Abrasion Loss, %	AASHTO T96	23	22	18
(5) Absorption, % (Coarse)	AASHTO T85	0.7	0.9	14.8
(6) Absorption, % (Fine)	AASHTO T84	2.2	2.2	2.2
(7) Void Capacity (UMA) %	FHWA Design [1]	31.5	31.3	41.8
(8) Vibrated Unit Wt.pcf	FHWA Design [1]	114	117	47
(9) Opt. Fine Content, %	FHWA Design [1]	6.3	5.6	15 (Vol)
				26 (Wt.)

1 pcf = 0.016 gm/cm<sup>3</sup>

Table 4 . Asphalt Cement Properties.

Test	Test Methods	AC-10	AC-20	AC-40
		Test Results	Test Results	Test Results
Viscosity @ 140°F, poises	ASTM D-2171	1576	1934	4815
Pen @ 77°F, dmm	ASTM D-5	118	63	63
Flash Point, °F	ASTM D-92	615	580	608
Ductility @ 77°F, cm	ASTM D-113	150+	141+	60
Ring and Ball, °F	ASTM D-36	102	120	132

$$^{\circ}\text{C} = \frac{5}{9}(\text{F} - 32)$$

$$1 \text{ in.} = 2.54 \text{ cm}$$



### Mixing Temperatures

The method of determining the optimum mixing temperatures mentioned in Section 6 of Appendix A was used throughout this program. For lightweight aggregates, such as Streetman, a 600-g sample should be prepared for the test instead of using a 1000-g sample indicated in Section 6.1.

Long haul distances of SEA-OGFC materials from the plant to the jobsite are not recommended due to the possible seepage of the sulfur throughout the porous mix. This could be minimized by using a higher viscosity asphalt to compensate for the viscosity reduction produced by the sulfur. Observations of the bottom of the plate, as described in Section 6.1, should be made after 15 and 45 minutes. In addition, one should ascertain any abnormal drainage of undissolved sulfur.

### Mixing the SEA Binders

One of the segments of this study investigated two methods for producing sulfur-extended asphalt binders: direct-substitution and emulsification.

In addition, two different methods of direct-substitution of sulfur and asphalt were employed in preparing specimens for treatment in a statistical analysis of variance study. The first method is referred to as "direct substitution" in which sulfur and asphalt were poured into the mixing bowl directly and separately during mixing. This method was used in preparing specimens with East Texas Sandstone. The second method is referred to as modified direct substitution. In this method, the desired amount of sulfur and asphalt are first added to a weight bucket, and then poured together into the mixing bowl. This method was used in preparing specimens with Streetman aggregates.

Emulsification of the sulfur-asphalt blends was accomplished using a 2-gallon Eppenbach mechanical shear-action homogenizer which was heated to and maintained at 250°F (121°C). Since sulfur has a melting point of about 240°F (116°C) this temperature prevented the undissolved sulfur from crystallizing during the emulsifying process. The desired amount of sulfur and asphalt was preheated to 300°F (149°C) and 285°F (141°C), respectively. When the blending temperature was reached, hot asphalt was poured into the homogenizer. After one minute of agitation, the sulfur was also introduced. Two minutes after the sulfur had been added, the sulfur-asphalt blend was drained from the homogenizer.

and stored in an oven for future use. Prolonged storage at temperatures above 240°F (116°C) is not recommended due to the evolution of toxic gases ( $H_2S$  and  $SO_2$ ) in the closed environment of the oven. For a discussion of the safety aspects of sulfur extended asphalt mixes the reader is referred to Appendix B. The blend was restirred before each use.

### Compaction

There were three different specimen sizes required in the evaluation test employed in this program. They include: 4" diameter x 2 1/2" height (10.16 cm x 6.35 cm) specimens used in resilient modulus, Hveem and Marshall tests; 4" diameter x 4" height (10.16 cm) specimens used in the immersion-compression tests; and finally 6" diameter x 1" height (15.24 cm x 2.54 cm) samples used in the permeability and freeze-thaw tests. These specimens were fabricated using the three different compaction methods discussed below.

Resilient Modulus, Hveem and Marshall specimens. The 4" diameter x 2 1/2" high (10.15 cm x 6.35 cm) used in the resilient modulus, Hveem and Marshall tests. The molding equipment used in the preparation of these specimens was a Texas Highway Department gyratory shear compactor. The procedure used here was similar to the one outlined in Texas Highway Department Designation Tex-206-F, Part II, Vol. I, Manual of Testing Procedures (39). However, in order to use this outlined procedure in compacting OGFC mixes, some modifications had to be made. A copy of the Texas Method procedure is included in Appendix C, and details of the modifications are given at the end of Appendix C. A summary of the procedure used is given below.

The molding temperature of the mix was 5°F (2.8°C) below the mixing temperature as determined by the drainage test procedure described in Appendix A. After depositing the mixture into the mold, the mold and its contents were placed on the platen of the compactor and centered in the molding position beneath the ram of the press. As soon as the mold was in the right position, the ram was lowered into the center of the mold until the lower pressure gauge registered 25 psi. The handle of the cam-lever was pulled down to a position to deliver an angle of gyration of 1°. After completion of the gyratory compaction, the cam-lever was raised to the vertical position, and the mold leveled. The pressure was readjusted to 25 psi and the mold was repositioned for a 1° angle of gyration. This process was repeated until one smooth stroke of the pump handle would cause the lower pressure gauge to indicate a pressure of 100 psi or more. At this end point, the pump handle was lowered slowly until the automatic gauge protector valve cut the low pressure gauge out of the system. Then, with about one stroke per second, the pressure was pumped up to 1500 psi, as measured on the high pressure gauge. At this point, the

compaction of the specimen had been completed.

Immersion-compression specimens. The compacting temperature for these specimens was 5°F (2.8°C) below the mixing temperature as determined in the drainage test (see Appendix A). The method of compaction in AASHTO T167 (40), with some appropriate changes in the procedures for use on OGFC systems, used used to prepare these samples. The original procedure, (AASHTO Designation T167) along with the suggested modifications, can be found in Appendix D.

Since the standard compaction pressure of 3000 psi caused excessive crushing of the aggregates, particularly in the case where a lightweight aggregate was used, a lower molding pressure was substituted. It had been found, as a result of a number of tests run at various loading pressures, that 1000 psi would successfully compact the mixture with little damage to the limestone and East Texas Sandstone aggregates. In the coase of the lightweight aggregates, a pressure of 750 psi had to be used.

Prior to sample molding, all hardware was brought to a temperature of 140°F (60°C). As soon as the mixture reached the molding temperature, approximately one half of the molding cylinder was filled and the mix spaded vigorously twenty-five times with a heated spatula. The remaining half of the molding cylinder was filled and the spading action repeated. With the top and bottom plunger in place, an initial load of 150 psi was applied to set the mixture against the sides of the cylinder. The pressure was increased to and maintained at 1000 psi (750 psi for lightweight aggregate mixture ) for 120 seconds. The pressure was subsequently released thus completing the molding procedure. After removal from the mold, specimens were oven cured for twenty-four hours at 140°F (60°C) before testing.

Permeability and freeze-thaw specimens. These specimens were molded at 5°F (2.8°C) below the mixing temperature as determined in the drainage test described in Appendix A. The method for compacting OGFC mixtures was in accordance with that outlined in Texas Method Designation Tex-126-E (41), with one exception. The angle of gyration should be reduced from 5° as specified in the procedure to 1°. The current method, along with the proposed changes, are given in Appendix E.

The original procedure outlined in Texas Method Designation Tex 126-E with a 5° angle of gyration, was used for molding the base for material meeting Texas Highway Department specifications and was produced by Young Brother's Asphalt Company located on Highway 21 west of Bryan, Texas. After casting the base, a layer of OGFG was molded on top of it. A summary of the procedure used for compacting OGFC mixtures is given below.

With the base already inside the mold, the OGFC material was introduced as soon as the mixture reached the molding temperature. The mold, with its contents, were placed at the center of the platen of the compactor. The compactor head was then lowered onto the material. The lift cam was adjusted to provide an angle of gyration of  $1^\circ$ . An initial pressure of 35 psi was induced and the specimen gyrated for two minutes at that pressure. Then the load was increased to 69 psi and gyration continued for another two minutes. Then the load was brought up to 104 psi and gyration continued until the pressure gauge needle remained constant for five revolutions of the platen. The gyratory press was turned off and the cam lift returned to its original position which reduced the angle lift to zero. The specimen was reloaded to 35 psi and given a few additional revolutions to square-up the end faces. A final pressure of 865 psi was induced on the specimen and maintained until the rate of consolidation was 0.005 in/min (.002 cm/sec) or less over a five minute period. The specimen was subsequently unloaded and removed from the compactor.

## CHAPTER IV

### LABORATORY EVALUATION PROCEDURES

At the outset of the laboratory evaluation, a factorial design matrix was set up to evaluate the effects of the different design variables on the mechanical behavior of the mixture. The design variables in this program included binder type, aggregate type, asphalt viscosity, and sulfur content in the binder. The factorial design used in this study is given in Figure 7.

Figure 8a shows the schematic diagram of the laboratory testing procedure performed on each aggregate and aggregate mixture. The first few items, up to and including the drainage test, have been discussed in the previous chapter and will not be repeated here. The standard test procedures for the determination of engineering properties are indicated in Figure 8b. Non-standard tests such as permeability and freeze-thaw are discussed below.

#### Experimental Design

A factorial statistical experimental design was conducted in this study to eliminate any redundancy in testing the engineering properties of the mixtures. For example, the factorial allows an evaluation of the effects of asphalt viscosity, aggregate type, percent sulfur and the blending method used. In addition, the effects of interaction of these parameters may also be evaluated. Four standard engineering property tests were used in this evaluation: (1) Marshall, (2) Hveem, (3) resilient modulus,  $M_R$  at 77°F (25°C) and (4) resilient modulus,  $M_R$  at 68°F (20°C). Based on this factorial design, only two of the three aggregates, the East Texas Sandstone and the Streetman lightweight aggregate, needed to be analyzed.

#### Permeability

Permeability is measure of the internal drainage capacity of different mixes. The test results can be used to estimate the maximum rainfall intensity the pavement is able to handle before flooding begins.

The permeability apparatus developed for this project is similar to the one employed by the U. S. Army Waterway Station (33). The apparatus basically consists of a plastic standpipe with a 2.75 in (6.98 cm) inside diameter and a 7 in. (17.78 cm) diameter metal

Factor	Level or Type			
	1	2	3	4
Binder Type	Sulfur Asphalt Emulsion	Sulfur Asphalt Direct Addition	————	————
Aggregate Type	Quartzite (East Texas Sandstone)	Coarse- Grained Limestone	Expanded Shale or Slag (Streetman)	————
Viscosity of AC	AC-10	AC-20	AC-40	————
Ratio of S/A	0/100	20/80	30/70	40/60

Figure 7. Experimental Design Variables for Evaluation of SEA-OGFC.

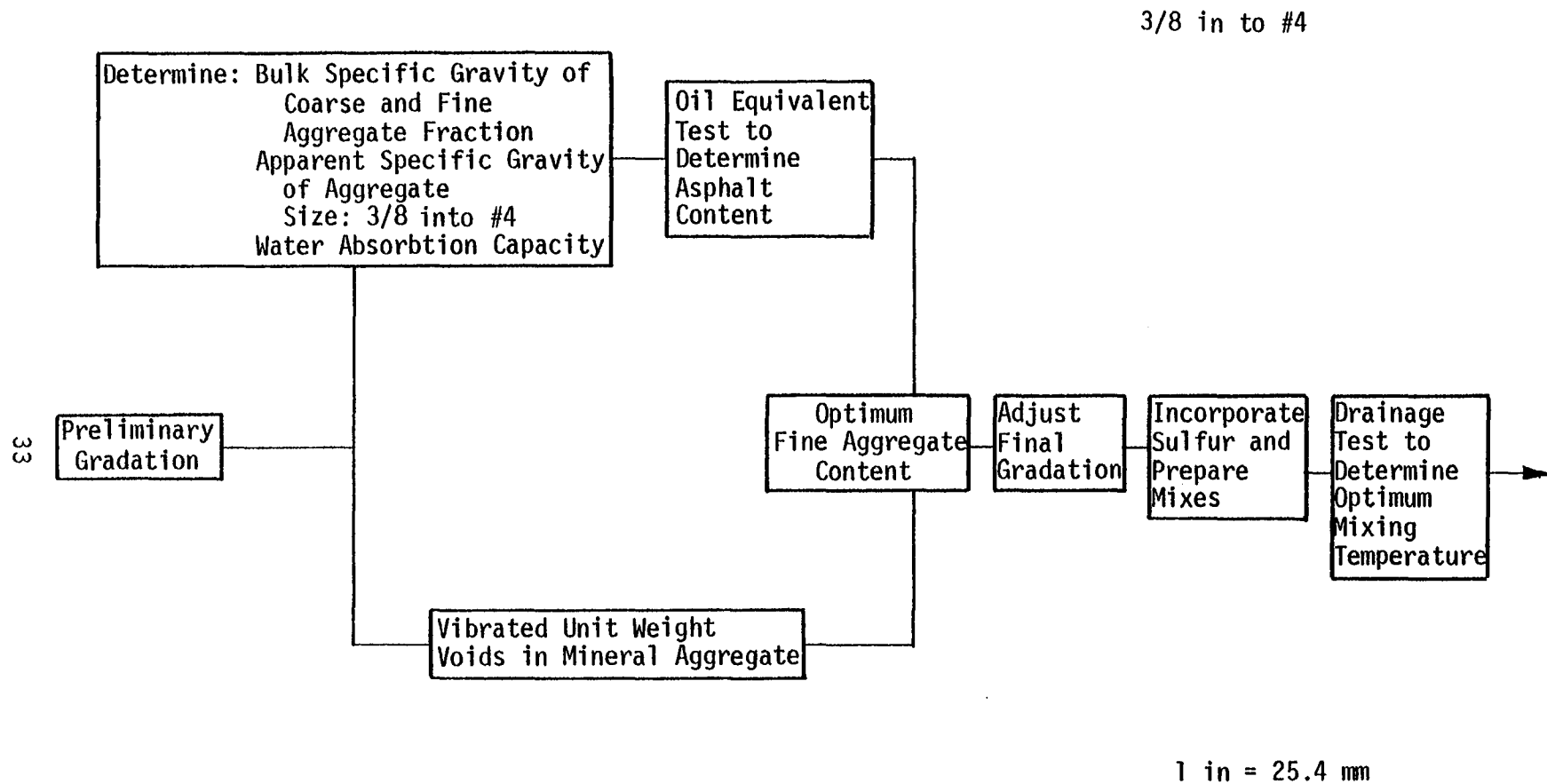
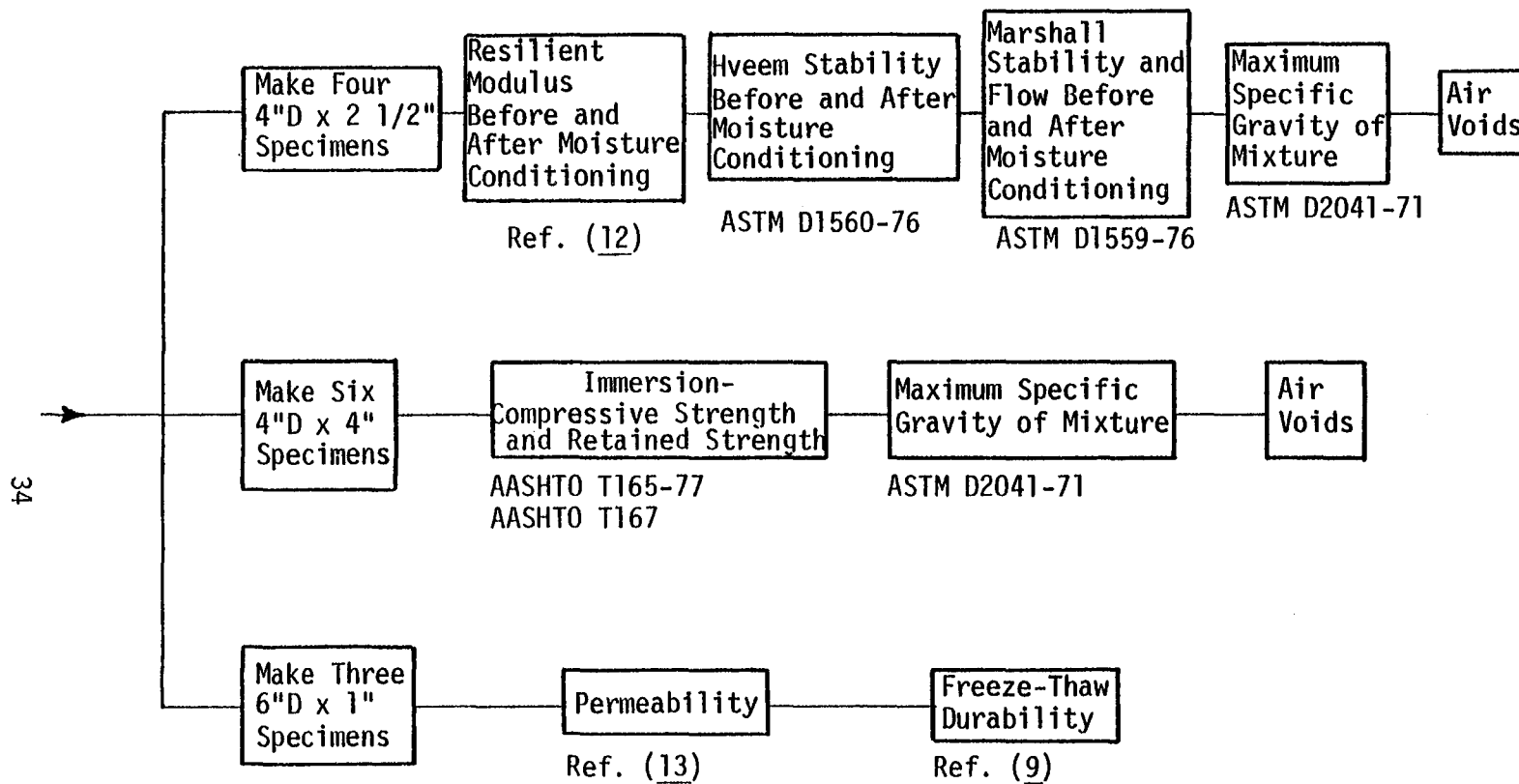


Figure 8a. Preliminary Laboratory Evaluation Procedure for Each Aggregate



1 in = 25.4 mm

Figure 8b. Procedure for Laboratory Evaluation of Structural and Performance Characteristics for Each Aggregate Mixture



standpipe base. A "silicone-sponge rubber" gasket is used as a seal between the standpipe base and the specimen surface. Tests are conducted on 6 in (15.24 cm) diameter samples.

The procedure employed to measure permeability is called the "Variable Head Test". Two timing marks are located at a pre-established distance apart on the face of the standpipe, and the time required for the water to flow from one mark to the other is recorded as "time to fall". Together with the area perpendicular to the flow path and the respective distance of the two timing marks from the specimen surface, a coefficient of permeability ( $K_v$ ), presented in units of cm/sec, is computed as an index of the drainage handling capacity of the specimen. The equation for finding the coefficient of permeability is as follows:

$$\text{Coefficient of Permeability: } K_v = 2.3 \frac{aL}{tA} \log_{10} \frac{h_0}{h_1}$$

where

$a$  = cross-sectional area of standpipe,  $\text{cm}^2$ ;

$L$  = length of flow path, cm;

$A$  = area perpendicular to flow path,  $\text{cm}^2$ ;

$t$  = time for water level to fall from  $h_0$  to  $h_1$ , seconds;

$h_0, h_1$  = the heads between which the permeability is determined, cm;

$$L/A = \frac{0.00326z^2 + 0.525/z + 0.117}{(z/2 + 5.87)} \text{ cm}^{-1};$$

$z$  = thickness of the OGFC layer

A detail description of the method and the theoretical derivation of the formula is given in Appendix F (13).

### Freeze-Thaw Durability

The Freeze-Thaw Durability test method was developed by Gallaway and Epps (9) and used to measure qualitatively the relative resistance of different mixes to rapidly repeated cycles of freezing and thawing in water.

The dimensions of the specimens are the same as those used in the permeability tests mentioned above. The test specimens are placed in a freeze-thaw chamber and subjected to 100 cycles of freezing and thawing between temperatures of 40°F (4.4°C) and 0°F (-17.8°C). Tests are run in triplicate at a rate of 6 cycles per day. The specimens are examined visually before and periodically during the test, and signs of degradation or aggregate loss are recorded.

The visual observations were reduced to numerical values by using double index code numbers. The first number tells the type of damage as follows:

- 0 = no damage visible;
- 1 = used only for beginning observation - aggregate not coated, damage not due to wear;
- 2 = used where cracks or breaks are observed in the aggregate, and aggregate loss is visible;
- 3 = used where aggregate loss in part or total has occurred

The second code-number is the percent of the aggregate affected. Example: 2-3 is fractured aggregate with 3 percent affected.

After removing samples from freeze thaw and all observations have been made, each sample is weighed to the nearest 0.1 g. ( $2.2 \times 10^{-4}$  lb). Then the sample is secured by any convenient means that will not in itself cause damage to the sample. With a "Scratch Brush", 3 overlapping strokes are applied to surface of the sample. The force used is 3 lb. (1.4 kg). This can be achieved by adding the required weight to the brush to equal 3 lbs. (1.4 kg). The brush only need be pushed or pulled across the sample with no downward force. The final weight is recorded and subtracted from original weight for test value.

The wire scratch brush used in this particular test is made of 2 by 1/16 in. (5.08 cm x 0.16 cm) flat No. 26 gauge wire bristles assembled in 60 groups of 10 bristles each and mounted to form 5 longitudinal rows and 10 transverse rows of bristles on a 7 1/2 in. by 2 1/2 in. (19.05 cm x 6.35 cm) hardwood block.

## CHAPTER V

### DATA ANALYSIS

This chapter contains a discussion of the results of the evaluation tests shown in Figure 8b. First, an Analysis of Variance was conducted to evaluate the effect of sulfur-asphalt blending method and process parameters (asphalt viscosity, sulfur content, aggregate type, etc.). The data from the evaluation tests are given in Appendix H.

#### Analysis of Variance Study

An Analysis of Variance (ANOVA) study was performed to examine the relative influence that each of the above parameters had on SEA-OGFC mixture properties or performance characteristics. This was done to eliminate any possible redundancy in the experiments, as for example the need to investigate both direct and preblended binder preparation.

Three sulfur-asphalt blending methods including two different direct-substitution methods (direct and modified-direct) and a preblending (emulsion) method were employed to produce the SEA binder for use in the preparation of specimens for this statistical analysis, (see Chapter III).

In order to determine whether or not the method of blending sulfur with asphalt has an effect on mixture properties, four factorial statistical experiments were conducted (Figure 9); which allowed the evaluation of the effects of variations of four factors (i. e., asphalt viscosity, aggregate type, percent sulfur and the blending method used). In addition the effects of interaction of these parameters were evaluated. Four standard engineering property tests were used in this evaluation: (a) Marshall stability, (b) Hveem stability; (c) resilient modulus,  $M_R$  at 68°F (20°C); and (d) resilient modulus,  $M_R$  at 77°F (25°C). With this factorial design, only two of the three aggregates, the East Texas Sandstone and the Streetman, needed to be analyzed. The results of the factorial analysis are shown in Table 5.

Aggregate type, asphalt viscosity and percent sulfur in binder were all found to have significant influences on the mixtures' engineering properties. Therefore, these factors must be retained in this study to further evaluate their effects on the SEA-OGFC performance. However, the method of blending sulfur with asphalt

A	B	C	D	Asphalt Grade		Aggregate		w/o Sulfur in Blender		Blending Method	
				AC-10		AC-40					
				Streetman Lt. Wt.	East Texas Sandstone	Streetman Lt. Wt.	East Texas Sandstone				
	D I R E C T	20	1 *	c	d	cd					
		40	b	bc	bd	bcd					
	E M U L S I O N	20	a	ac	ad	acd					
		40	ab	abc	abd	abcd					

Figure 9 . Four-Factor Factorial Analysis

\*Note: In block 1 we have established the control mix. That is a mix of AC-10; 20% sulfur, ST aggregate and direct blending. The lower case letters in the remaining 15 blocks indicate which factor or combination of factors vary from the control in block 1.

Table 5. Summary of Results of Factorial Analysis.

Four Most Significant Factors or Interactions		F Calc.	F Statistic F (Tabulated) *	Is Factor Statistically Significant
Marshall	Aggregate	162.5	4.5	Yes
	AC Viscosity	67.8	4.5	Yes
	% Sulfur	21.5	4.5	Yes
	Interaction of Above	4.5	4.5	Yes
Hveem	Blending Method	0.11	4.5	No
	AC Viscosity	0.09	4.5	No
	Aggregate	0.09	4.5	No
	Interaction of Above	0.09	4.5	No
36 $M_R$ @ 77°F	% Sulfur	63.2	4.5	Yes
	AC Viscosity	10.0	4.5	Yes
	% Sulfur & Aggregate	8.6	4.5	Yes
	Aggregate	3.2	4.5	Yes
$M_R$ @ 68°F	% Sulfur	216.2	4.5	Yes
	AC Viscosity	26.2	4.5	Yes
	Interaction of AC viscosity % Sulfur and Blending Method	23.2	4.5	Yes
	Interaction % Sulfur and Blend Type	22.1	4.5	Yes
Marshall	Blending Method	0.07	4.5	No
Hveem	Blending Method	0.11	4.5	No
$M_R$ @ 77°F	Blending Method	2.79	4.5	No
$M_R$ @ 68°F	Blending Method	1.63	4.5	No

\*  $F(\alpha = .05, v_1 = 1, v_2 = 16)$

(direct or emulsion) proved to have no statistical significance. All of the findings were based on a level of significance of 95 percent.

An additional conclusion provided by these analyses was that the Hveem test is insensitive to the mixture variables being tested. The reason for this insensitivity is that the Hveem tests reflect the mixtures' internal friction and as such is primarily dependent on the percent of asphalt or binder and aggregate shape and gradation.

As a result of the above analysis, the emulsification blending process was eliminated from this study. In addition, the modified direct substitution seems to yield data which are much closer to those obtained from corresponding emulsified mixes. Therefore, the modified-direct substitution method, as proposed by the Bureau of Mines, was used to prepare all mixes for the direct mix-blend phase of the program.

#### Analysis of East Texas Sandstone SEA-OGFC Mixtures

Structural characterization. Although SEA-OGFC are not considered to be a structural component of the pavement, it is expected to possess certain structural properties so that load can be transferred through the layer to the base without causing any excessive deformation to the surface layer.

The characterization of the mixes prepared with this as well as the other two aggregate systems were based on examining the influence of sulfur content and asphalt grade on the following properties:

- (1) Marshall Stability
- (2) Marshall Flow
- (3) Hveem Stability
- (4) Resilient Modulus at
  - T = 34°F (1°C)
  - T = 68°F (20°C)
  - T = 77°F (25°C)
- (5) Air Voids
- (6) Voids in Mineral Aggregates
- (7) Index of Retained Strength

The results of these tests are given in Figures 10 to 23.

Figure 10 shows the Marshall stabilities with respect to sulfur content in SEA binder. For all asphalt grades, the figure indicates that SEA-OGFC mixtures consistently possess higher stabilities than

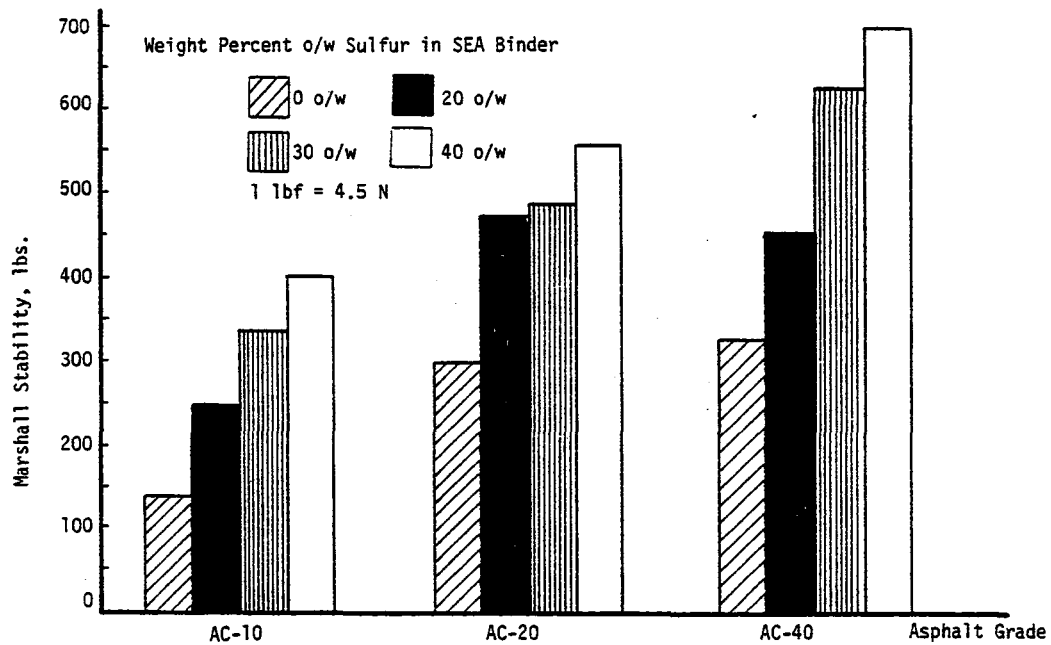


Figure 10. Marshall Stability vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

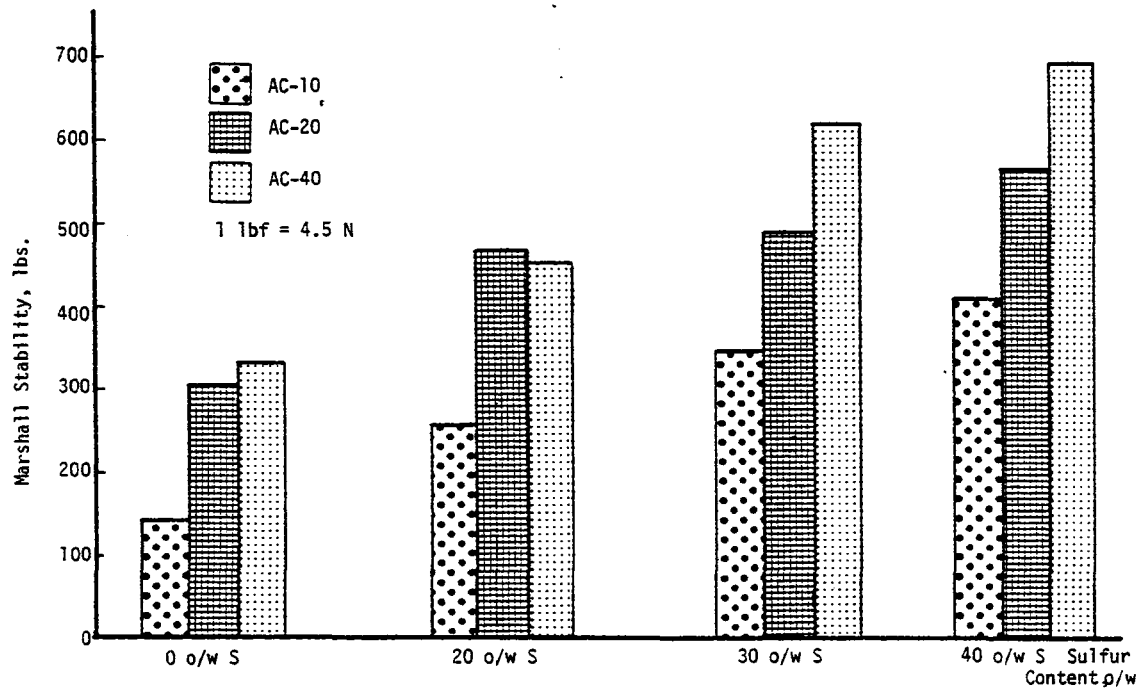


Figure 11. Marshall Stability vs Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures.

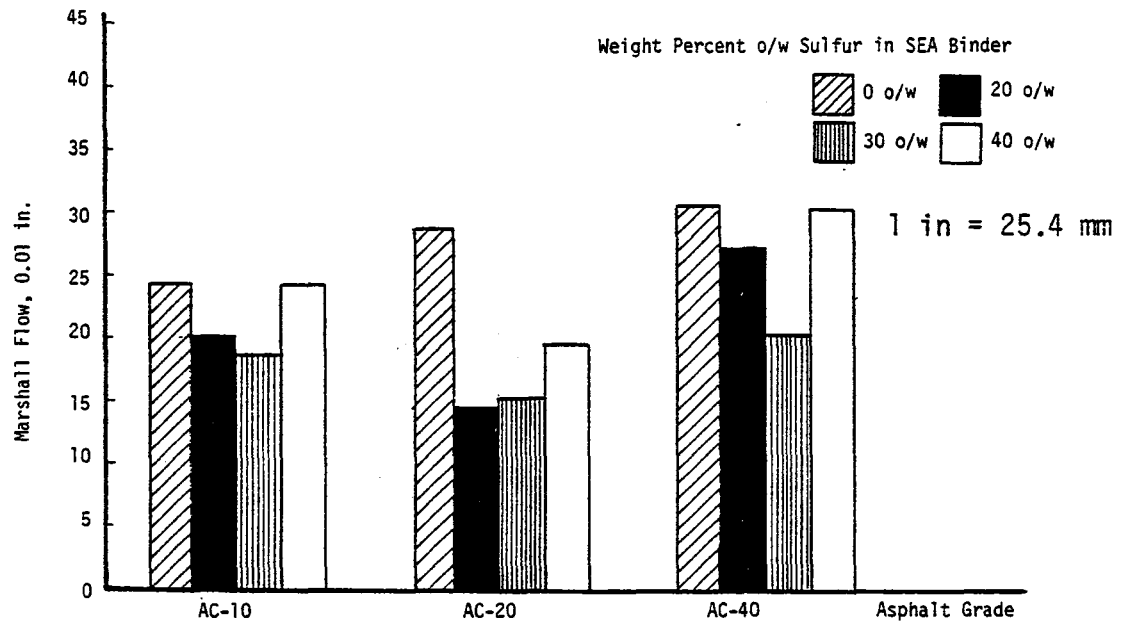


Figure 12. Marshall Flow vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

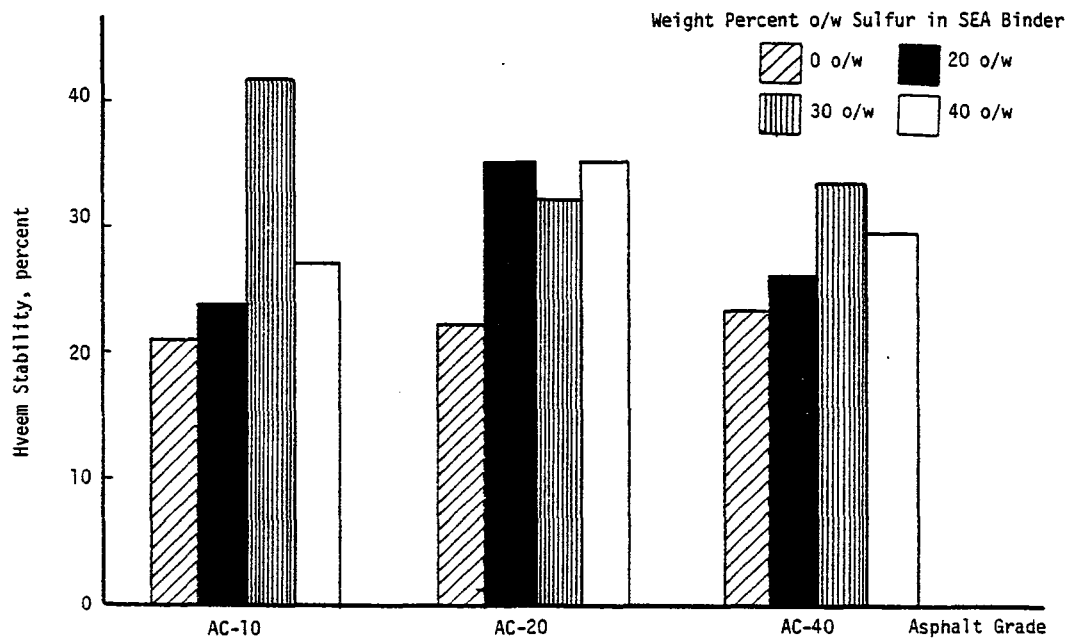


Figure 13. Hveem Stability vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.



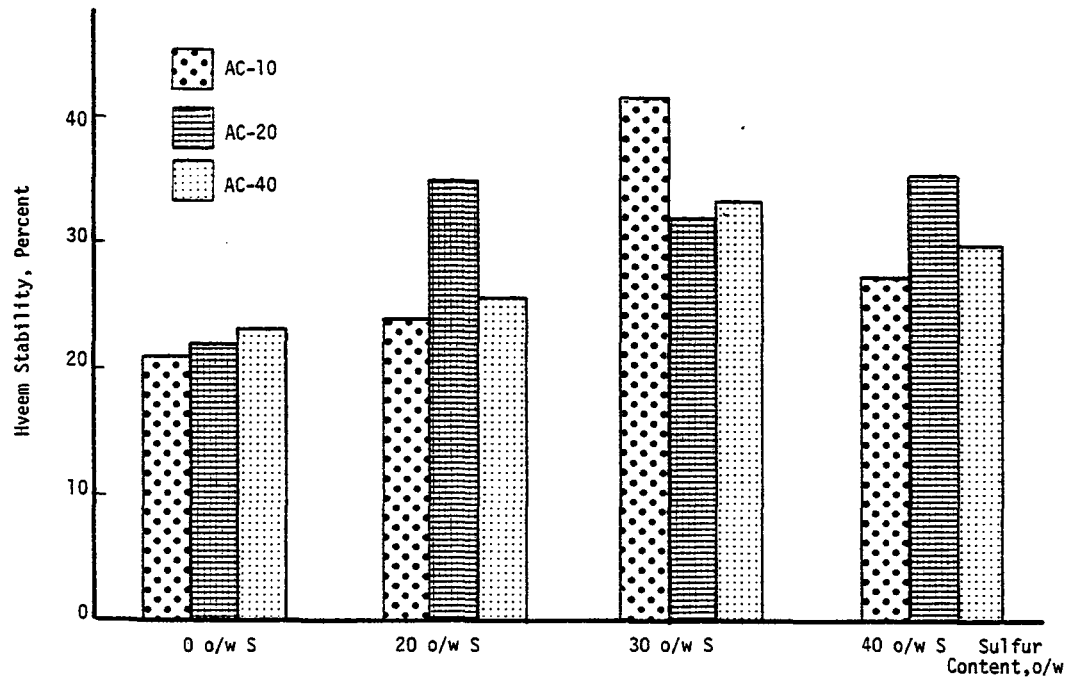


Figure 14. Hveem Stability vs AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures.

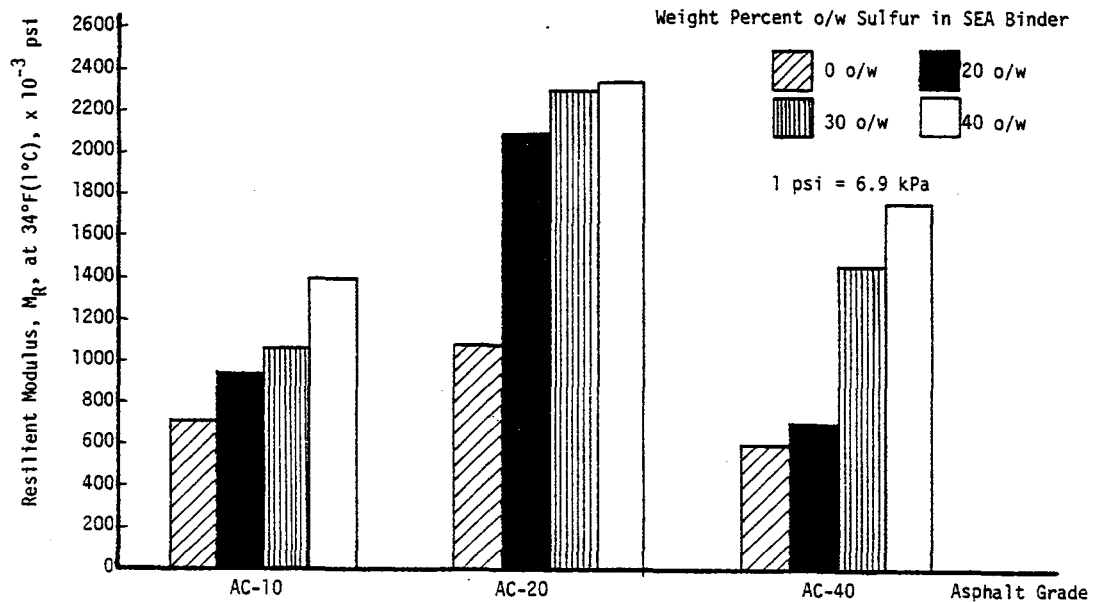


Figure 15. Resilient Modulus at 34°F (1°C) vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

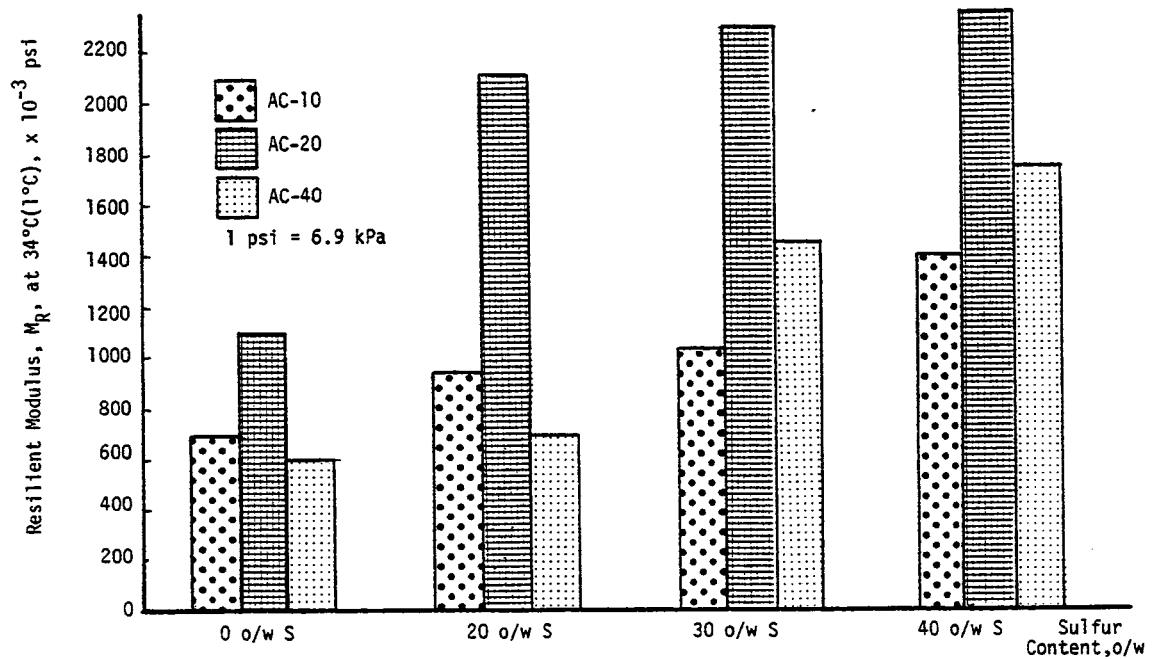


Figure 16. Resilient Modulus at 34°F (1°C) vs AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures.

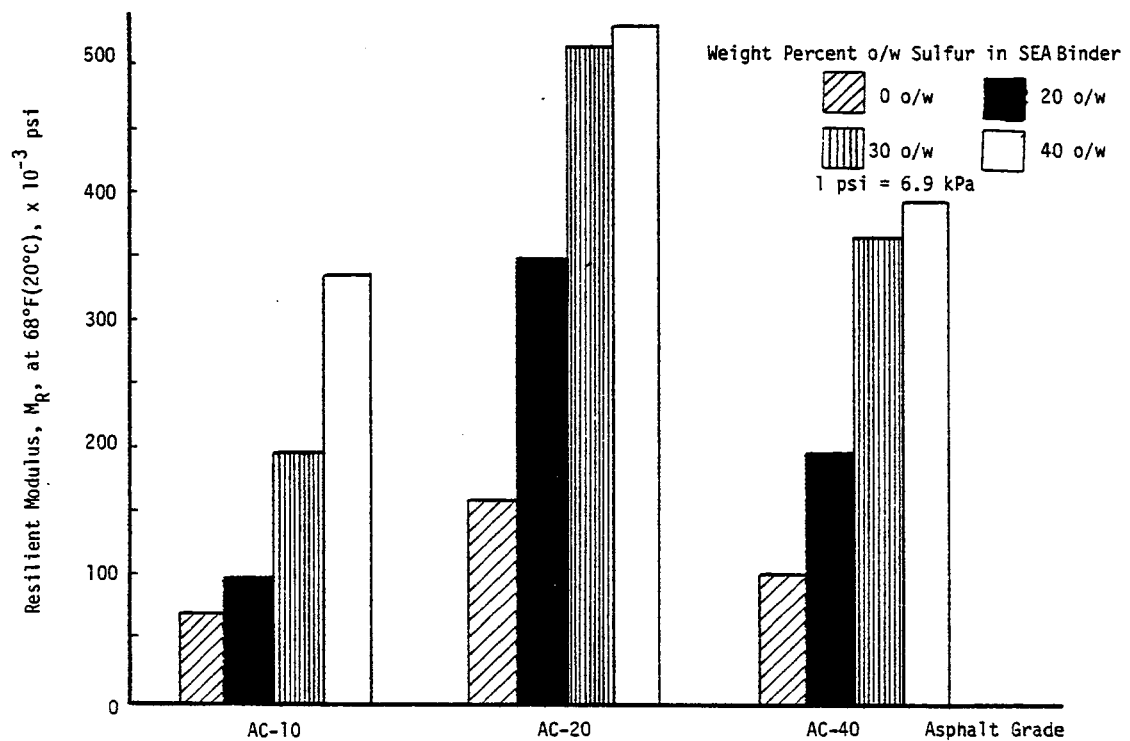


Figure 17. Resilient Modulus at 68°F (20°C) vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

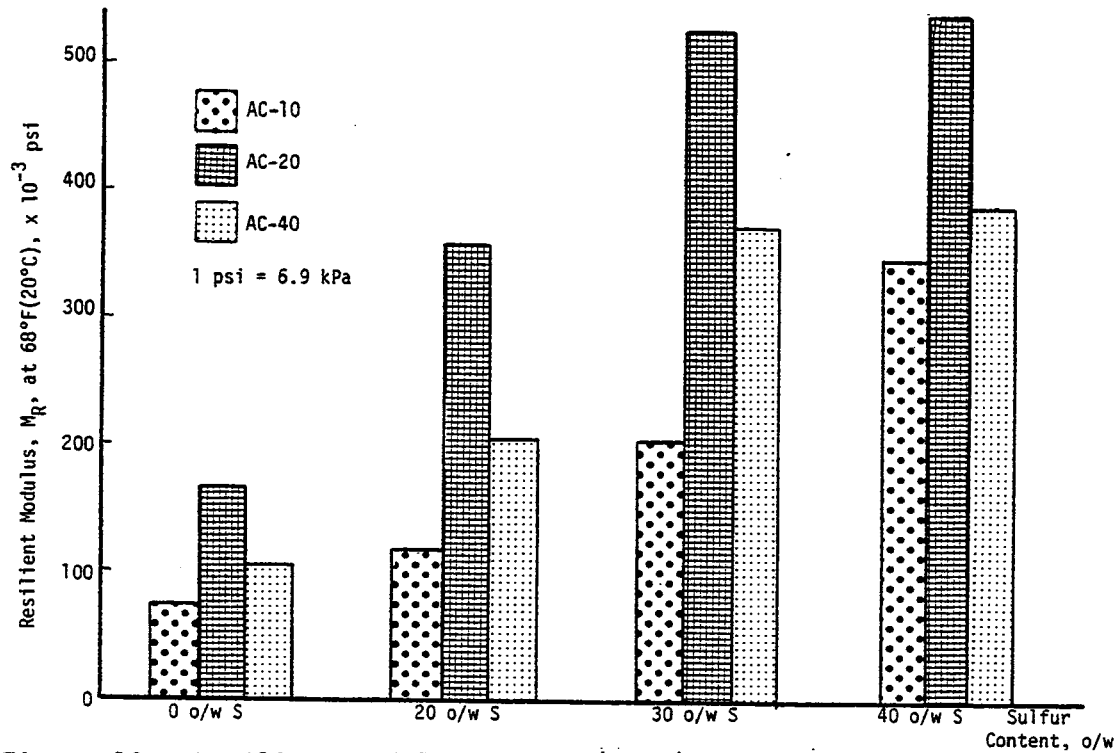


Figure 18. Resilient Modulus at 68°F(20°C) vs AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures.

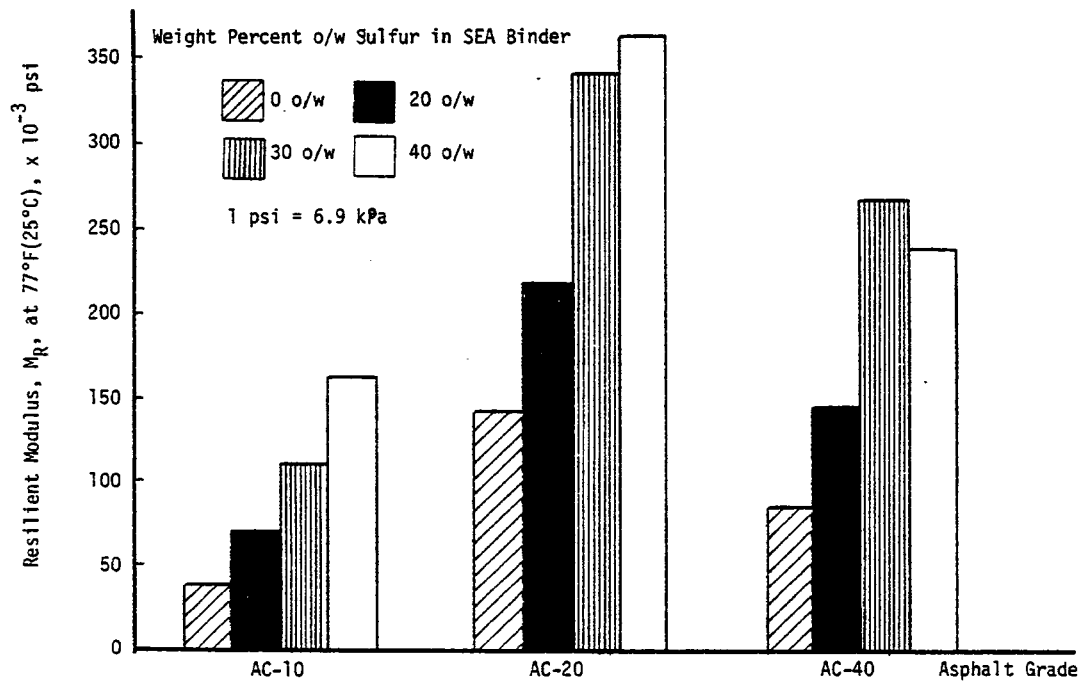


Figure 19. Resilient Modulus at 77°F (25°C) vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

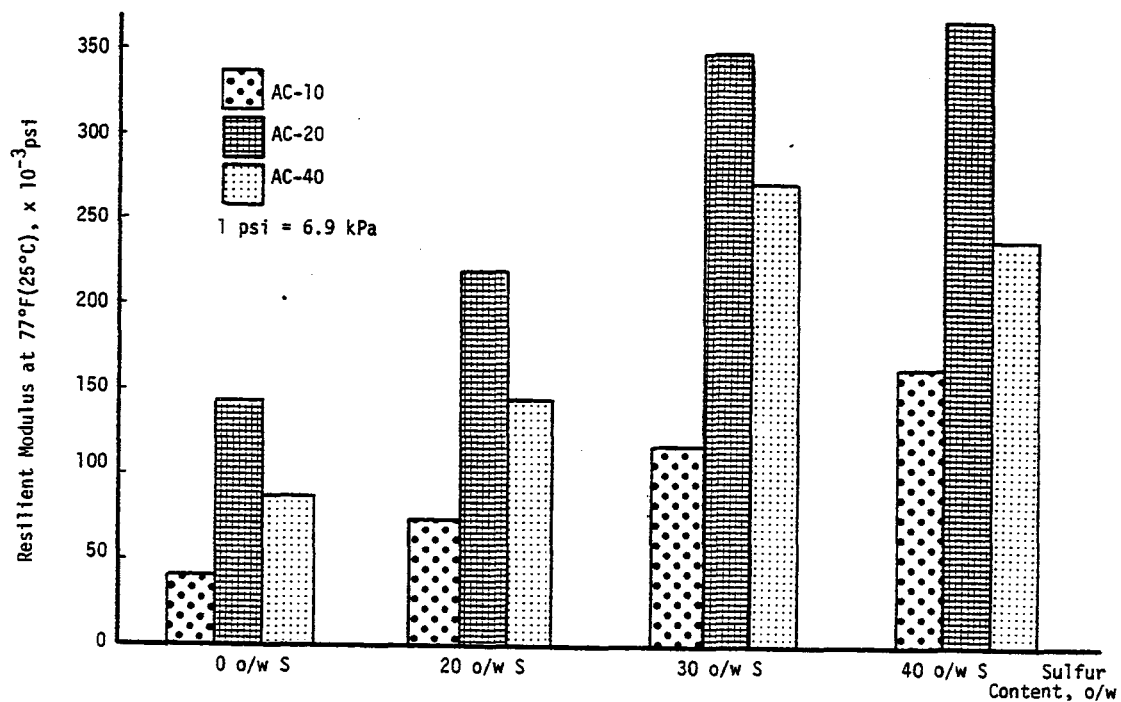


Figure 20. Resilient Modulus at 77°F(25°C) vs AC Grade with Corresponding Sulfur Content for East Texas Sandstone SEA-OGFC Mixtures.

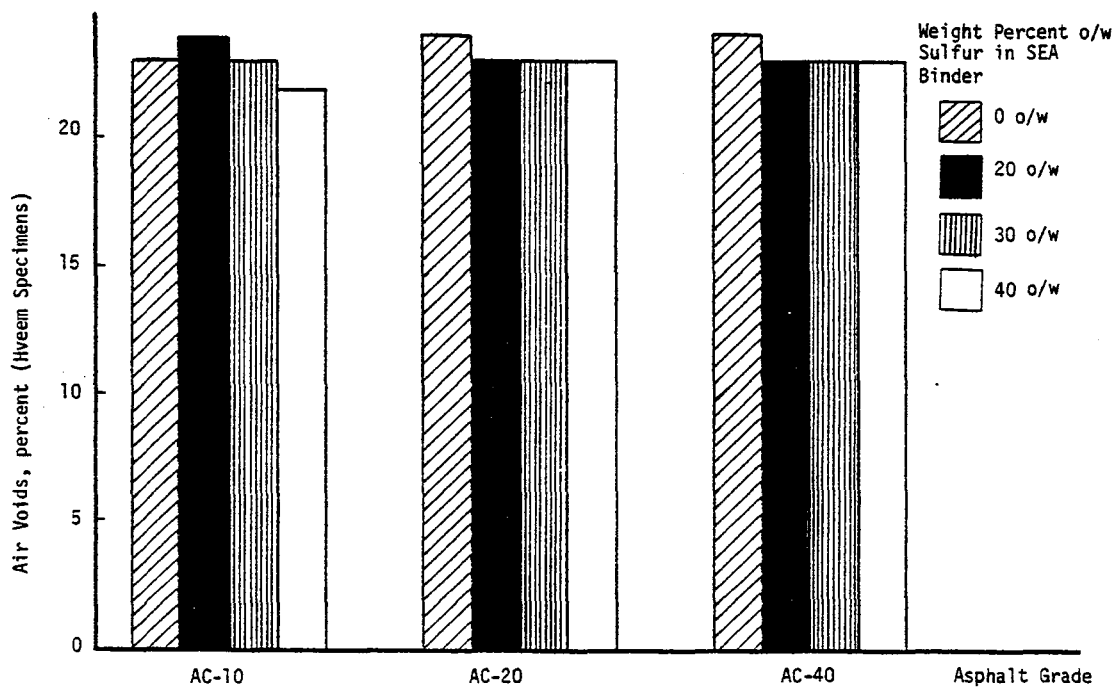


Figure 21. Air Voids vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

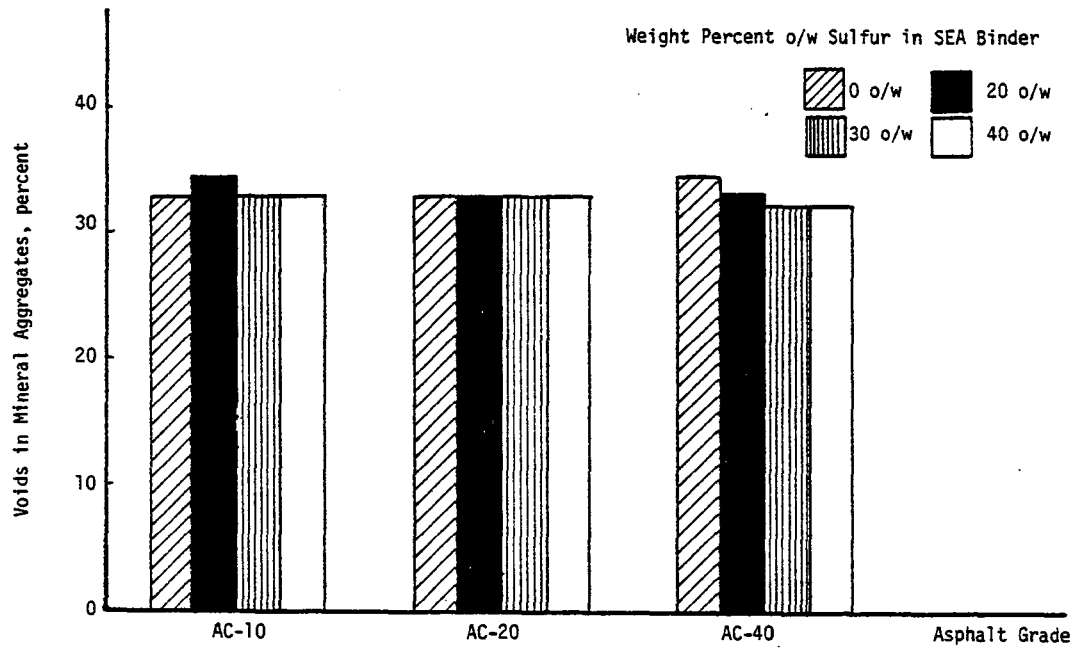


Figure 22. Voids in Mineral Aggregates vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

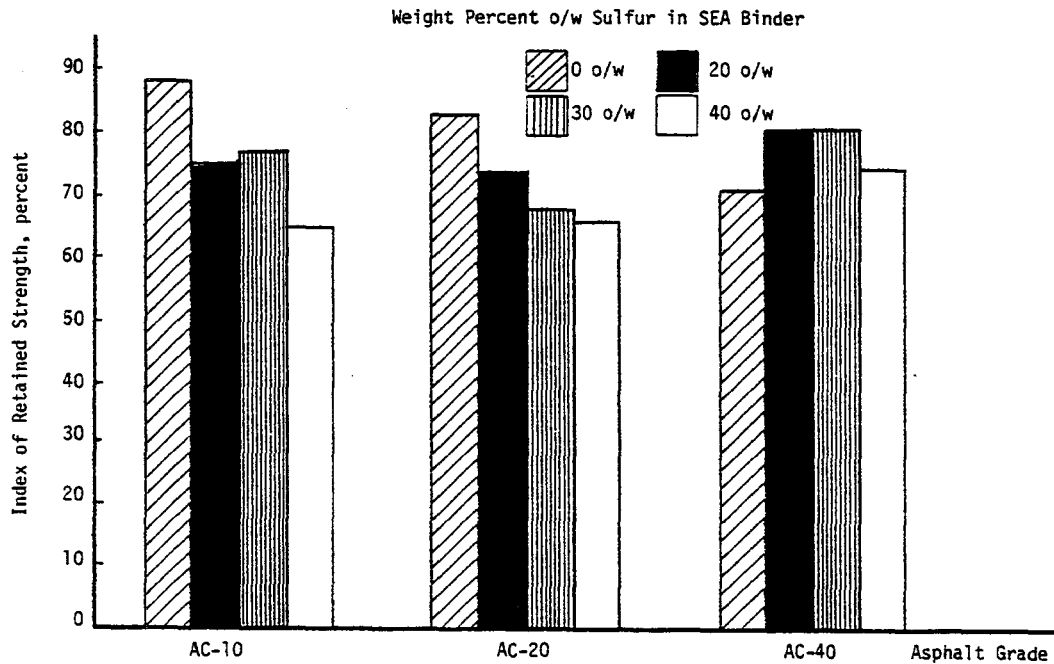


Figure 23. Index of Retained Strength vs Sulfur Content with Corresponding Asphalt Grade for East Texas Sandstone SEA-OGFC Mixtures.

those prepared with pure asphalt binders. This result agrees with the findings in other sulfur-asphalt mixtures (25, 26, 42). In addition, an increase in sulfur content in the binder gives a corresponding increase in the Marshall stability. This may indicate that sulfur adds to the structural bonding strength of the mixture. Figure 10 also seems to indicate that a mixture prepared with a S/A weight ratio of 40/60 would have a Marshall stability as much as 100 percent higher than a pure asphalt OGFC mixture.

Figure 11 shows that, with the same percent sulfur in SEA binder, mixtures prepared with a higher viscosity asphalt will result in higher stabilities. This result is logical because the higher viscosity should provide more stiffness to the mixture.

From Figures 10 and 11, it can be seen that the SEA mixture prepared with AC 40 and 40 weight percent sulfur possesses the highest Marshall stability of 690 lbs. (3105 N). This result is logical because both an increase in sulfur content and asphalt viscosity should be expected to cause an increase in Marshall stability.

It is also obvious that the majority of the Marshall stabilities, particularly those mixtures prepared with an AC 10 or AC 20, fall short of the minimum value specified by The Asphalt Institute (43). However, since the OGFC is not expected to function as a structural component of the pavement, a lower value of Marshall stability value can be tolerated.

Figure 12 presents the Marshall flow values versus weight percent sulfur in SEA binder. It can be seen that the data scatter was so great that a direct correlation between flow values and percent sulfur in binder could not be drawn. However, SEA-OGFC mixtures prepared with AC 20 have consistently lower flow values than those prepared with AC 10 or AC 40 and corresponding sulfur contents. In addition, the flow values of the SEA-OGFC mixtures prepared with AC 20, values between 14 and 19, fall within the range of flow values (8-20) recommended by The Asphalt Institute (43). It is also apparent that SEA-OGFC mixture exhibit consistently lower flow values than those prepared with pure asphalt binders.

The Marshall stability and flow results indicate that SEA-OGFC mixtures prepared with AC 20 exhibit comparatively high stabilities and yet fall within the range of flow values suggested by The Asphalt Institute (43).

Figure 13 shows the relationship between Hveem stability and sulfur content in SEA binder. All SEA-OGFC mixtures prepared with AC 20 shows favorable Hveem stabilities, values between 32 to 35,

those prepared with pure asphalt binders. This result agrees with the findings in other sulfur-asphalt mixtures (25, 26, 42). In addition, an increase in sulfur content in the binder gives a corresponding increase in the Marshall stability. This may indicate that sulfur adds to the structural bonding strength of the mixture. Figure 10 also seems to indicate that a mixture prepared with a S/A weight ratio of 40/60 would have a Marshall stability as much as 100 percent higher than a pure asphalt OGFC mixture.

Figure 11 shows that, with the same percent sulfur in SEA binder, mixtures prepared with a higher viscosity asphalt will result in higher stabilities. This result is logical because the higher viscosity should provide more stiffness to the mixture.

From Figures 10 and 11, it can be seen that the SEA mixture prepared with AC 40 and 40 weight percent sulfur possesses the highest Marshall stability of 690 lbs. (306 N). This result is logical because both an increase in sulfur content and asphalt viscosity should be expected to cause a increase in Marshall stability.

It is also obvious that the majority of the Marshall stabilities, particularly those mixtures prepared with an AC 10 or AC 20, fall short of the minimum value specified by The Asphalt Institute (43). However, since the OGFC is not expected to function as a structural component of the pavement, a lower value of Marshall stability value can be tolerated.

Figure 12 presents the Marshall flow values versus weight percent sulfur in SEA binder. It can be seen that the data scatter was so great that a direct correlation between flow values and percent sulfur in binder could not be drawn. However, SEA-OGFC mixtures prepared with AC 20 have consistently lower flow values than those prepared with AC 10 or AC 40 and corresponding sulfur contents. In addition, the flow values of the SEA-OGFC mixtures prepared with AC 20, values between 14 and 19, fall within the range of flow values (8-20) recommended by The Asphalt Institute (43). It is also apparent that SEA-OGFC mixtures exhibit consistently lower flow values than those prepared with pure asphalt binders.

The Marshall stability and flow results indicate that SEA-OGFC mixtures prepared with AC 20 exhibit comparatively high stabilities and yet fall within the range of flow values suggested by The Asphalt Institute (43).

Figure 13 shows the relationship between Hveem stability and sulfur content in SEA binder. All SEA-OGFC mixtures prepared with AC 20 shows favorable Hveem stabilities, values between 32 to 35,

and all are above the minimum stability value set forth by The Asphalt Institute (44).

Figure 14 does not indicate any particular trend of Hveem stability with respect to asphalt viscosity. However, the data do shows that Hveem stabilities are consistently higher in the SEA-OGFC mixtures. The SEA-OGFC mixtures have Hveem stabilities in the range of 24 to 42 percent, compared to the range of 21 to 23 percent for pure asphalt OGFC mixtures. This is the result of structural bonding strength due to the introduction of sulfur in the binder.

Figures 15 and 16 indicate that the resilient modulus at the low temperature (34°F (1°C)) increases with the same AC with increasing sulfur content in the binder. However, the degree of the effect of sulfur apparently depends on the asphalt viscosity. For mixtures with AC 10 and AC 20, a 40 percent sulfur substitution in the binder will increase the Resilient Modulus by 100 percent, while in the case of mixtures prepared with AC 40, the resulting increase in resilient modulus for a 40 percent sulfur substitution in the binder is as much as 200 percent. In Figure 16, it can also be seen that resilient modulus of SEA-OGFC mixtures prepared with AC 20 range from 2,100,000 - 2,350,000 psi ( $14.5 \times 10^6$  -  $16.2 \times 10^6$  kPa) are superior to those of mixtures prepared with the other two asphalt cements which range from 700,000 - 1,750,000 psi ( $4.8 \times 10^6$  -  $12.1 \times 10^6$  kPa).

Figures 17 and 18 present the resilient modulus values at 68°F (20°C) at various sulfur contents and asphalt viscosities. The test results support the findings in Figure 15 and 16 which indicate that an increase in sulfur content in the binder will cause a corresponding increase in resilient modulus, and SEA-OGFC mixtures prepared with AC 20 seem to have higher resilient moduli to those prepared with AC 10 or AC 40. This indicates that SEA binder with AC 20 may provide the mixture with the optimum properties. In addition, SEA-OGFC mixtures prepared with AC 20 display resilient modulus values comparable to other sulfur asphalt mixtures (25, 26, 45, 46, 47).

Resilient modulus at 77°F (25°C) versus sulfur content in the SEA binder and asphalt viscosity, respectively, are shown in Figure 19 and 20. Data further confirm the above findings.

Figure 21 shows the effect that sulfur content has on the mixture's air voids. It is obvious from the distribution of the data that the amount of sulfur introduced to replace the asphalt cement does not affect the amount of air voids produced in the mixture. The range of air voids content in the mixture is between 22 to 24 percent.



The voids in mineral aggregates (VMA) versus sulfur content in SEA binder is given in Figure 22. The VMA which is around 33 percent is about the same for all mixtures tested. The reason is that VMA is the volume of intergranular void space between the aggregate particles of a compacted mixture and therefore is a function of aggregate shape and gradation and the degree of compaction.

Figure 23 shows how the trend of the Index of Retained Strength (IRS) varies with respect to sulfur content in the binder. It can be seen that an increase in sulfur content in the binders leads to a decrease in percent retained strength in the mixtures with the exception of the mixes prepared with the AC-40 cement. No reason can be offered for this inconsistency. The percent reduction in IRS is in the range of 15 to 25 units. However, all the mixtures tested had more than 60 percent retained strength which meet the minimum value requirement proposed by FHWA (1).

After obtaining the results of the above analysis, a series of preliminary screening tests were conducted to eliminate any mixes which did not exhibit adequate mechanical behavioral characteristics for use in highway pavement surfaces.

The preliminary screening program included the following characterization tests:

1. Marshall Stability and Flow (ASTM D1559)
2. Hveem Stability (ASTM D1560)
3. Resilient Modulus at 68°F (20°C) (12)
4. Index of Retained Strength (AASHTO T167)

Since the Marshall and Hveem tests are primarily used for dense-graded hot-mix mixtures and since the SEA-OGFC is not to function as a structural component of the pavement, the published criteria of the two evaluation tests will not be used in this study to evaluate the SEA-OGFC mixtures. Instead, a set of evaluation criteria were arbitrarily established in this SEA-OGFC study to provide a means of minimizing excessive deformation of the mixture. Although there is no standardized requirement on resilient modulus, a minimum value was established to insure that adequate stiffness is maintained in the mix. For the compressive-strength test, the percent retained strength requirement varies from 50 to 75 percent by various agencies. Therefore, a nominal value was chosen to evaluate the SEA-OGFC mixtures.

The resulting arbitrarily selected evaluation criteria for the screening tests are listed below:

Marshall stability, lb.	300, min.
Flow, .01 in.	8-20
Hveem stability, percent	30, min.
Resilient modulus @ 68°F (20°C), psi	300,000, in.
Index of Retained Strength, percent	60, min.
<hr/>	
1 lb. = 4.5 N	1 in. = 25.4 mm
	1 psi = 6.9 kPa

Those SEA-OGFC mixtures using the East Texas Sandstone aggregate which meet the above requirements are listed in Table 6.

All the above SEA-OGFC mixtures which met the requirements were further subjected to permeability and freeze-thaw durability tests to optimize the mix design.

Optimization tests. Permeability tests are used to measure the internal drainage capacity of the different mixes. The test results are given in terms of a coefficient of permeability,  $K_v$ , which is used to compute the maximum rainfall intensity the pavement is able to handle before flooding begins. The data obtained from these tests are shown in Table 7.

In order to relate the coefficient of permeability ( $K_v$ ), to the predicted flooding rainfall intensity,  $I_f$ , the following formula is employed (see Appendix C in Reference 13):

$$K_f = Z [0.948 K_v + 0.021] S^{1/2} / 2.54L \text{ in/hr}$$

where

$I_f$  = average rainfall intensity where incipient flooding occurs, in/hr,

$Z$  = thickness of drainage test specimen, cm,

$K_v$  = permeability coefficient, cm/sec x 3600 = cm/hr.

$S$  = slope of the channel, cm/cm, and

$L$  = flow length or lane width, cm.

Table. 6. Preliminary Screening Test Results for East Texas Sandstone SEA-OGFC Mixtures.

Asphalt Grade	o/w S in Binder	Marshall Stability, lb.	Marshall Flow, .01 in.	Hveem Stability, percent	Resilient Modulus @ 68°F (20°C), psi	Index of Retained Strength, Percent
AC 10	30	340	18	42	200,000*	76
AC 20	20	470	14	35	350,000	72
AC 20	30	480	15	32	520,000	66
AC 20	40	550	19	35	530,000	64
AC 40	30	610	20	33	370,000	79

\* Number below specified minimum value.

1 lb F = 4.5N

1 in. = 25.4 mm

1 psi = 6.9 kPa

Using the above formula, and assuming a 12 ft. (2.66 m) lane with a 1 in. (2.54 cm) SEA-OGFC layer and a 2 percent cross-slope, the predicted flooding rainfall intensity ( $I_f$ ) were computed and tabulated as follows:

Table 7. Permeability Test Results for East Texas Sandstone SEA-OGFC Mixtures.

Asphalt Grade	o/w S in. Binder	Permeability Coefficient $K_v$ , (cm/sec)	Predicted Flooding Rainfall Intensity $I_f$ , (in/hr)
AC 10	30	.505	.666
AC 20	20	.519	.684
AC 20	30	.400	.527
AC 20	40	.502	.662
AC 40	30	.401	.529

1 in/hr = 0.0007 cm/sec.

In the report by Tomasini (13), OGFC pavements having a predicted flooding rainfall intensity factor of 0.264 or more were found to perform well in the reduction of splash and spray in normal wet weather driving. In addition, OGFC pavements with a permeability coefficient of 0.200 are classified as having a high permeability capacity. Therefore, it is concluded that all the above SEA-OGFC mixtures listed in Table 7 have good and adequate drainage capacities.

After the permeability tests were performed on the specimens, the same specimens were subjected to freeze-thaw testing to evaluate their durability to repeated cycles of freezing and thawing. The test results are tabulated in Table 8. The data indicate that only one of the mixtures tested showed any sign of failure after one hundred cycles of freezing and thawing. Therefore, these mixtures are expected to provide satisfactory performance in the field. Mixtures prepared with AC 40 and 30 percent sulfur SEA binder showed signs of minor aggregate fracture in the specimens.

Optimum mix-design. The optimum SEA-OGFC mix-design was selected on the basis of its ability to provide adequate structural behavior,

Table 8. Freeze-Thaw Test Results for East Texas Sandstone SEA-OGFC Mixtures.

Asphalt Grade	o/w S		0 Cycle	25 Cycles	50 Cycles	75 Cycles	100 Cycles	Brush Test Weight Loss (gm)
55	AC 10	30	(1)	0	0	0	0	0.3
			(2)	0	0	0	0	0.6
			(3)	0	0	0	0	0.8
	AC 20	20	(1)	0	0	0	0	0.4
			(2)	0	0	0	0	0.4
			(3)	0	0	0	0	0.6
	AC 20	30	(1)	0	0	0	0	0.3
			(2)	0	0	0	0	0.2
			(3)	0	0	0	0	0.6
	AC 20	40	(1)	0	0	0	0	0.3
			(2)	0	0	0	0	0.4
			(3)	0	0	0	0	0.4
	AC 40	30	(1)	0	0	0	2-1	2.6
			(2)	2-1	2-1	2-1	3-1	3.8
			(3)	0	0	0	2-1	2.0

1 gm = 0.0022 lb.

good drainage capacity, and high-freeze thaw durability. Emphasis was also placed on maximizing the sulfur content used to reduce the asphalt demand in the SEA binder.

From the data on Marshall stability and flow, it has been shown earlier that SEA mixtures prepared with AC 20 consistently show favorable stabilities and flow values. However, the mixtures prepared with AC 10 and 30/70 SEA binder possesses only a marginal stability of 320 lb. (1440 N) despite of the acceptable flow value. On the other hand, mixtures prepared with AC 40 and 30/70 SEA binder display a comparatively high stability value of 610 lb. (2745 N) with a marginal flow value of 20.

It can also be seen from Table 6 that for 30/70 and 40/60 SEA binders with AC 20, mixtures exhibit comparatively higher resilient moduli of 520,000 and 530,000 psi ( $3.6 \times 10^6$  and  $3.7 \times 10^6$  kPa), respectively. On the contrary, mixtures prepared with AC 10 and 30/70 SEA binder were found to have resilient moduli lower than the specified minimum vlaue. In addition, the test results in Table 7 indicate that all the SEA-OGFC mixtures tested exhibit high coefficients of permeability and all can be expected to adequately handle normal rainfall conditions.

At this point it appears that mixtures prepared with AC 20 and 30/70 or 40/60 SEA binder will perform equally well in the field as would the mixtures prepared with AC 40 and 30/70 binder. However, the results of freeze-thaw tests indicate that mixtures prepared with AC 40 and 30/70 SEA binder begin to show sign of aggregate fracture, which may further lead to aggregate loss and ravelling, after 100 repeated cycles of freezing and thawing. As a result, mixtures prepared with AC 20 and 30/70 or 40/60 SEA binder seem to be slightly more favorable than the one prepared with AC 40 using 30/70 SEA binder.

Since the maximized usage of sulfur content in the binder is also an intended objective, the mixture prepared with AC 20 using 40/60 SEA binder was selected as the optimum SEA-OGFC mix design for East Texas Sandstone aggregate system.

#### Analysis for Limestone SEA-OGFC Mixtures

Structural characterization. The results of the structural characterization of the mixes prepared with this aggregate system are shown in Figures 24 through 37.

Figure 24 presents the results for Marshall stability versus sulfur content for limestone SEA-OGFC mixtures. The data shows a

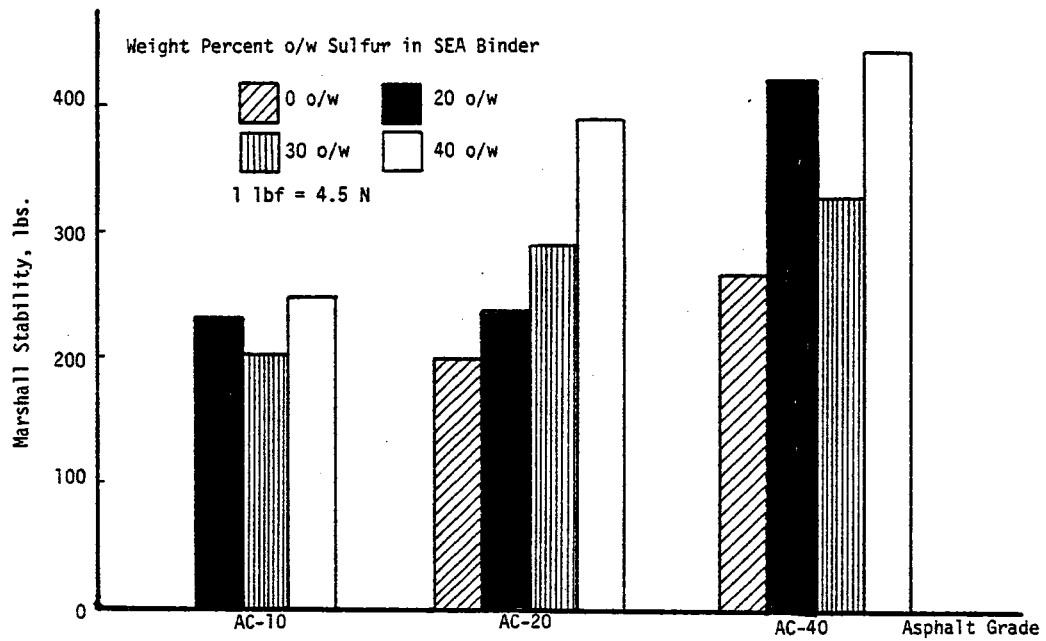


Figure 24. Marshall Stability vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

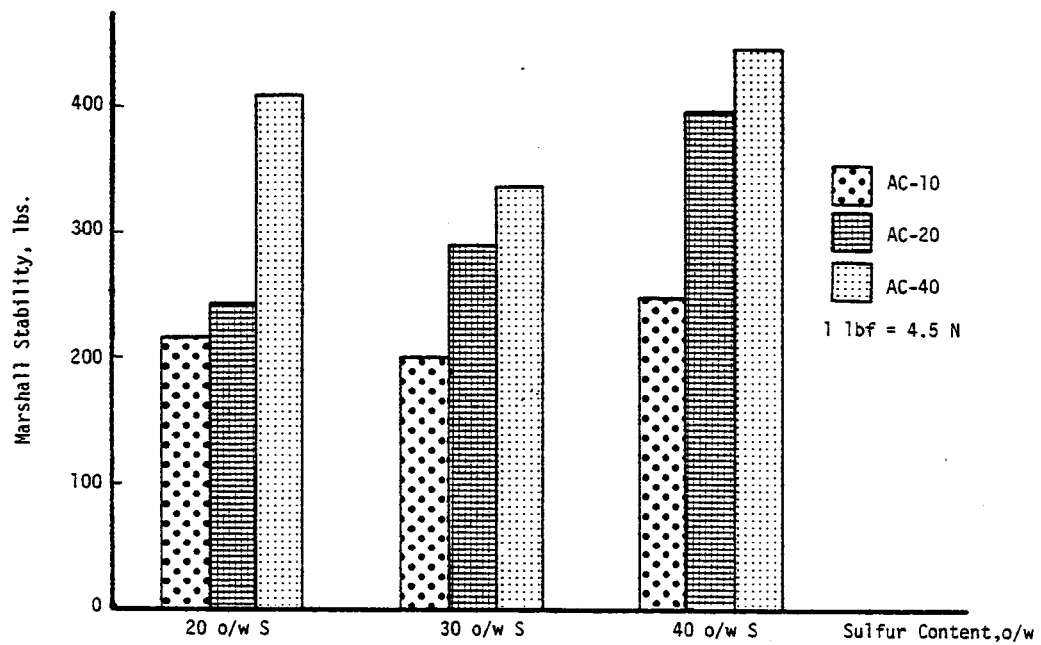


Figure 25. Marshall Stability vs AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures.

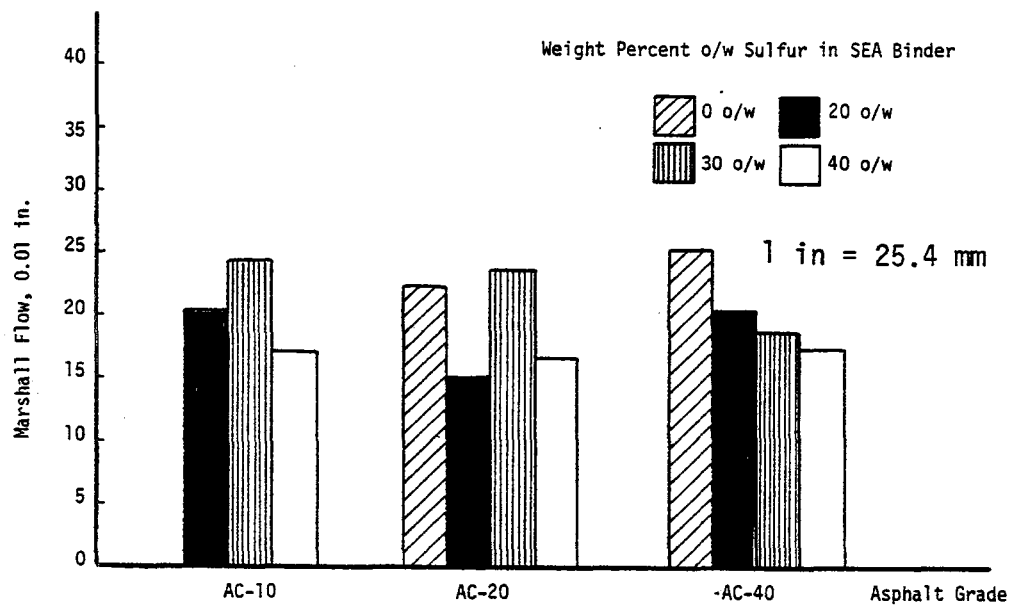


Figure 26. Marshall Flow vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

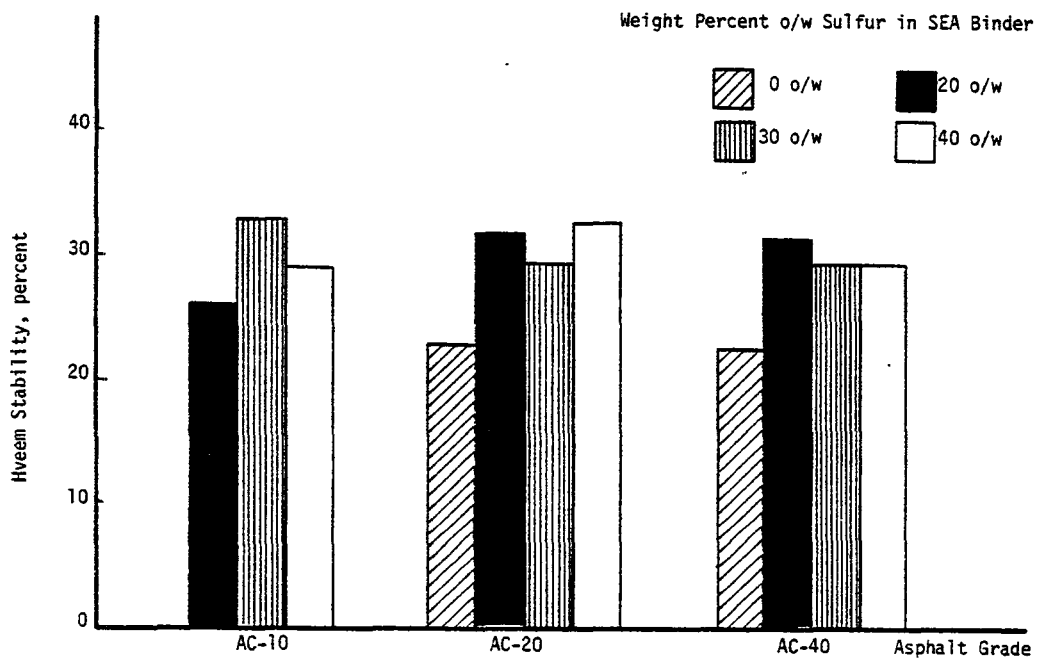


Figure 27. Hveem Stability vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.



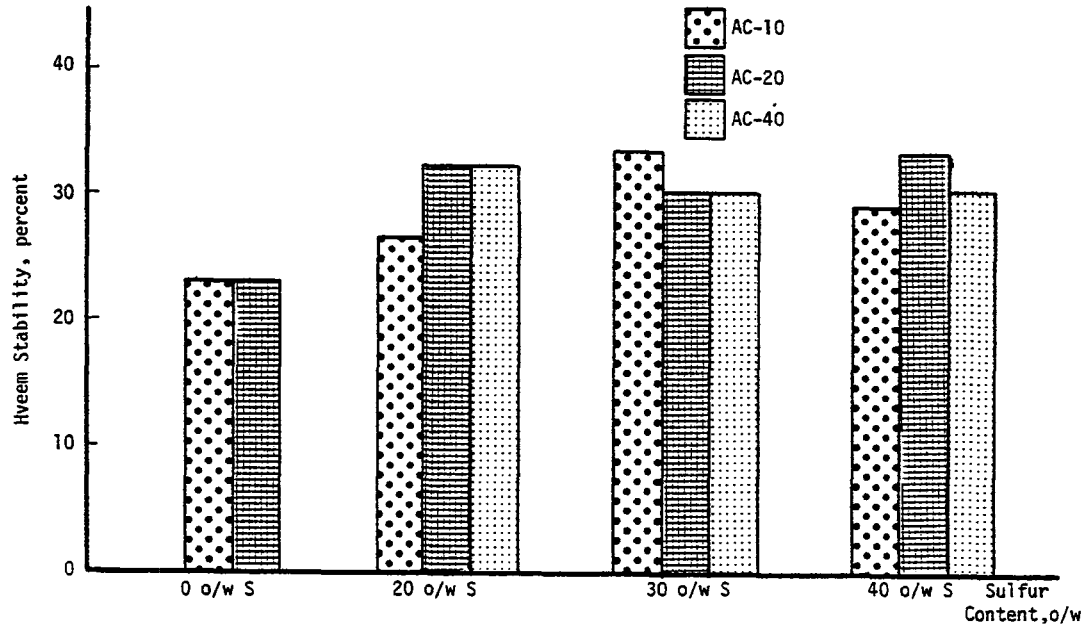


Figure 28. Hveem Stability vs AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures.

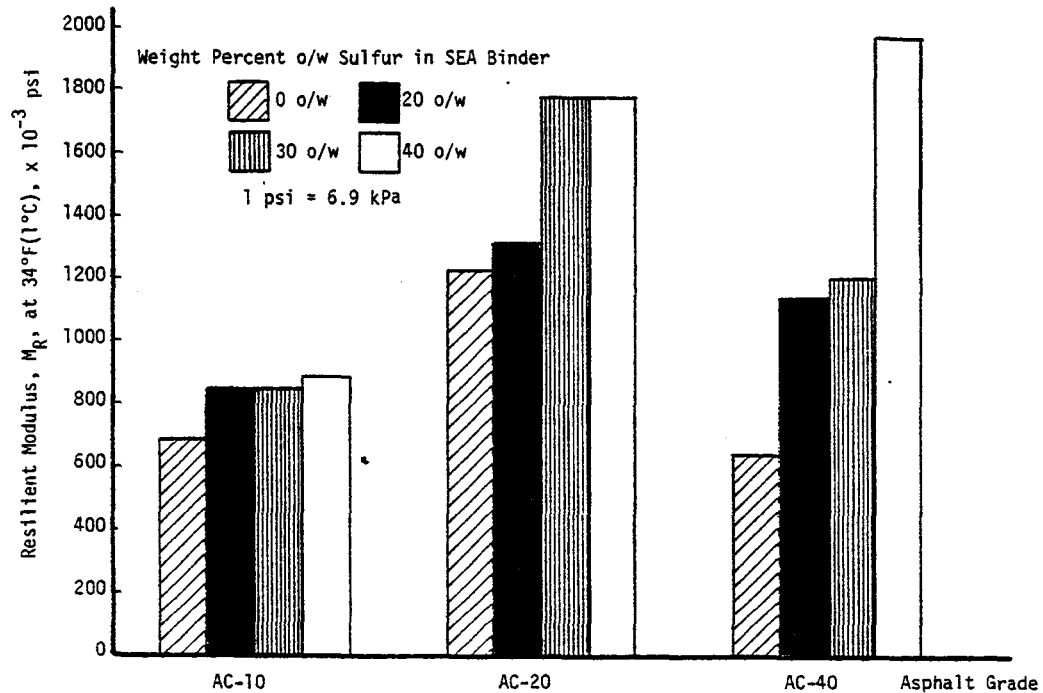


Figure 29. Resilient Modulus at 34°F (1°C) vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

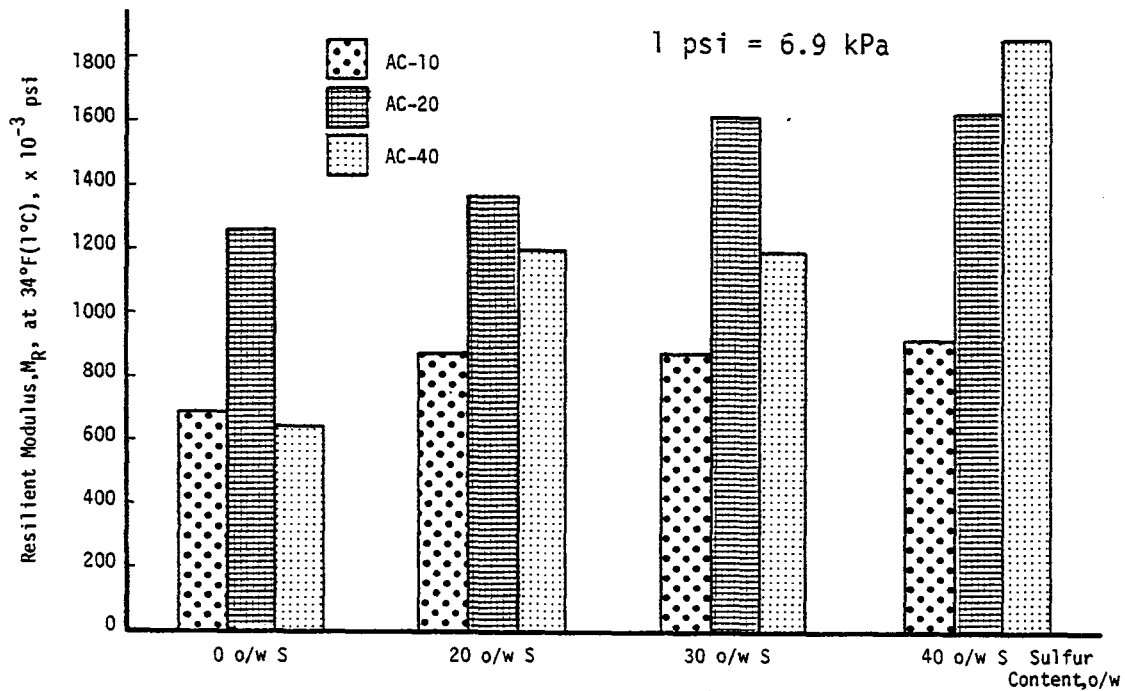


Figure 30. Resilient Modulus at 34°F(1°C) vs AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures.

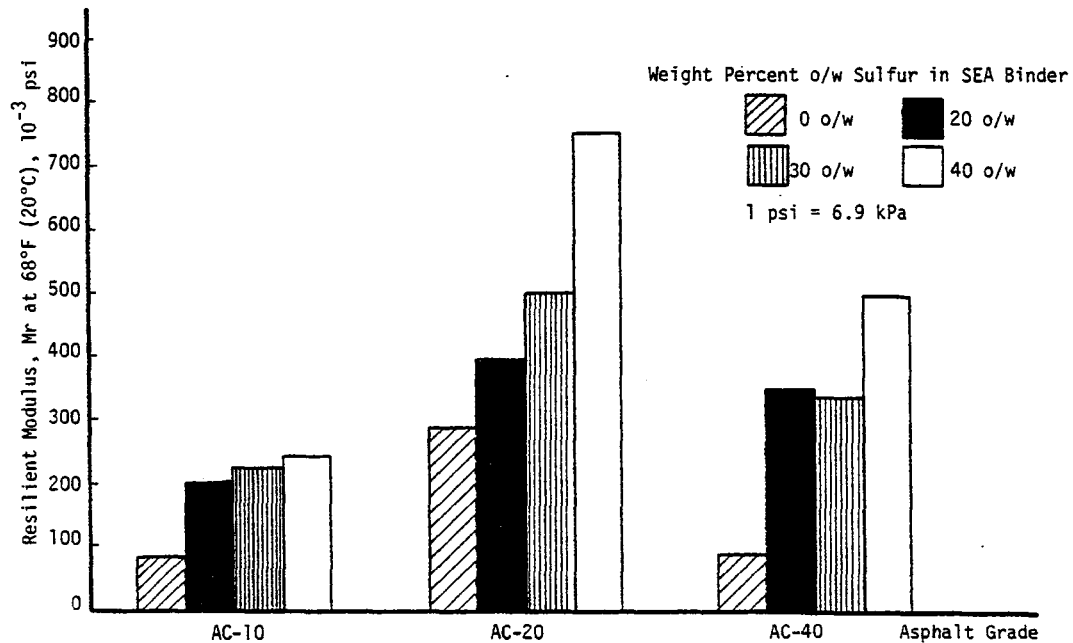


Figure 31. Resilient Modulus at 68°F(20°C) vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

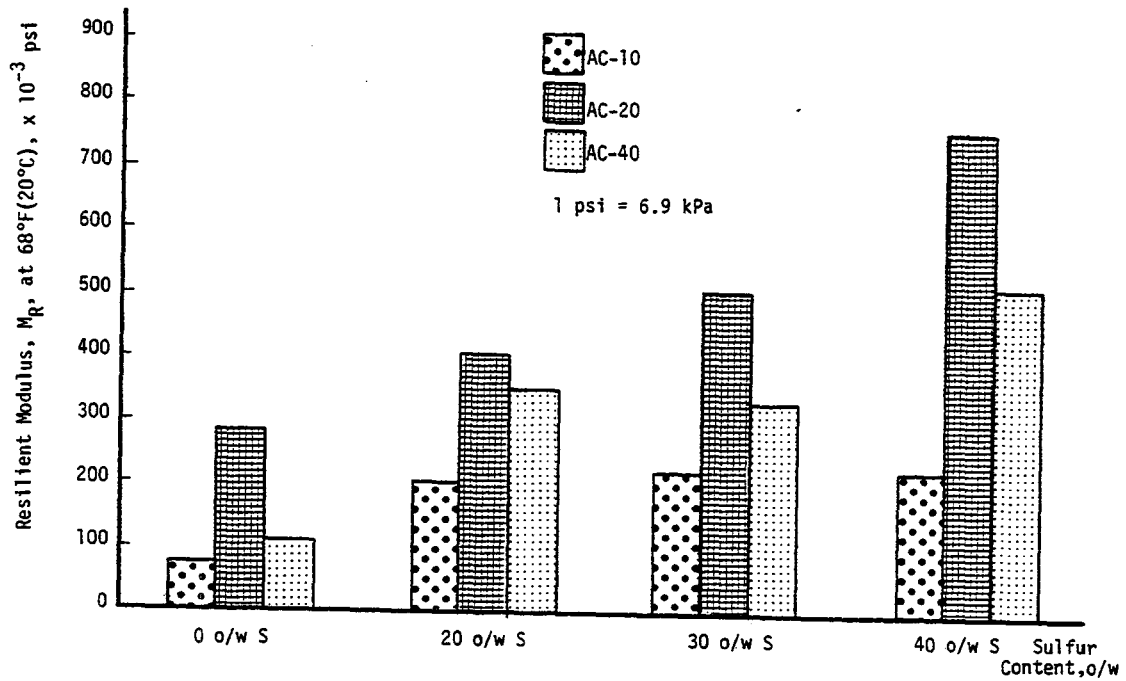


Figure 32. Resilient Modulus at 68°F(20°C) vs AC Grade with Corresponding Sulfur Content for SEA-OGFC Mixtures.

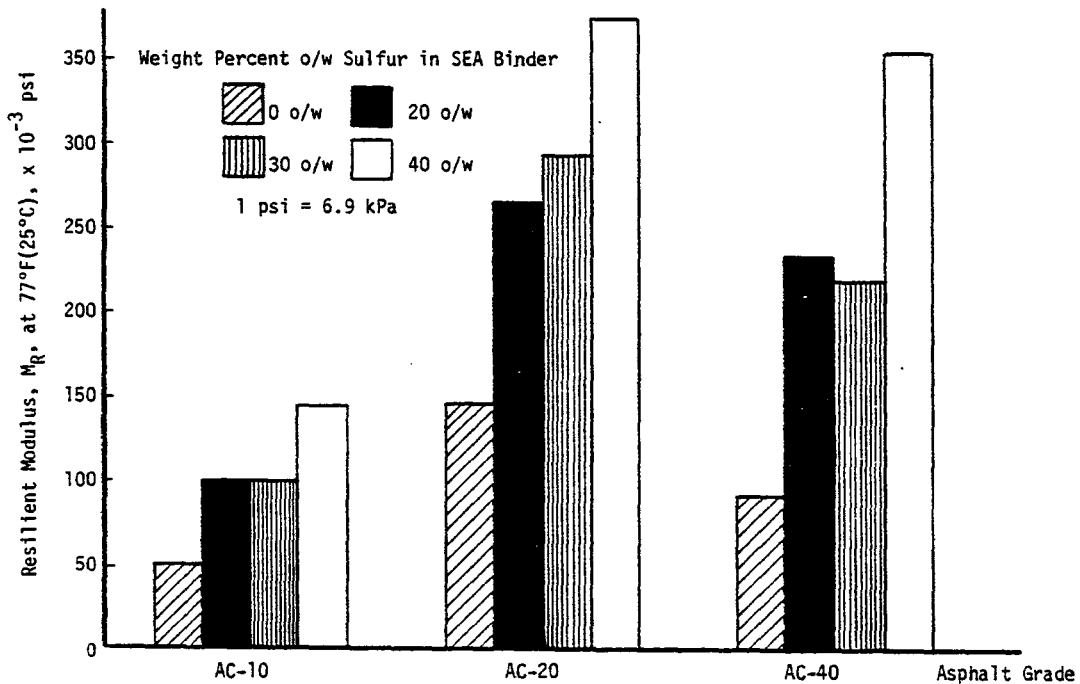


Figure 33. Resilient Modulus at 77°F(25°C) vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

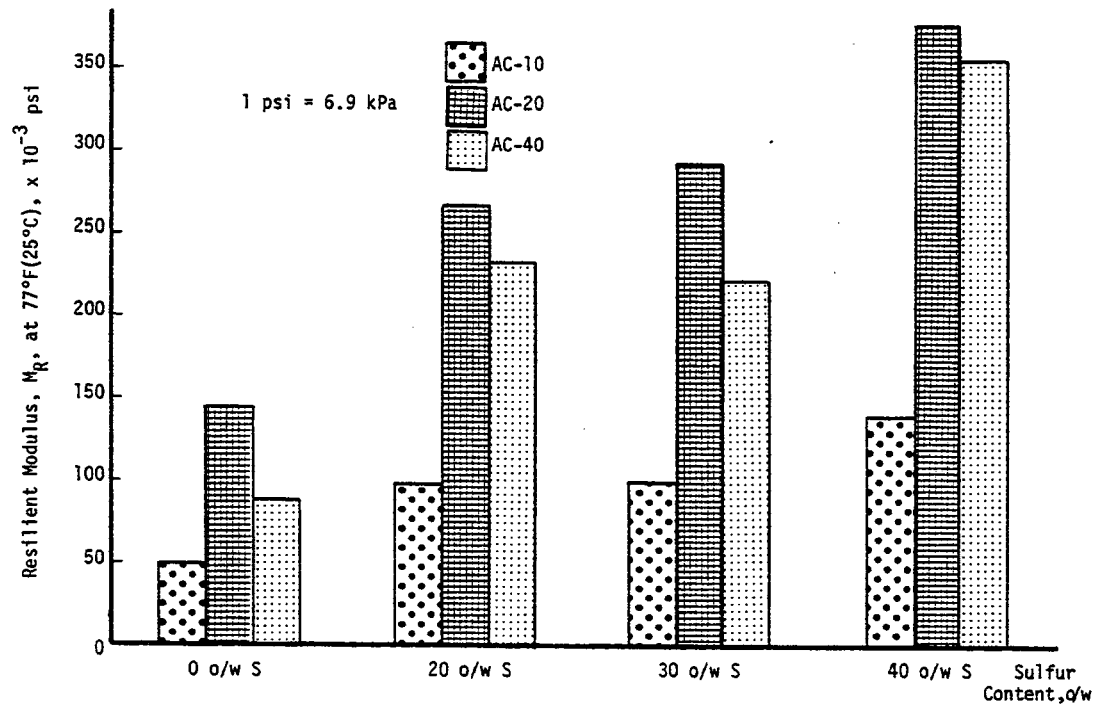


Figure 34. Resilient Modulus at 77°F (25°C) vs AC Grade with Corresponding Sulfur Content for Limestone SEA-OGFC Mixtures.

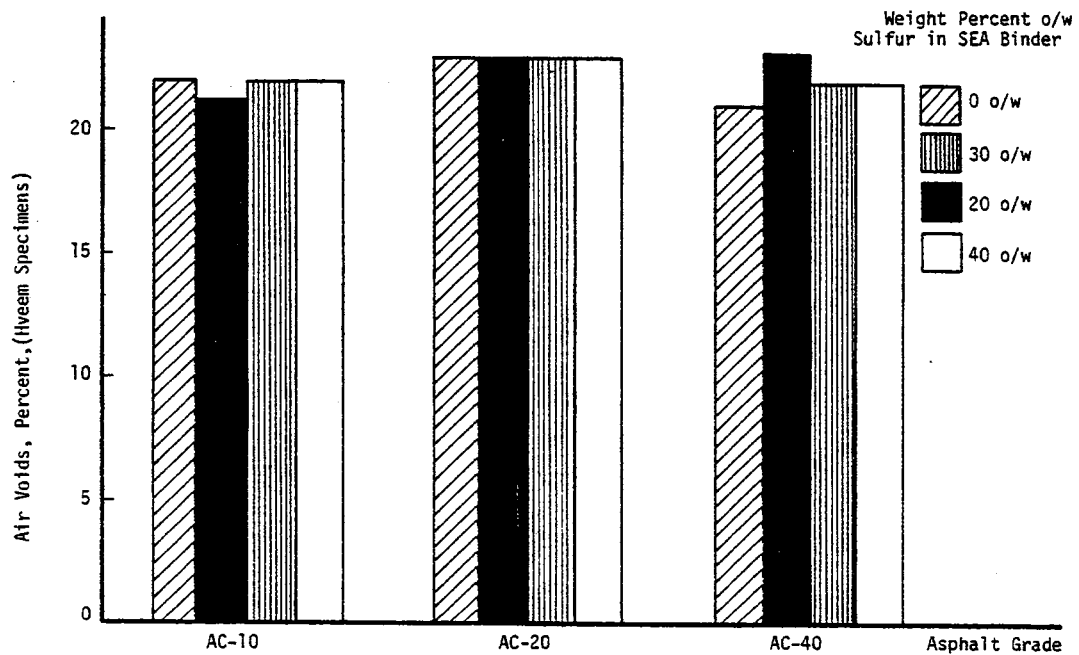


Figure 35. Air Voids vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

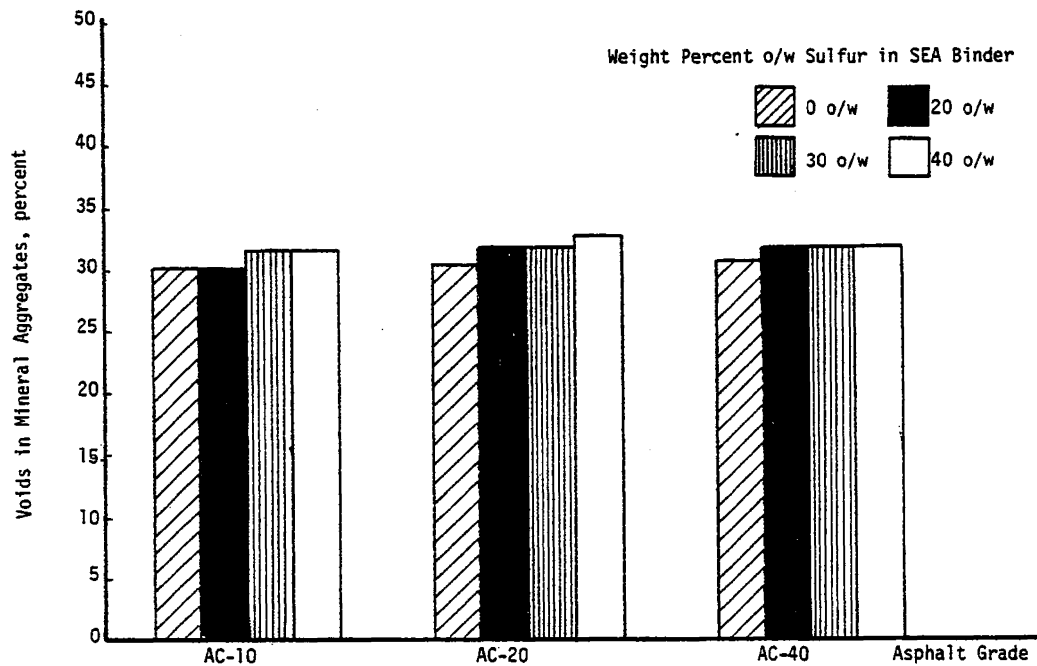


Figure 36. Voids in Mineral Aggregates vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

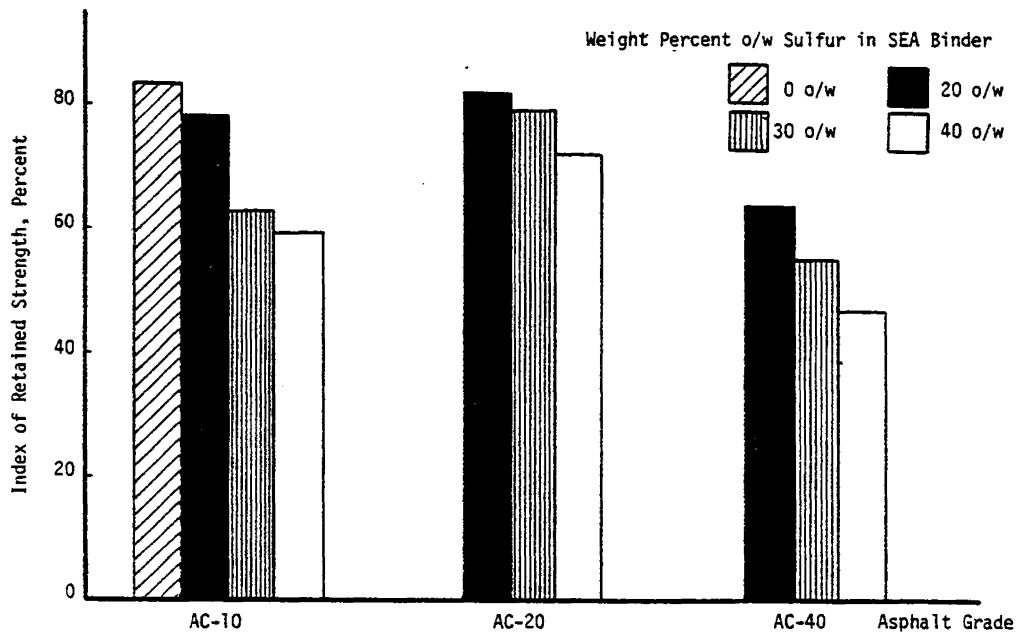


Figure 37. Index of Retained Strength vs Sulfur Content with Corresponding Asphalt Grade for Limestone SEA-OGFC Mixtures.

general tendency for Marshall stability to increase with an increase in sulfur content. However, mixtures prepared with AC 40 and 20 percent sulfur do not appear to follow this trend. No explanation for this phenomenon can be given at this point.

Figure 25 shows a similar trend where increasing stabilities increase directly with increasing asphalt viscosities. It is also observed that none of these SEA-OGFC mixtures exhibited Marshall stabilities which meet the requirement suggested by The Asphalt Institute (43). However, the highest stability value occurs in mixture prepared using AC 40 with a 40 percent sulfur SEA binder, as was the case of East Texas Sandstone mixtures.

Figure 26 indicates that SEA-OGFC mixtures have lower Marshall flow values than pure asphalt OGFC mixtures. This is consistent with the findings for the East Texas Sandstone mixtures. It also shows that SEA-OGFC mixtures prepared with AC 40 have flow values in the range (8-20) which is in accordance with that recommended by The Asphalt Institute (43).

Figures 27 and 28 show that SEA-OGFC mixtures prepared with either AC 20 or AC 40 exhibit approximately the same Hveem stabilities. These results were all in the range of 30 to 32 percent which meet the minimum requirement specified by The Asphalt Institute (46). For mixtures prepared with AC 10, only those with 30 percent sulfur substitution binder possess Hveem stabilities above 30 percent. Nevertheless, the data indicate that SEA-OGFC mixtures possess much higher stabilities than pure asphalt OGFC mixtures (range of 26-33 percent compared to 23 percent for asphalt mixtures). This observation is also in agreement with that found for East Texas Sandstone mixtures. The final observation made from Figures 27 and 28 is that SEA-OGFC mixtures with 30 or 40 percent sulfur in the binder will yield stability values which are compatible with the minimum requirements suggested by The Asphalt Institute (46).

Resilient modulus at 34°F (1°C) versus sulfur content in SEA binder and asphalt viscosity are respectively shown in Figures 29 and 30. The data indicate that the effect of sulfur on the stiffness of the mixture is more significant when a higher viscosity AC is used. For mixtures prepared with AC 10, the resilient modulus increased only about 30 percent as a result of the introduction of sulfur. For mixtures prepared with AC 20, sulfur contents of 40 weight percent of the binder increased the resilient modulus by 45 percent. However, for mixtures prepared with AC 40, the increase in resilient modulus, with a 40 weight percent sulfur substitution, is significantly higher by a factor of 3. In addition, both pure asphalt OGFC and SEA-OGFC mixtures prepared with AC 20 exhibit resilient moduli equal or superior

to those obtained from AC 10 or AC 40 with corresponding sulfur content in binder.

Figure 31 and 32 present resilient modulus at 68°F (20°C) versus sulfur content and asphalt viscosity, respectively. Figure 31 clearly indicates that sulfur has a definite effect in increasing the resilient modulus of the asphalt OGFC mixtures. However, the degree of influence varies with different asphalt grades (200 percent in AC 20 mixtures and 300 percent in AC 10 or AC 40 mixtures). Once again both pure asphalt and SEA-OGFC mixtures prepared with AC 20 show superior performance in their resilient moduli when compared to mixtures prepared with AC 10 or AC 40. However, all SEA-OGFC mixtures prepared with AC 20 and AC 40 exhibit comparable resilient modulus at 68°F (20°C) to those obtained in other types of sulfur asphalt mixtures (25-27, 45, 47).

Figures 33 and 34, respectively, show the relationship between resilient modulus at 77°F (25°C) and sulfur content and asphalt viscosity. The figures indicate that 200 percent (as in the case of AC 10 and AC 20) to 300 percent (as in case of AC 40) increases in resilient modulus can be accomplished by the introduction of 40 percent sulfur substitution in the binder. Figure 34 shows that mixtures prepared with AC 20 again display higher resilient modulus values compared to those mixtures prepared with AC 10 and AC 40.

The observations made on mixture resilient moduli at various temperatures can be summarized as follows:

1. The effect of sulfur in providing stiffness to asphalt mixtures is more significantly exemplified in mixtures prepared with AC 10 or AC 40 than in those with AC 20.
2. The loss of resilient modulus due to rise in temperature is less in SEA-OGFC mixtures than in pure asphalt OGFC mixtures.
3. SEA-OGFC mixtures prepared with AC 20 consistently show superior resilient moduli to those prepared with AC 10 or AC 40 with corresponding sulfur content in binder.
4. SEA-OGFC mixtures prepared with either AC 20 or AC 40 have comparable resilient modulus at 68°F (20°C) to other types of sulfur asphalt mixtures.
5. The introduction of sulfur into the binder tends to restrain the drop of resilient modulus as temperature of the mix rises.

Figure 35 indicates that neither the sulfur content nor the asphalt viscosity have any effect on the amount of air voids available in the SEA-OGFC mixtures. The air voids content is about 22 percent of the mixture.

The VMA available in the mixtures does not appear unduly affected by the introduction of sulfur into the asphalt binder, as indicated in Figure 36. This is consistent with that of the East Texas Sandstone mixtures discussed in the last section.

The index of retained strength (IRS) versus sulfur content in binder is shown in Figure 37. It can be seen that IRS decreases as sulfur content in the binder increases. The figure also indicates that all SEA-OGFC mixtures prepared with either AC 10 or AC 20 have IRS values equal to or above the minimum specified value of 60 percent used in this study. However, for AC 40 SEA mixtures, IRS values fall below the specified minimum requirement when sulfur content in the binder exceeds 20 weight percent.

After obtaining the results of the above characterization tests, a series of preliminary screening tests was conducted to eliminate any mixes which did not exhibit adequate mechanical behavioral characteristics for use as a surface course in highway pavements.

Preliminary screening tests. The evaluation criteria for these screening tests are once again listed below:

Marshall stability lbs.	300, min.
Marshall Flow, .01 in.	8-20
Hveem stability, percent	30, min.
Resilient modulus @ 68°F (20°C), psi	300,000, min.
Index of Retained Strength, percent	60, min.
<hr/>	
1 lb. = 4.5 N	1 in = 25.4 mm
	1 psi = 6.9 kPa

Those mixtures with properties which met the above requirements are listed in Table 9.



Table 9. Preliminary Screening Test Results for Limestone SEA-OGFC Mixtures.

Asphalt Grade, o/w S in Binder	AC 20, 40 o/w S	AC 40, 20 o/w S
Marshall stability, lb.	360	410
Marshall Flow, .01, in.	16	19
Hveem stability, percent	33	32
Resilient Modulus @ 68°F (20°C), psi	761,000	346,000
Index of Retained Strength, percent	73	64

1 lb. = 4.5 N  
1 in. = 25.4 mm  
1 psi = 6.9 kPa

Only two of the mix designs exhibited engineering properties compatible with the minimum requirement specified in this study. These two SEA-OGFC mixtures were further subjected to permeability and freeze-thaw tests to optimize the mix design. In addition, other mix types have also been tested and serve for the purpose of comparison.

Optimization tests. The permeability test is used to measure the internal drainage capacity of the pavement mixes. The test results, (i.e., coefficients of permeability,  $K_v$ ), are converted to the predicted flooding rainfall intensity factor ( $I_f$ ) which in turn, serves as a indication of the maximum rainfall intensity the pavement is able to handle before flooding begins. The data obtained from these tests are shown in Table 10. The assumptions that have been made in arriving at the predicted flooding rainfall intensity,  $I_f$ , for the analysis of East Texas Sandstone mixtures are as follows: (a) the width of pavement lane = 12 ft. (366 cm), (b) the thickness of the SEA-OGFC layer = 1 in. (2.54 cm), and (c) the percent cross-slope of the pavement = 2 percent.

Britton (48) reported that OGFC pavements having a predicted flooding rainfall intensity factor of 0.264 ( $K_v = .200$ ), or more, were found to have performed well in the reduction of splash and spray for normal wet weather driving. Therefore, the data obtained here indicate that most of the SEA mixtures have good and adequate drainage

Table 10. Permeability Test Results for Limestone Mixes.

Asphalt Grade	o/w S in Binder	Coefficient, $K_v$ (cm/sec)	Predicted Flooding Rainfall Intensity $I_F$ , (in/hr)
AC 10	0	.071	.094
	20	.200	.264
	30	.259	.342
	40	.287	.378
AC 20	0	.056	.074
	20	.238	.314
	30	.210	.277
	40	.187	.247
AC 40	0	.107	.141
	20	.262	.345
	30	.230	.303
	40	.248	.327

1 in/hr = 0.0007 cm/sec

capacities for use as pavement surface courses. However, one of the mix designs in question (AC 20 SEA-OGFC mixture with 40 weight percent sulfur) exhibited a comparatively low coefficient of permeability (0.187 cm/sec) which makes the merit of having such a surface course questionable. Nevertheless, the other mix design in question (AC 40 SEA-OGFC mixtures with 20 percent sulfur in binder) shows a relatively high coefficient of permeability of ( $K_v = 0.262$  cm/sec). In addition, it can also be seen from the data that the coefficients of permeability are consistently higher in OGFC mixtures prepared with SEA binders than those prepared with pure asphalt binders. An explanation of this phenomenon is recommended for further study.

After the permeability tests were completed, the same specimens were subjected to freeze-thaw testing using the test procedure described earlier. The test results are tabulated in Table 11.

The data collected in this test indicate that OGFC mixtures prepared with pure asphalt binders (AC 10, AC 20 or AC 40) begin to show signs of aggregate fracture after 75 cycles and aggregate loss after 100 cycles. Apparently SEA-OGFC mixtures with AC 10 also seem to show sign of either aggregate fracture or aggregate loss after 100 cycles. For SEA-OGFC mixes with AC 20 and 30 weight percent sulfur content, the mixtures consistently show minor aggregate fracture after 100 cycles. However, for SEA-OGFC with AC 20 and 20/80 or 40/60, no sign of distress can be found after 100 cycles. For SEA-OGFC mixtures prepared with AC 40, the data indicate no sign of failure after 100 cycles. In addition, none of the mixtures showed much aggregate loss in the brush test which was conducted after the freeze-thaw testing.

Optimum mix design. The optimum mix design was selected based on the mixtures structural behavior, surface drainage capacity and freeze-thaw durability. The maximization of sulfur content used to replace the asphalt demand in the SEA binder was also considered.

There were only two mix designs that could successfully pass the preliminary screening tests. One of them was the mixture prepared with 40 percent sulfur and 60 percent AC 20. The other was the mixture prepared with AC 40 and 20 percent sulfur in the binder. Among these two mix designs, the former seems to show slightly more favorable structural behavior in the mixture. However, the permeability of the former ( $K_v = 0.262$  cm/sec) is more than adequate to handle more rainfall conditions. Both of the above mixtures had good resistance to repeated cycles of freezing and thawing.

Since the primary function of any OGFC pavements is to provide

Table 11. Freeze-Thaw Test Results for Limestone Mixes.

1 gm = 0.0022 lb.

Asphalt Grade	o/w S	<u>Visual Observation at</u>						Brush Test Weight Loss (gm)
			0 Cycles	25 Cycles	50 Cycles	75 Cycles	100 Cycles	
AS -10	0	(1)	0	0	0	2-1	2-1, 3-1	0.4
		(2)	0	0	2-1	2-1, 3-1	2-1, 3-1	0.1
		(3)	0	0	0	2-1	2-2	0.2
AC -10	20	(1)	0	0	0	2-1	3-1	0.2
		(2)	0	0	0	0	0	0.1
		(3)	0	0	0	0	2-1	0.3
AC -10	30	(1)	0	0	0	2-1	3-1	0.2
		(2)	0	0	0	0	0	0.2
		(3)	0	0	0	0	0	0.1
AC -10	40	(1)	0	0	0	2-1	2-1	0.3
		(2)	2-1	2-1	2-1	2-1	2-1, 3-1	0.9
		(3)	2-1	2-1	2-1	2-1	2-1	0.4
AC -20	0	(1)	0	0	0	2-1	2-1, 3-1	0.2
		(2)	0	0	0	2-1	2-1, 3-1	0.4
		(3)	0	0	0	2-1	2-2, 3-1	0.3

(Continued)

Table 11. Continued.

1 gm = 0.0022 lb.

Asphalt Grade	O/w S	<u>Visual Observation at</u>						Brush Test Weight Loss (gm)
			0 Cycles	25 Cycles	50 Cycles	75 Cycles	100 Cycles	
AC -20	20	(1)	0	0	0	0	2-1	0
		(2)	0	0	0	0	0	0
		(3)	0	0	0	0	0	0
AC -20	30	(1)	0	0	0	0	2-1	0
		(2)	0	0	0	0	2-1	0
		(3)	0	0	0	0	2-1	0.1
AC -20	40	(1)	0	0	0	0	0	0.2
		(2)	0	0	0	0	0	0.2
		(3)	0	0	0	0	0	0.2
AC -40	0	(1)	0	0	0	2-1, 3-1	2-2, 3-1	0.2
		(2)	0	0	0	2-1	2-1, 3-1	0.4
		(3)	0	0	0	2-1	2-1, 3-1	0.4
AC -40	20	(1)	0	0	0	0	0	0
		(2)	0	0	0	0	2-1	0.3
		(3)	0	0	0	0	0	0.3

(Continued)

Table 11. Continued.

Asphalt Grade	o/w S	<u>Visual Observation at</u>					Brush Test Weight Loss (gm)	
		0 Cycles	25 Cycles	50 Cycles	75 Cycles	100 Cycles		
AC -40	30	(1)	0	0	0	0	0	0.4
		(2)	0	0	0	0	0	0.1
		(3)	0	0	0	0	0	0.1
AC -40		(1)	0	0	0	0	0	0.4
		(2)	0	0	0	0	0	0.2
		(3)	0	0	0	0	0	0.1

1 gm = 0.0022 lb

drainage channels for surface water, a high coefficient of permeability is desired. However, the maximization of sulfur content in the SEA binder was also emphasized in this study. As a result, either of these two mix designs could be selected as the optimum, with initial preference given to the design with the AC 40 and 20/80 SEA binder.

#### Analysis for Streetman SEA-OGFC Mixtures

Structural characterization. The result of the structural characterization of the mixes prepared with this aggregate system are shown in Figures 38 through 49.

Figures 38 and 39 show a trend of increasing Marshall stability with increasing asphalt viscosity and sulfur content in the binder. For all asphalt grades, data seem to indicate that mixtures prepared with 40/60 SEA binder increased the Marshall stability about 35 percent. However, all the mixtures had Marshall stabilities above the minimum value specified by The Asphalt Institute (43). Marshall flow versus asphalt grade and sulfur content is shown in Figure 40. It is noteworthy that none of the flow values fall within the range recommended by The Asphalt Institute (43).

Hveem stability versus sulfur content and Hveem stability versus asphalt viscosity are respectively shown in Figures 41 and 42. It is observed that SEA-OGFC mixtures prepared with AC 20 consistently shown Hveem stabilities, ranging from 30 to 33 percent, thus meeting minimum requirement of The Asphalt Institute (43). On the other hand, for AC 10 and AC 40, the majority of the mixes do not show adequate stabilities. These data indicate that the AC 20 provided the best stability to the SEA mixtures. It can also be seen that for all asphalt grade, mixtures with 30/70 S/A ratios consistently exhibited Hveem stabilities above 30 percent (the minimum requirement).

Figures 43 and 44 present resilient modulus test results generated at 34°F (1°C) versus sulfur content and asphalt viscosity, respectively. The observation here does not follow the findings for Limestone and East Texas Sandstone mixtures. For AC 20, resilient moduli of the mixtures increased with sulfur content in the binder. On the other hand, for both AC 10 and AC 40, the resilient modulus seems to reach a peak value at S/A ratios of 30/70. Additionally, OGFC and SEA-OGFC mixtures prepared with AC 20 consistently show higher resilient modulus values than those prepared with AC 10 or AC 40.

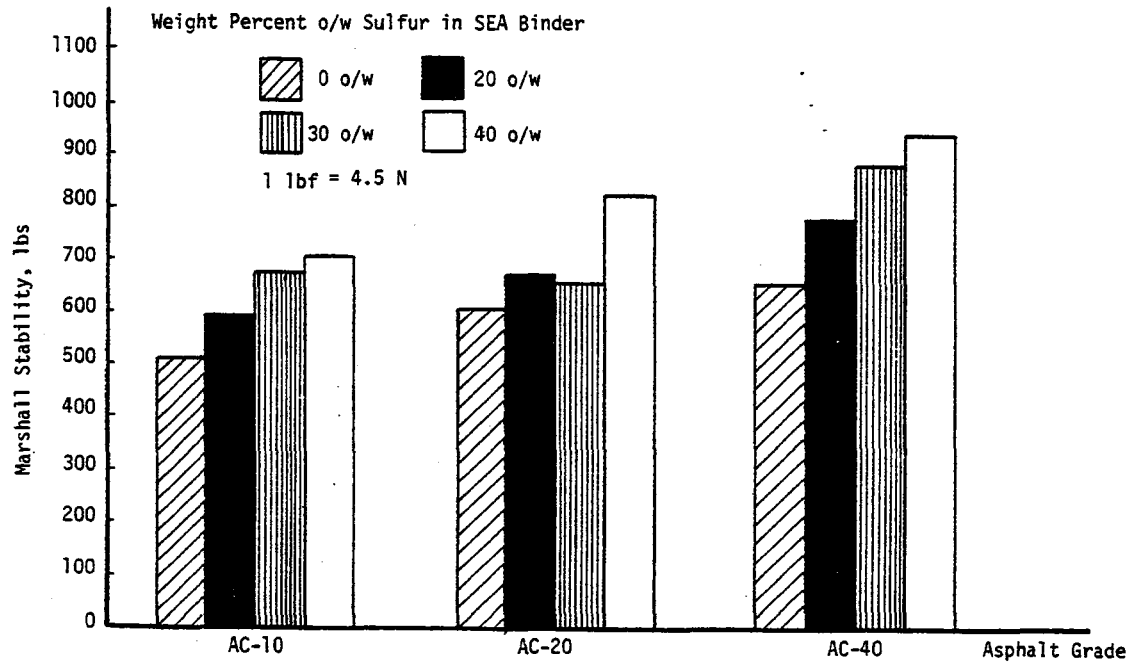


Figure 38. Marshall Stability vs Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures.

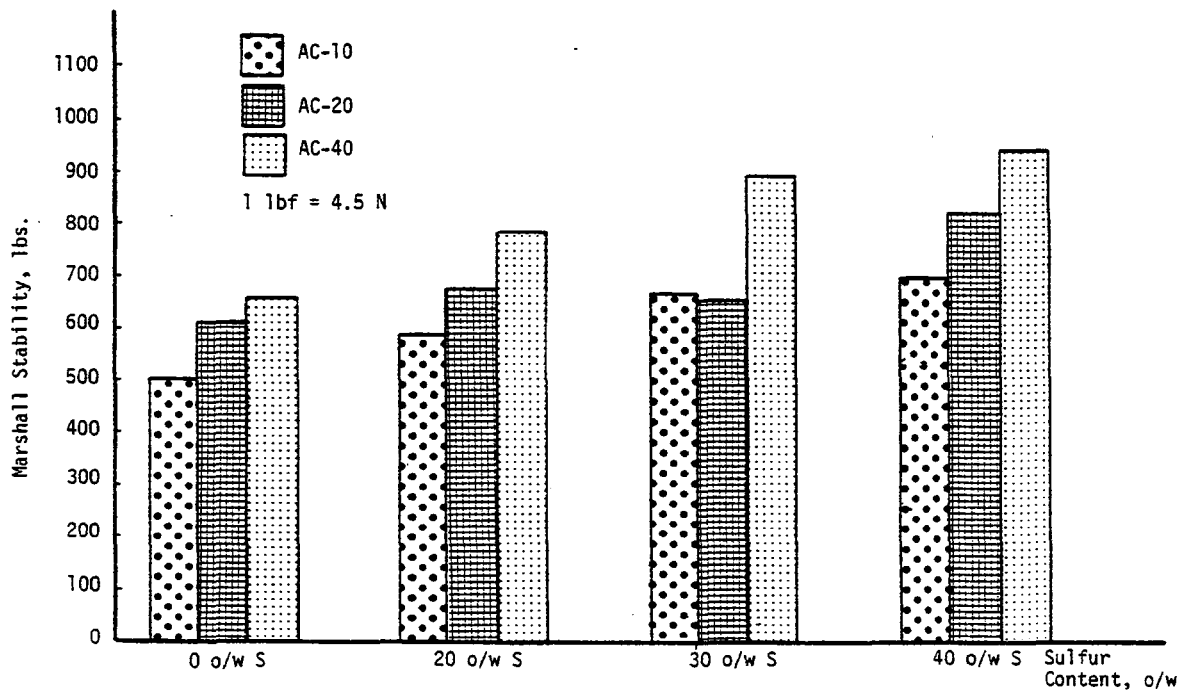


Figure 39. Marshall Stability vs AC Grade with Corresponding Sulfur for Streetman SEA-OGFC Mixtures.



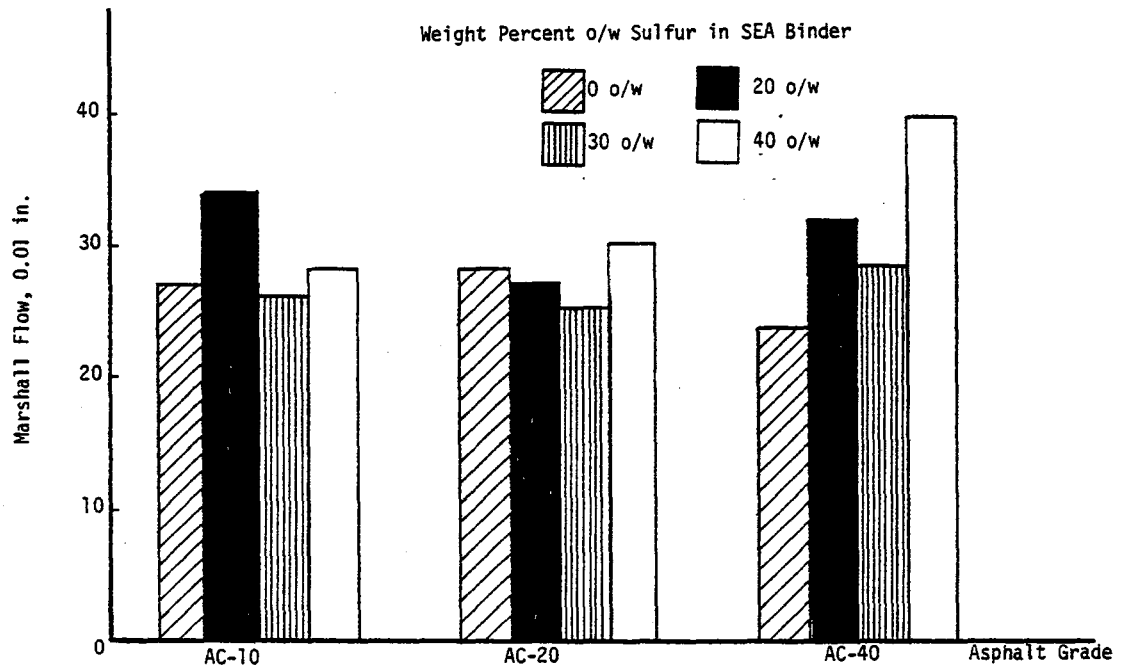


Figure 40. Marshall Flow vs Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures.

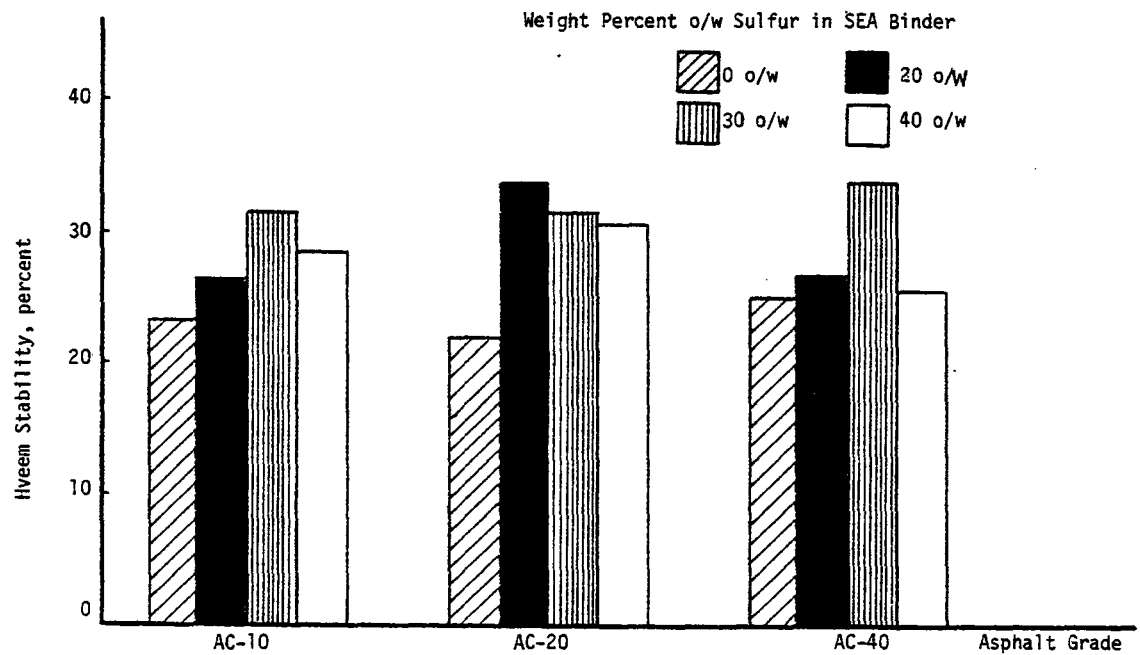


Figure 41. Hveem Stability vs Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures.

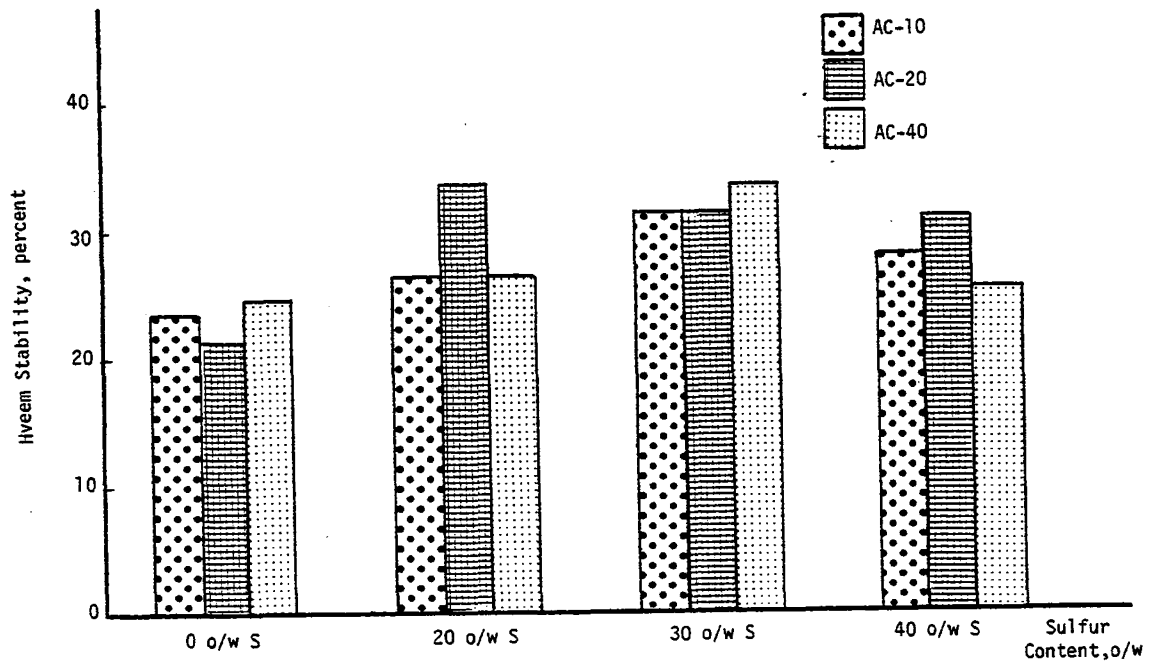


Figure 42. Hveem Stability vs AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures.

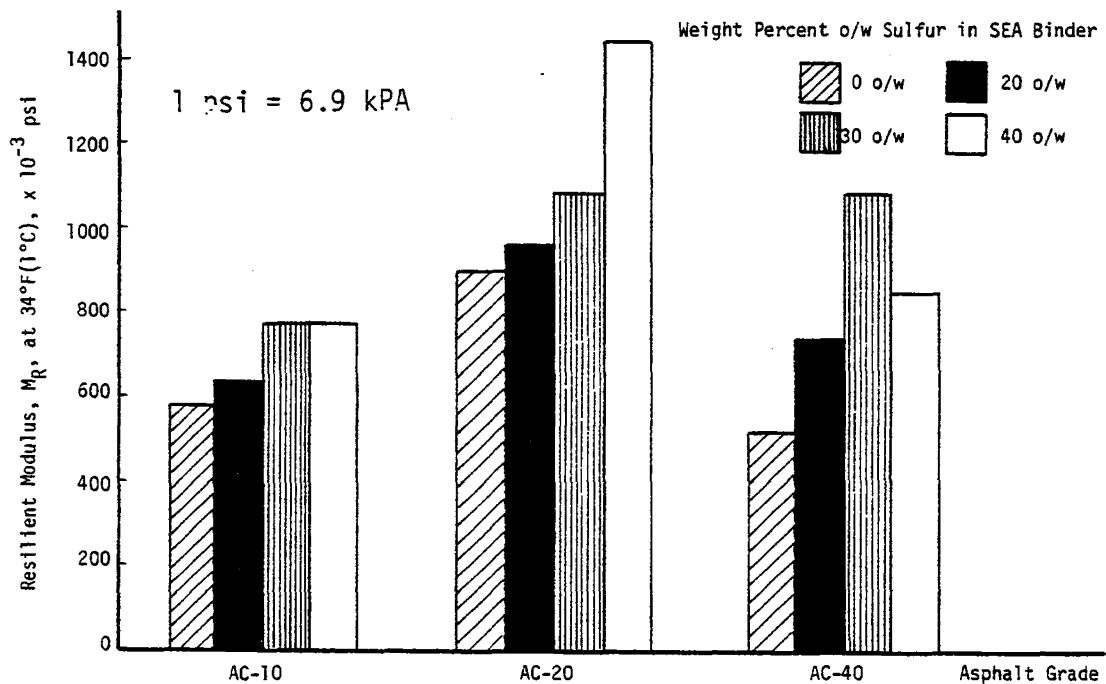


Figure 43. Resilient Modulus at 34°F (1°C) vs Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures.

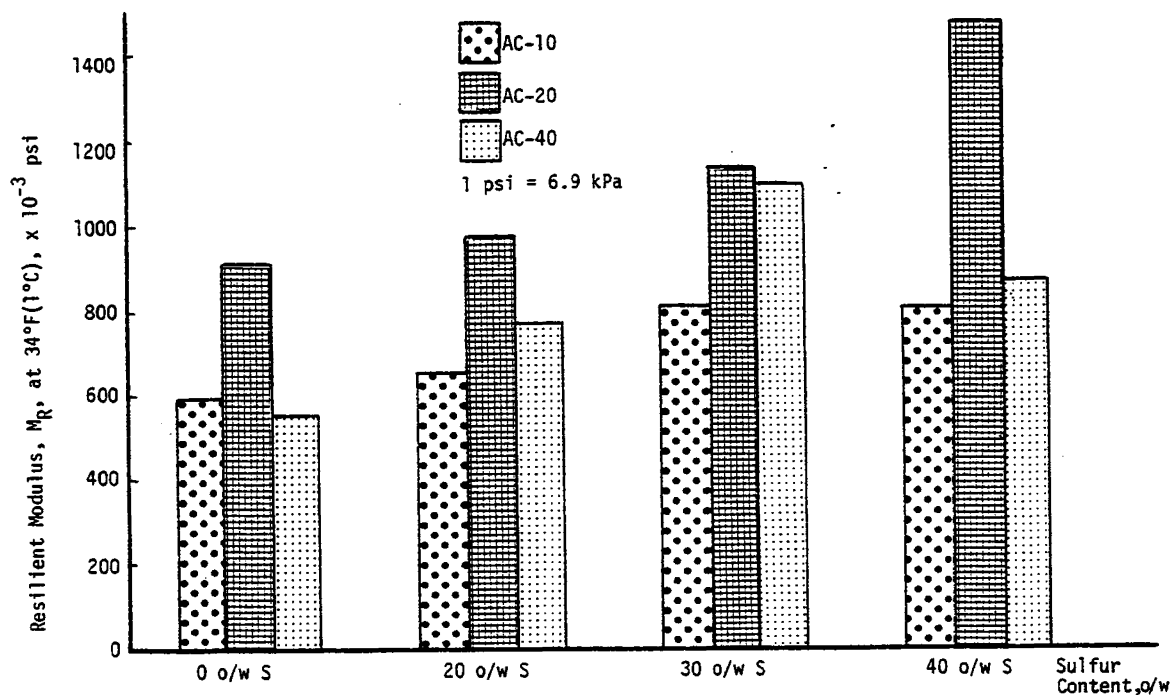


Figure 44. Resilient Modulus at 34°F (1°C) vs AC Grades with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures.

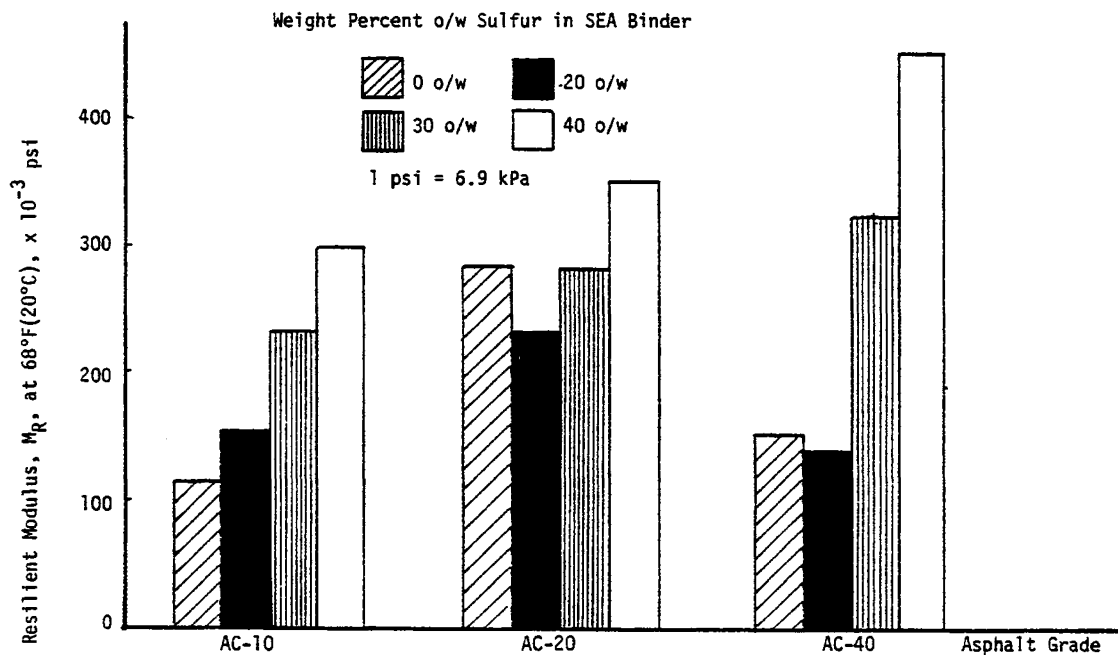


Figure 45. Resilient Modulus at 68°F (20°C) vs Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures.

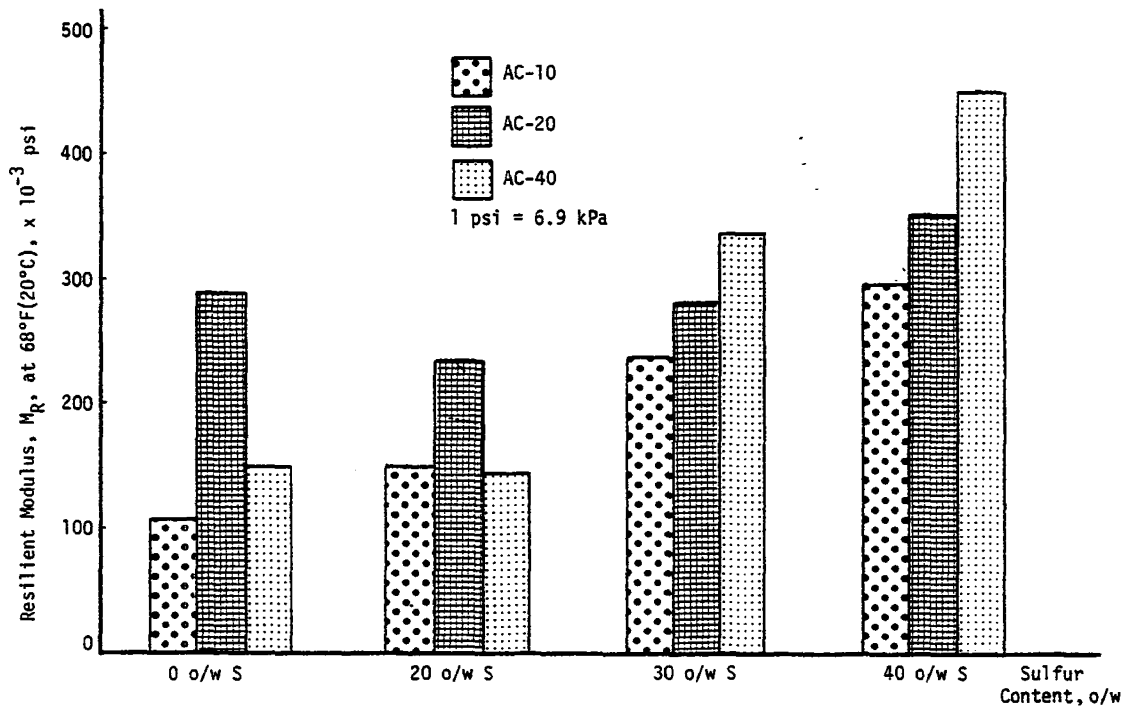


Figure 46. Resilient Modulus at 68°F (20°C) vs AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures.

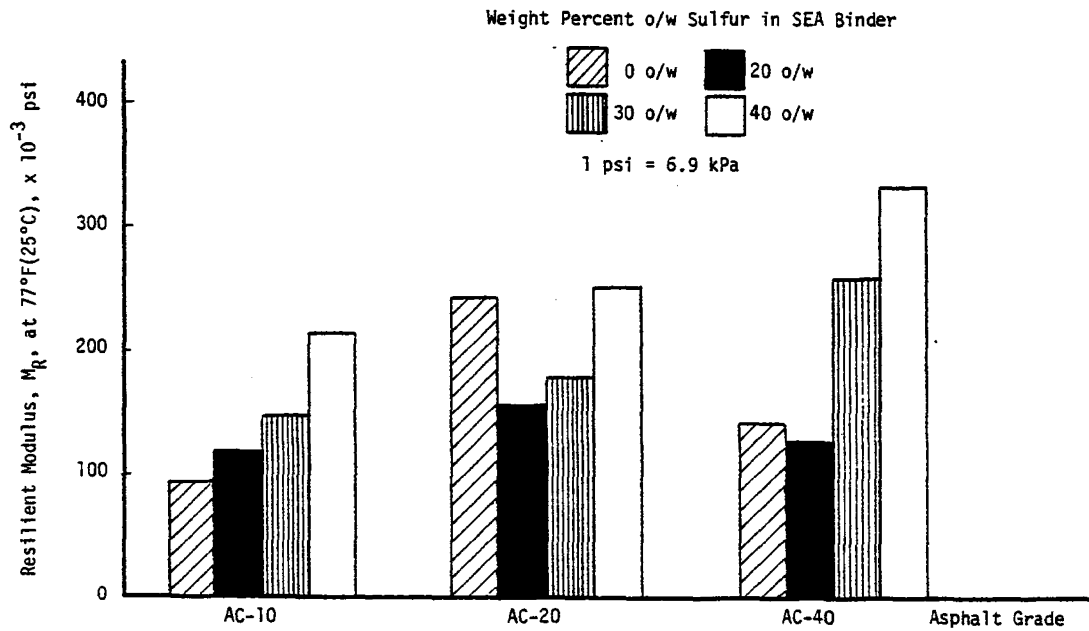


Figure 47. Resilient Modulus at 77°F (25°C) vs Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures.

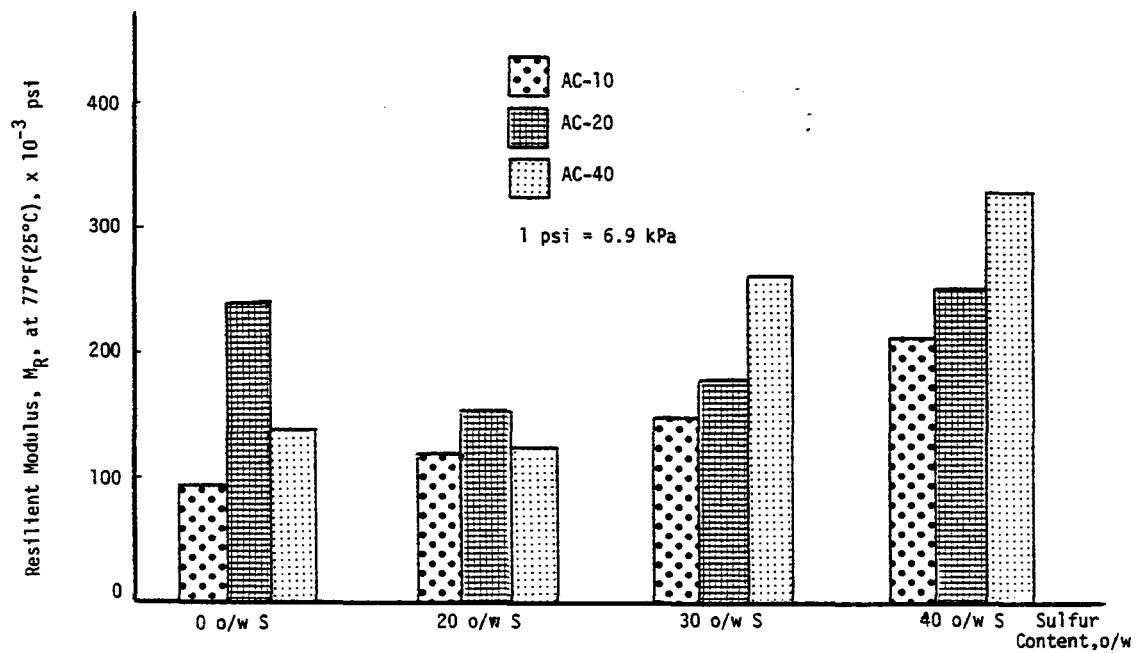


Figure 48. Resilient Modulus at 77°F(25°C) vs AC Grade with Corresponding Sulfur Content for Streetman SEA-OGFC Mixtures.

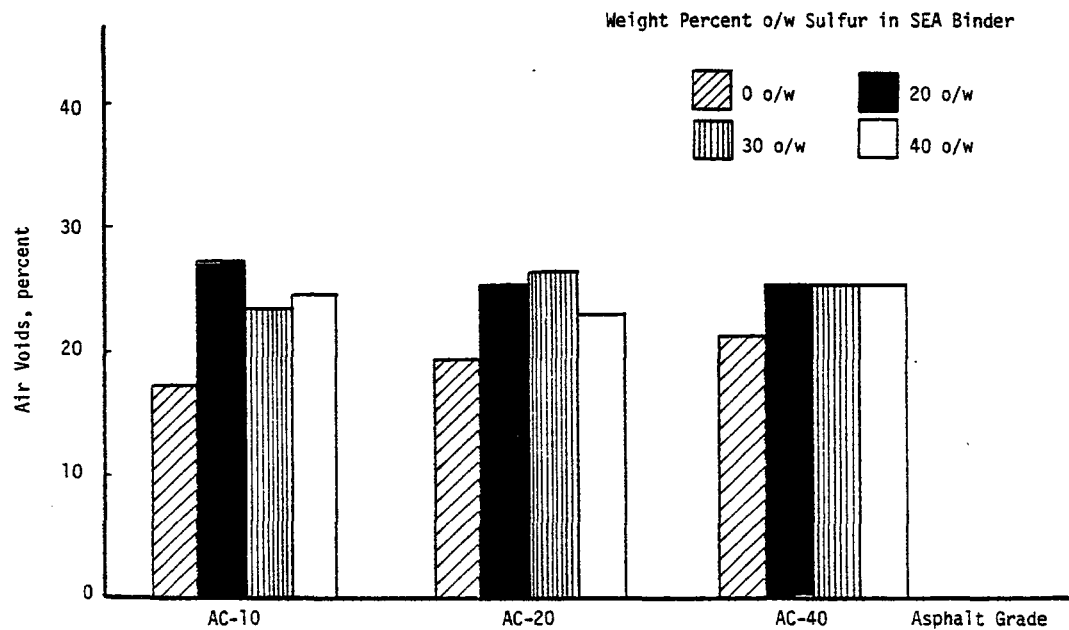


Figure 49. Air Voids vs Sulfur Content with Corresponding Asphalt Grade for Streetman SEA-OGFC Mixtures.

Figure 45 and 46 indicate that the majority of the mixes with up to 30 percent sulfur in the binder have  $M_R$  values at 68°F (20°C) below 300,000 psi ( $20.2 \times 10^3$  kPa). Both the pure asphalt and 20 percent sulfur mixtures experienced a reduction in  $M_R$  when A/C 40 was used. All others showed increases with asphalt viscosity. The reduction in  $M_R$  between 0 and 20 o/w sulfur can be attributed, in part, to the fluxing effect sulfur has on asphalt up to its solubility threshold (approximately 20 percent at normal mix temperature - Ref 26).

Resilient modulus at 77°F (25°C) versus sulfur content and asphalt viscosity are respectively shown in Figure 47 and 48. The data indicate that similar trends to those shown in Figure 45 and 46 may be drawn here. One final observation which can be made is that the introduction of sulfur into the binder does not seem to restrain the relative drop of resilient modulus as the temperature of the mix rises. This was also indicated for Limestone and East Texas Sandstone mixtures.

Figure 49 shows the effect sulfur content in the binder has on the amount of air voids produced in the mixtures. The SEA-OGFC mixtures have air voids content in the range of 23 to 27 percent compared to the range of 16 to 20 percent of OGFC mixtures. This indicates a 5 to 6 percent increase in air voids content due to the introduction of sulfur into the binder. One of the reasons offered for this increase is that the solubility threshold of sulfur in asphalt at the mix temperatures is reached at about a 20 percent sulfur content. At higher sulfur contents the undissolved sulfur recrystallizes with an inherent shrinkage in volume. Some recent data generated by McBee [49] suggested that the real solubility threshold could be closer to 5 percent at ambient temperature with the excess sulfur being transformed into faceted needles. The increase in air voids with sulfur content was also indicated in tests performed for the Lufkin, Texas SEA field trials [45]. The post-cure, long-range effects on the role sulfur plays in the mix with different aggregate systems is an area which requires more extensive study.

Upon completion of the above characterization tests, a series of preliminary screening tests was conducted to eliminate any mixes which did not exhibit adequate mechanical behavioral characteristics for use as a surface course in highway pavements.

Preliminary screening tests. The screening tests and the associated evaluation criteria are listed below:

Marshall Stability, lbs.	300, min.
Flow, .01 in.	8-20
Hveem Stability, percent	30, min.
Resilient Modulus @ 68°F (20°C), psi	300,000, max.
Index of Retained Strength (IRS), %	60, min.
1 lb. = 4.5 N	1 in. 25.4 mm
	1 psi = 6.9 kPa

Those mixtures with properties compatible with the above requirements are listed in Table 12. The Marshall flow of every mix design listed exceeded of the range recommended by The Asphalt Institute (42). The Marshall stabilities, however were within the suggested range. Only SEA-OGFC mixtures with AC 20 and 40/60 SEA binder had resilient moduli higher than the specified minimum value arbitrarily established in this study.

All of the mix designs except the 40 percent SEA system using the AC 20 asphalt exceeded the minimum allowable index of retained strength. In fact, this was the only mix design including all three aggregates that did not meet the screening criteria, although, all of the mix designs were considered in the optimization tests.

Optimization tests. Permeability tests were conducted on all the mix designs listed in Table 12 to evaluate their internal drainage capacity. The resulting coefficients of permeability were then converted to predicted flooding rainfall intensity factor,  $I_f$ , which is an indication of the maximum rainfall intensity the pavement is able to handle before flooding begins. The coefficients of permeability ( $K_v$ ) and the corresponding predicted flooding rainfall intensity factor ( $I_f$ ) are shown in Table 13. The assumptions that have been made in arriving at the predicted flooding rainfall intensity,  $I_f$ , for the analysis in East Texas Sandstone mixtures were also used here. They include (a) width of the pavement lane = 12 ft. (366 cm), (b) thickness of the SEA-OGFC layer = 1 in. (2.54 cm), and (c) percent cross-slope of the pavement = 2 percent.

Britton (48) reported that OGFC pavements having a predicted flooding rainfall intensity factor ( $I_f$ ) of 0.264 or more (equivalent to a coefficient of permeability of 0.200 or more) were found to perform well in the drainage of surface water in normal rainfall conditions. Since these mixtures have  $I_f$  in the range of 0.365 to 0.423, they are expected to provide adequate surface drainage capacity

Table 12. Preliminary Screening Test Results for Streetman SEA-OGFC Mixtures.

Asphalt Grade	o/wS in Binder	Marshall Stability, lb.	Marshall Flow, .01in	Hveem Stability, percent	Resilient Modulus @ 68°F(20°C), psi	Index of Retained Strength, %
AC 10	30	680	26 <sup>*</sup>	31	230,000 <sup>**</sup>	62
AC 20	20	680	25 <sup>*</sup>	33	230,000 <sup>**</sup>	68
AC 20	30	670	27 <sup>*</sup>	31	280,000 <sup>**</sup>	63
AC 20	40	840	30 <sup>*</sup>	30	352,000	58
AC 40	30	900	28 <sup>*</sup>	33	230,000 <sup>**</sup>	70

\* Number falls out of recommended range of values.

\*\* Number below specified minimum value.

1 lb = 4.5 N

1 in = 25.4 mm



for normal wet weather driving. However, the data seem to indicate that SEA-OGFC with AC 20 and S/A of 40/60 and SEA-OGFC with AC 10 and S/A of 30/70 have the best permeability characteristics.

Table 13. Permeability Test Results for Streetman SEA-OGFC Mixtures.

Asphalt Grade	o/w S in Binder	Permeability Coefficients, $K_v$ (cm/sec)	Predicted Flooding Rainfall Intensity $I_f$ (in/hr)
AC 10	30	.321	.423
AC 20	20	.296	.390
AC 20	30	.283	.373
AC 20	40	.320	.422
AC 40	30	.277	.365

1 in/hr = 0.0007 cm/sec

In order to further characterize the durability of these SEA-OGFC mixtures, they were subjected to freeze-thaw testing for 100 cycles. The data obtained are tabulated in Table 14.

It can be seen that only mixtures prepared with AC 10 and 30/70 SEA binder and mixtures prepared with AC 20 and 30/70 SEA binder remain intact after 100 cycles of freezing and thawing. The other three mixtures suffered both aggregate fracture and aggregate loss after 75 cycles. Therefore, only the former two SEA-OGFC mixtures can be considered for use as a surface course for highway pavements.

Optimum mix design. The freeze-thaw test data indicate that there are only two SEA-OGFC mix designs that can be considered in the selection of the optimum mix design. However, they both exhibit similar structural properties and comparable values of coefficients of permeability. Therefore, both of these SEA-OGFC mix designs were selected to be the optimum mix designs for this particular aggregate system. (The reader is cautioned that both of these mix designs exhibit flow values outside the range recommended by The Asphalt Institute (43)).

Table 14. Freeze-Thaw Test Results for Streetman SEA-OGFC Mixtures.

Asphalt Grade	o/w S		0 Cycle	25 Cycles	50 Cycles	75 Cycles	100 Cycles	Brush Test Weight Loss (gm)
AC 10	30	(1)	0	0	0	0	0	1.3
		(2)	0	0	0	0	0	1.0
		(3)	0	0	0	0	0	1.2
AC 20	20	(1)	0	0	0	3-1	3-2	1.3
		(2)	0	0	0	2-1	3-1	1.2
		(3)	0	0	0	3-1	3-2	2.0
AC 20	30	(1)	0	0	0	0	0	0.7
		(2)	0	0	0	0	0	0.5
		(3)	0	0	0	0	0	0.4
AC 20	40	(1)	0	0	0	0	2-1	0.8
		(2)	0	0	0	0	0	2.4
		(3)	0	0	0	3-1	3-1	2.0
AC 40	30	(1)	0	0	0	0	0	0.7
		(2)	0	0	0	2-1	2-1, 3-1	0.5
		(3)	0	0	0	0	3-1	1.2

1 lb = 454 gm

### Selection of Optimum SEA-OGFC Mix Design

After the selection of an optimum mix design for each aggregate system, a comparison of the engineering properties of these mix designs was conducted to select the best mix design out of all the aggregate systems considered in this study. The comparison was based on the same evaluation tests used in the selection of the optimum mix design for each aggregate system and they include:

1. Marshall Stability and Flow (ASTM D1559)
2. Hveem Stability (ASTM D1560)
3. Resilient Modulus (12)
4. Index of Retained Strength (AASHTO T167)
5. Air Voids (14)
6. Predicted Flooding Rainfall Intensity (37)
7. Freeze-Thaw Durability (9)

The mix designs that were considered and their engineering properties are shown in Table 15.

Marshall Stability. Streetman SEA-OGFC mixtures seem to possess the highest Marshall stabilities followed by the East Texas Sandstone mixture. However, the Marshall flow values in the Streetman mixtures fall outside The Asphalt Institute recommended range. On the other hand, the East Texas Sandstone mixture exhibit Marshall stability and flow values (550 lbs. and 19, respectively), which meet the requirement suggested by The Asphalt Institute (43). The limestone mixtures seem to have acceptable flow values despite their lower Marshall stabilities. Nevertheless, both Marshall stability and Marshall flow are important indicators of the ability of the mix to resist applied load without excessive deformation. Thus the East Texas Sandstone SEA-OGFC mixtures were considered to have the most favorable results among the mixes evaluated.

Hveem Stability. All the SEA-OGFC mixtures were found to have Hveem stabilities above the minimum specified by The Asphalt Institute (46). Therefore, all these mixtures are expected to perform adequately as a surface course in highway pavements. However, the highest Hveem stability was found in the East Texas Sandstone mixture. It had a stability value of 35 compared to the range of 31 to 33 in the other mixtures. Consequently, the East Texas Sandstone SEA-OGFC mixture was selected over to the other SEA-OGFC mixtures.

Table 15. Properties of Optimum SEA-OGFC Mix-Design for Each Aggregate System.

Mix Design	Marshall Stab., lb.	Marshall Flow, .01 in.	Hveem Stab., %	Resilient Modulus @ 68°F (20°C) psi	Air Voids %	Index of Retained Strength %	Predicted Flooding Rainfall Intensity I <sub>F</sub> , in/hr	Visual Observations at 100 Cycles	
8 East Texas Sandstone AC 20 40/60 SEA	550	19	35	530,000	23	64	.662	(1)	0
								(2)	0
								(3)	0
	390	16	33	761,000	23	73	.247	(1)	0
								(2)	0
								(3)	0
Limestone AC 40 20/80 SEA	410	19	32	346,000	23	64	.345	(1)	0
								(2)	2-1
								(3)	0
Streetman AC 10 30/70 SEA	680	26	31	230,000	23	62	.423	(1)	0
								(2)	0
								(3)	0
Streetman AC 20 30/70 SEA	670	27	31	280,000	26	63	.373	(1)	0
								(2)	0
								(3)	0

1 lb f = 4.5 N

1 in = 25.4 mm

1 in/hr = 0.0007 cm/sec

Resilient Modulus. Both the East Texas Sandstone SEA-OGFC mixture and limestone SEA-OGFC mixture (with AC 20 and 40/60 SEA binder) have resilient moduli comparable with other types of sulfur asphalt mixtures designed primarily to function as a structural component of a highway pavement (25, 26, 44, 45, 47). On the other hand, both of the Streetman SEA-OGFC mixtures exhibited resilient moduli below the minimum value specified in this study.

Air Voids. All except one SEA-OGFC mixtures have the same air voids content of 23 percent in the mixture. This value is comparable to those used in the recent improved OGFC mix designs placed in the districts of the Texas Highway Department (50).

Index of Retained Strength (IRS). Both East Texas Sandstone and limestone SEA-OGFC mixtures (with AC 40 and 20/80 SEA binder) have marginal IRS values of 64 percent. However, this value is well above the FHWA recommended minimum value of 50 percent.

Predicted Flooding Rainfall intensity ( $I_f$ ). It would appear that there is no correlation between the coefficient of permeability (or predicted flooding rainfall intensity) and air voids content in the mixtures. The data collected indicate that East Texas Sandstone SEA-OGFC mixtures possesses the highest  $I_f$  value of 0.662 in/hr. The two Streetman SEA-OGFC mixtures also seem to offer comparative values of 0.423 and 0.373 in/hr. Since the primary function of SEA-OGFC's is to provide drainage of surface water in a pavement, the maximization of the coefficient of permeability (or the predicted flooding rainfall intensity factor) is highly desired. Consequently, the East Texas Sandstone SEA-OGFC mixture was the most favorable among the mixtures considered in this section.

Freeze-thaw Durability. The data indicate that all five of the SEA-OGFC mixtures have good resistance to repeated cycles of freezing and thawing. Only the limestone (with AC 40 and S/A of 20/80) mixture showed minor aggregate fracture in one of its three tests after 100 cycles of repeated freezing and thawing.

On the basis of the above analyses, it would be most favorable to conclude that the East Texas Sandstone SEA-OGFC mixture prepared with AC 20 and S/A of 40/60 is the optimum SEA-OGFC mix design among all those tested in this study. The bases for this selection may be summarized as follows:

1. Marshall stability and flow values meet the requirements recommended by The Asphalt Institute (43).

2. The Hveem stability is the highest among the optimum SEA-OGFC mix designs for each aggregate system.

3. Resilient Modulus is comparable to those found in other types of sulfur-asphalt mixtures (25, 26, 44, 45, 47) which serve as structural components of a pavement.

4. Air voids content is comparable to those used in the recent improved OGFC mix designs placed in the districts of the Texas Highway Department.

5. Index of retained strength is above the minimum requirement established in this study.

6. It has the highest coefficient of permeability (or predicted flooding rainfall intensity) which indicates it can handle a higher rainfall intensity.

7. It has good durability to repeated cycles of freezing and thawing.

8. The sulfur content used to replace the asphalt binder required in the mixture is maximized.

#### Evaluation of Low Temperature Mixes

In this task the influence of compaction temperature on the structural integrity and drainage of SEA-OGFC mixes was evaluated. The optimum mix-design as determined above (East Texas Sandstone with AC 20 and 40/60 sulfur asphalt ratio) was selected to be studied here. In this treatment the term "Normal Temperature Mix" is used to indicate a mix which is prepared at 250°F (121°C) and compacted at 245°F (118°C); and the term "Low Temperature Mix" designates one which was prepared at 250°F (121°C) and compacted at 230°F (110°F). Hence the only difference between normal and low temperature mixes is the compaction temperature where low temperature mixes are compacted at a temperature below the crystallization temperature of sulfur.

The characterization test results of both normal and low-temperature mixes are shown in Table 16. The structural characterization test results, such as those from resilient modulus, Hveem and Marshall tests, indicate that the structural integrity of the mixes are adversely affected if the mixture is compacted at the low temperature. In addition, the loss of cohesion from water was higher in the low temperature mix, as reflected by the Index of Retained Strength. Although the IRS of the low temperature mix falls below the minimum requirement value of 60 in this study, it still meets the minimum value of 50 suggested by the FHWA (7). The amount of air voids

present in both mix types are comparatively the same. Moreover, the data show that the coefficient of permeability of both mixes are also the same which indicates an equivalent ability to drain water. Both of these mixes endured the minimum number of freeze-thaw cycles established in this study without any visible loss or damage.

Table 16. Comparison Between Normal and Low Temperature Mixes

	Normal Temperature	Low Temperature
Resilient Modulus @ 34°F (1°C)	2,355,000	1,424,000
(psi)                      68°F (20°C)	534,000	190,000
77°F (25°C)	356,000	81,000
Index of Retained Strength, %	64	58
Coefficient of Permeability, cm/sec	.502	.495
Freeze-Thaw Test	0	0
Hveem Stability, %	35	27
Marshall Stability, lbs.	550	320
Marshall Flow, .01 in.	19	24
Air Voids, percent	23	22
1 psi = 6.9 pKa 1in = 25.4mm 1bl f =4.5N 1 in/hr = 0.0007 cm/sec		

#### Loss of Permeability by Fouling

In this section, actual field data were used to evaluate the fouling potential which results while SEA-OGFC's are in service. The data were obtained from a SEA-OGFC field trial constructed on Loop 495 in Nacogdoches, Texas on August 7, 1980; (32) a four-lane highway with two northbound SEA-OGFC lanes and two southbound conventional OGFC lanes. The two southbound asphalt-OGFC lanes served as control sections in this experiment. Two sets of permeability field test data were analyzed which include: (a) data taken after completion of construction and (b) data taken nine months later. Results of these tests are shown in Table 17. The data indicate an

18 percent improvement in permeability reflected in the SEA system at the time the pavement was open to traffic. The SEA-OGFC suffered a 40 percent loss of permeability while the asphalt OGFC suffered only a 30 percent loss after the pavement surface course has been nine months in service. Although the margin of increase in permeability was significantly reduced, the data show that the nine month drainage abilities of SEA-OGFC's are still better than those in the conventional OGFC. The field test permeability measurements were performed in the wheel path of each lane and the factors involved in fouling the permeability in the surface course reflect the susceptibility of the two systems which were introduced by traffic, wheel load, sand and dust, debris from a nearby construction site, and possible oil and chemical spills from passing vehicles.

Table 17. Change in Permeability on Nacogdoches SEA-OGFC pavement after nine months service life.

	SEA-OGFC		Conventional OGFC	
	Lane		Lane	
	L	M	R	S
Original Permeability Measurements, Kv, cm/sec	.525	.533	.467	.432
Nine-Month Permeability Measurement, Kv, cm/sec	.279	.374	.283	.338
Loss of Permeability in Percent	46.8	29.8	39.4	21.8

1 psi = 6.9 pKa 1 in = 25.4 mm 1 lbf = 4.5N 1 in/hr = 0.0007 cm/sec

The next field permeability measurements from this project are due to be made during August of 1982. These results will indicate whether these fouling trends are continuing, have been stabilized or reversed.

#### Structural Considerations

Purpose. An open graded friction course is not expected to perform a structural function. However, it must possess certain minimum structural capabilities so that loads can be transmitted to underlying structural layers without deformation or fracture of the open graded friction layer.



The primary purpose of this section is to evaluate the capability of the SEA-OGFC to sustain high contact stresses without excessively deforming or without premature fracture or fatigue.

Analyses discussed in previous sections address the structural response of SEA-OGFC's with respect to Marshall and Hveem stability, Marshall flows and resilient modulus. Generally, SEA open graded friction courses have higher stabilities and stiffnesses than the transitional open graded mixtures and are, therefore, less likely to present signs of low stability distress. Hveem and Marshall stabilities and resilient moduli of the optimum SEA mixtures studied here are certainly adequate for thin open graded courses.

Testing Philosophy. The testing programs for determining structural response may be divided in two parts: 1) permanent deformation and 2) fatigue due to fracture caused and propagated by movement of the underlying structure-reflection cracking.

The initial approach was to use the VESYS II users manual (51) creep test to evaluate deformation potential and the controlled-stress beam fatigue test to evaluate fatigue potential. Beam fatigue testing of OGFC mixtures proved cumbersome and generally unreliable due to compaction difficulties, high void contents of the mix and the general nature of the open graded mixtures.

In addition open graded friction courses are supposed to be placed above well designed pavement systems. These systems should incorporate an asphalt concrete surface course, leveling course and perhaps base course of substantial stiffness. Tensile stresses of the magnitude required to cause load related fatigue fracture should simply not occur within the thin OGFC for even under-designed pavements. Thus the flexural beam fatigue test is not warranted in that it simulates a controlled stress mode of load-related fatigue failure not normally pertinent to the OGFC.

To illustrate this point the pavement sections in Figure 50 were evaluated using layered elastic techniques. As expected the tensile stresses developed in the OGFC were of no consequence with respect to load-induced flexural fatigue.

The most likely mode of fracture in an OGFC is that due to the propagation of cracks in the existing surface. These cracks probably existed at the time of the OGFC construction. This reflection cracking could lead to deterioration of the OGFC.

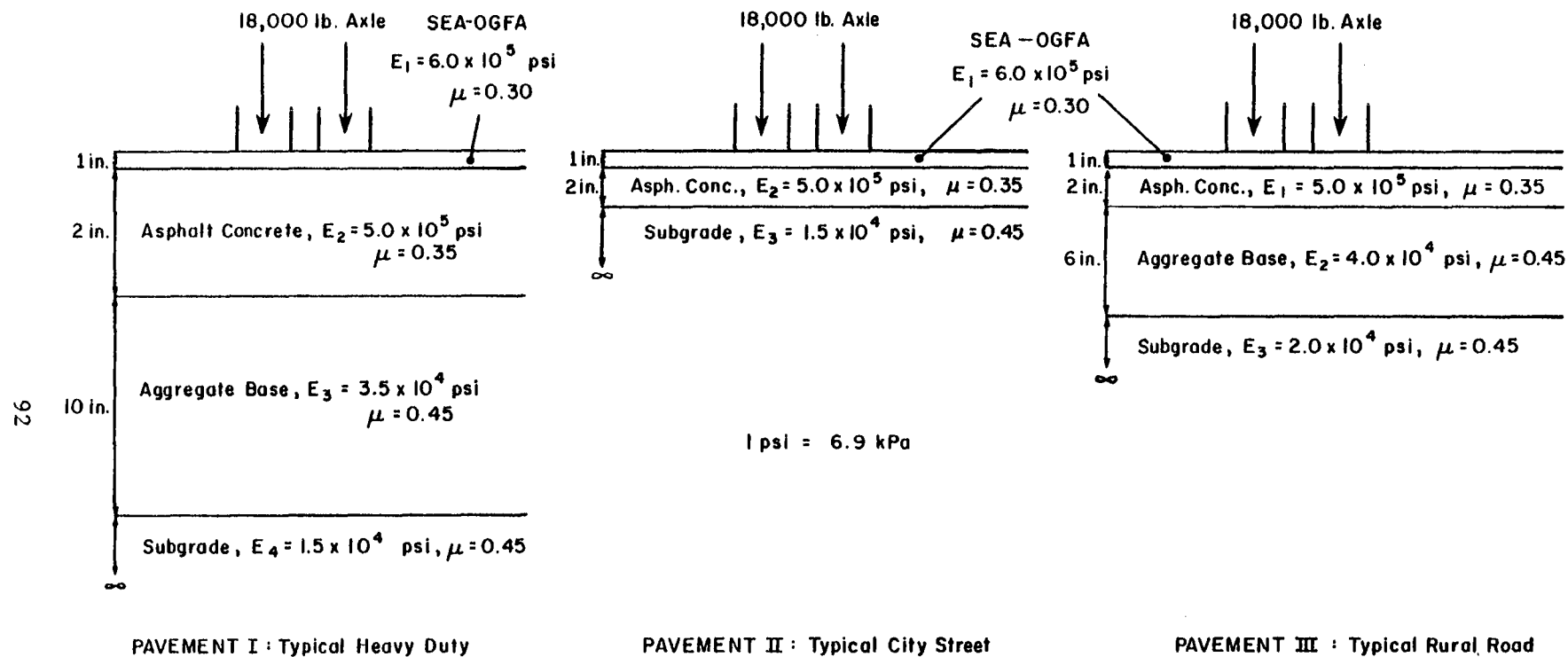


Figure 50. Typical Layered Elastic Pavement Section Used to Evaluate Tensile Strains in OGFC.

Thus a decision was made to approximate the fatigue fracture properties of the SEA OGFC based on the results of the creep test.

Shapery (52) has presented a theory of crack growth in viscoelastic materials in which he found a relationship between material properties of the viscoelastic material and the coefficients A and n in the Paris and Erdogan power law equation of crack growth based on the principles of fracture mechanics (53). This equation states that

$$\frac{da}{dn} = A K^n$$

where

$da/dn$  is the crack length per cycle,  
K is the change in the stress intensity factor and  
A and n are determined graphically.

Shapery's procedure predicts A as a function of maximum stress at separation of the material, stresses within the failure zone, Poisson's ratio, fracture energy, wave-shape of the stress intensity factor, the period of cyclic loading and the slope of the creep compliance versus load duration curve at the time required to propagate the crack.

The procedure predicts n as a function of the slope of the logarithmic creep compliance versus load duration curve, m,

$$n = 2\left(1 + \frac{1}{m}\right).$$

Germann and Lytton (54) showed that Shapery's theory can be extended to asphalt concrete. Their work with asphalt concrete overlays subjected to reflection cracking proved the validity of this application of Shapery's theory.

It was not our purpose to predict fatigue lives or crack propagation times in the SEA-OGFC's but to compare the fracture potential of the SEA-OGFC's with conventional OGFC's based on the results of the creep test which gives the necessary information to predict n. As n is the slope of the  $\frac{da}{dn}$  versus stress intensity factor curve during stable crack growth, it is considered a viable parameter for evaluating the crack propagation potential.

### Test Sequence

The sequence of testing is shown in Table 18. The SEA-OGFC mixtures with the three aggregate systems and the conventional OGFC mixtures with the three aggregate system and AC-10 were subjected to creep testing.

Table 18. Structural Test Sequence

<u>Mixture</u>	<u>Test</u>	<u>Temperature C</u>	<u>Replicates per Temperature</u>
SEA-Limestone	Creep & Permanent Deformation	0, 25, 40	3
SEA-East Texas Sandstone	Creep & Permanent Deformation	0, 25, 40	3
SEA-Streetman Aggregate	Creep & Permanent Deformation	0, 25, 40	3
AC-20 Limestone (Traditional)	Creep & Permanent Deformation	0, 25, 40	3

$$^{\circ}\text{F} = \frac{9}{5} (\text{C} + 32)$$

The essence of this study is to compare the permanent deformation and fracture potentials of the SEA-OGFC system with the conventional OGFC system.

## Results

Crack Propagation Potential. Creep tests were performed on the following open graded friction course mixtures:

1. AC-20 Asphalt with limestone aggregate,
2. SEA - limestone,
3. SEA - East Texas Sandstone and
4. SEA - Streetman lightweight aggregate.

All SEA mixtures contained the optimum percentage of sulfur based on the analyses previously discussed.

The results of the creep tests were plotted as the logarithm of time versus the logarithm of compliance,  $(D(t))$ . A master creep compliance curve was developed by employing  $\sigma_0$  the time-temperature super position principal. A power law equation form was used to express the relationship between compliance,  $D(t)$ , and time,  $t$ . This equation is of the well known form

$$D(t) = D_0 + D_1 t^m$$

where  $D_0$  = the glassy compliance,  
 $D_1$  = the intercept of the creep compliance curve on log-log plot with the one second time line and  
 $m$  = the slope of the logarithmic creep compliance curve between the glassy and rubbery compliance.

The values of  $m$  derived from the creep compliance curves are summarized in Table 19. Also in Table 19 are the corresponding approximated values of  $n$ , the slope of the log  $da/dn$  vs. log curve.

The typical value of  $n$  for a densely graded asphalt concrete is 4.00 based on both theoretical and empirical studies. However, one might expect crack propagation to be more rapid in high void material as presented by Epps and Monismith (55). Thus the steep slopes in Table 19 are not surprising.

The SEA mixtures possessed relatively steep slopes,  $n$ , of the  $da/dn = A\Delta K^n$  plot indicating a more rapid potential for propagation of cracks in the SEA open graded mixtures than in the control AC-20 open graded mixture with a limestone aggregate. Similar results have been noted (56) where the crack growth potential of sulfur extended, densely graded asphalt mixtures has been shown to exceed that of traditional densely graded asphalt mixtures (54).

Table 19. Approximate Slopes of Log da/dn versus Log  $\Delta K$  curves for Open Graded Mixtures.

<u>Mixture</u>	<u>Slope of Creep Compliance Curve, m</u>	<u>Approximate Slope, n of log da/dn vs. log <math>\Delta K</math></u>
AC-20 and Limestone	0.60	5.33
SEA and Limestone	0.34	7.88
SEA and East Texas Sandstone	0.41	6.88
SEA and Streetman Aggregate	0.40	7.00

$$n = 2(1 + 1/m)$$

These results must be substantiated by full scale crack growth studies. A valid requirement exists for further research in the area of crack growth based on fracture mechanics in sulfur extended asphalt overlays of any kind, dense or open graded. Crack propagation studies such as those performed on the overlay test at Texas A&M will allow one to predict the rate of reflective crack propagation in SEA overlays and thus the sensitivity of these materials to reflection in various payment systems.

Deformation Potential. The response of the SEA open graded mixtures and the control mixture to repeated load deformation testing is summarized in Figure 51. As can be seen all the open graded mixtures have similar deformation potential but the SEA mixtures have a greater resistance to deformation than the control mixture. This is very likely due to the structuring effect of the sulfur in the SEA binders.

A band of permanent strain versus load cycles responses is shown for densely graded asphalt concrete mixtures as a comparison.

The results of the permanent deformation testing substantiate the previous stability and resilient modulus testing indicating the SEA-OGFC mixtures are less likely to develop plastic deformation.

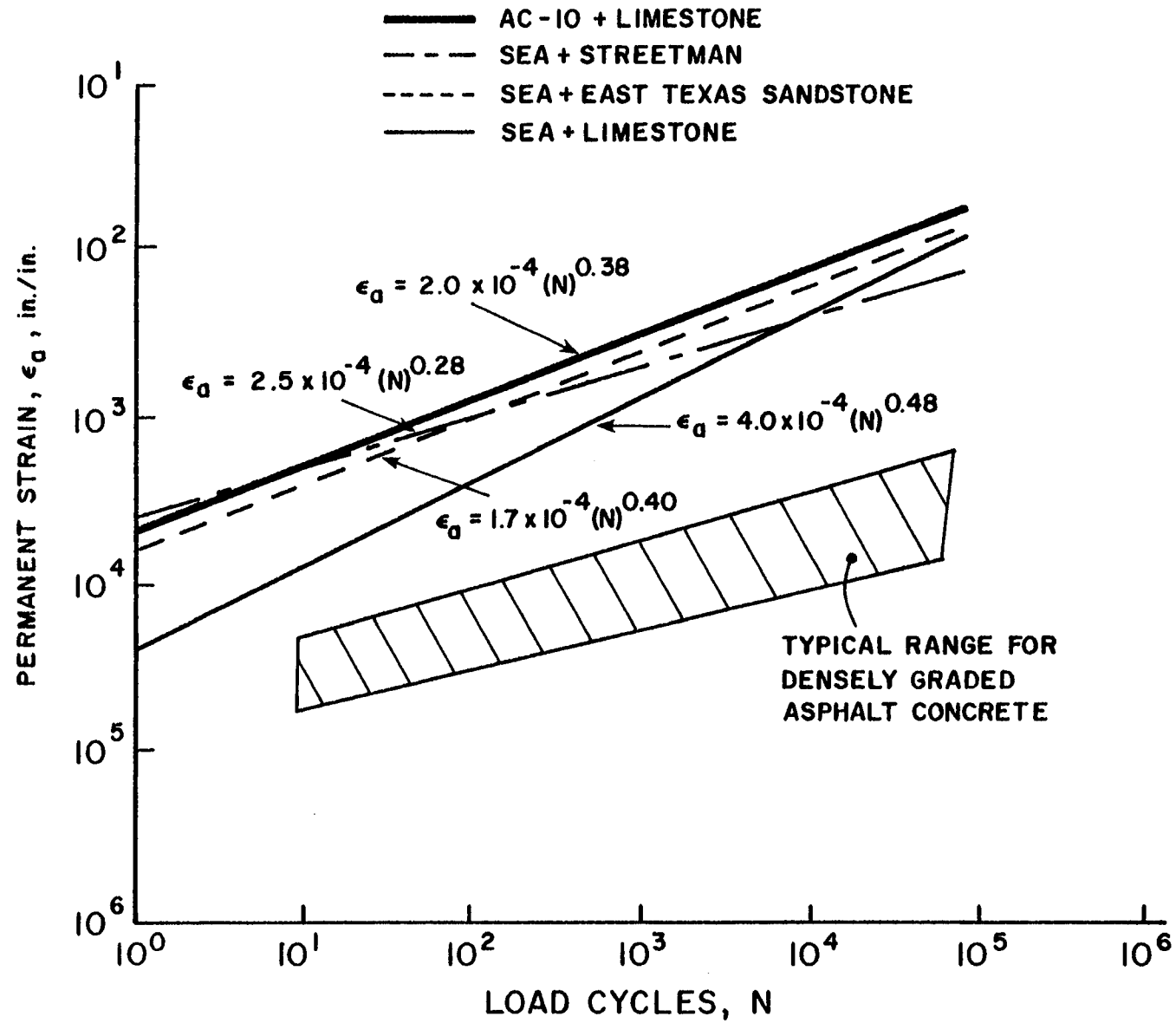


Figure 51. Permanent Deformation Versus Load Cycles.



## CHAPTER VI

### CONCLUSIONS

From the experimental data obtained in this study, the following conclusions were drawn:

1. The FHWA procedure (7), along with the proposed modifications for SEA-OGFC mixtures provided in Appendix A, can be used to characterize the materials used and to determine optimum mixing temperatures.

2. The method of blending sulfur with asphalt for the SEA binder does not to any statistical significance affect the engineering properties of the SEA-OGFC mixtures.

3. Conventional compaction procedures must be modified to avoid excessive crushing of lightweight aggregates during molding.

4. For all aggregate systems, Marshall stability increased with either the sulfur content or asphalt viscosity in the binder.

5. For East Texas Sandstone aggregate system, the Marshall flow values of all SEA-OGFC mixtures prepared with AC-20 fell within The Asphalt Institute recommended range. The same phenomenon occurred in the SEA-OGFC mixtures prepared with AC-40 for Limestone aggregate system. However, for Streetman aggregate system, all the flow values of the mixtures fell outside The Asphalt Institute recommended range.

6. For all aggregate systems, SEA-OGFC mixtures prepared with AC-20 exhibited Hveem stabilities greater than 30 percent. In addition, for Limestone aggregate system all SEA-OGFC mixtures prepared with AC-40 displayed Hveem stabilities higher than 30 percent. However, for Streetman aggregate system, all SEA-OGFC mixtures prepared with S/A weight ratio of 30/70 SEA binder had Hveem stability values greater than 30 percent, regardless of asphalt viscosity.

7. For all aggregate systems, the Marshall stability and Hveem stability were higher in SEA-OGFC's than in pure asphalt OGFC's.

8. For East Texas Sandstone and Limestone aggregate systems, an increase of sulfur content in the binder caused a resulting increase in the resilient modulus. In addition, SEA-OGFC mixtures prepared with AC-20 tended to have higher resilient moduli comparable to other sulfur asphalt mixtures.

9. For East Texas Sandstone and Limestone aggregate systems, the air voids content was not affected by the introduction of sulfur

into the binder. In addition, all these mixtures had the same air voids content of 23 percent which is comparable to that used in the recent mix designs placed in districts of the Texas Highway Department.

10. For the Streetman aggregate system, the air voids content of SEA-OGFC's is about 5 to 7 percent greater than in OGFC's using pure asphalt.

11. For East Texas Sandstone and Limestone aggregate systems, the index of retained strength decreases as sulfur content in the binder increases. However, all East Texas Sandstone SEA-OGFC mixtures have a retained strength greater than 60 percent.

12. For East Texas Sandstone and Streetman aggregate systems, SEA-OGFC mixtures prepared with AC-20 showed consistently superior overall structural properties to the other mixes within the same aggregate system.

13. For all aggregate systems, no correlation between the air voids content and permeability was evident in the data collected.

14. All the SEA-OGFC's tested have higher coefficients of permeability than pure asphalt OGFC's.

15. The optimum mix design for all aggregate systems considered in this study was the East Texas Sandstone SEA-OGFC prepared with AC-20 and 40 percent sulfur in the binder. It had structural characteristics comparable to the requirements specified by The Asphalt Institute for mixes used in highway pavements. The drainage capacity of this mix was comparatively higher than the other SEA mixes. It also possessed good resistance to repeated freezing and thawing. Finally, this mixture accommodates the maximum sulfur content criteria as considered in this study.

16. Improved permeability characteristics of laboratory-prepared SEA-OGFC mixtures were also indicated in the results of drainage tests generated in the field. Although initial field test results indicate some adverse fouling trends during service these data need further verification through additional samplings from the field.

17. SEA open graded friction courses have better resistance to permanent deformation than traditional asphalt cement bound open graded mixtures perhaps due to the structuring effects of the sulfur. However, SEA-OGFC's appear to be somewhat more susceptible to the propagation of cracks from movement in the base layers.

## CHAPTER VII

### RECOMMENDATIONS FOR FUTURE WORK

Based on the results of this investigation, the following recommendations are made:

1. Further study of SEA-OGFC with higher sulfur contents should be considered to maximize the usage of sulfur to replace the asphalt binder required in the mixture.
2. Additional research similar to that conducted by McBee (49) is needed to establish the effects sulfur recrystallization and changes in morphology have on structural integrity and in service performance of SEA systems.
3. Further study should examine the reasons for the improved permeability and the relative in service fouling trends of SEA and conventional OGFC systems.
4. Investigation should be made to evaluate the relative aging effects on OGFC and SEA-OGFC due to actinic light, ozone, etc.
5. A study should be conducted to evaluate the relative skid-resistance of the various SEA-OGFC mix designs considered in this study.
6. Investigation should be conducted to evaluate the environmental impact due to the possible sulfur run-off and leaching from SEA-OGFC's. These problems could have a bearing on the maximum allowable sulfur content in the mix.
7. The conclusions and concepts prepared here should be tested in the field under actual traffic conditions.

## REFERENCES VIII

1. Engineering News Record, April 24, 1980.
2. Smith, R. W., et al., "Design of Open-Graded Asphalt Friction Courses", Report FHWA-RD-74-2, January, 1974, as supplemented by FHWA Bulletin dated 11 July, 1975.
3. Szatkowski, W. S. and Brown, J. R., "The Design and Performance of Previous Surfacing for Roads in Britain 1967-76", Transport and Road Research Laboratory, Crowthorne, England, International Symposium on Porous Asphalt, The Netherlands, 1976.
4. Maupin, G. W., Jr., "Virginia Experience With Open-Graded Surface Mix", Virginia Highway and Transportation Research Council.
5. Kandhal, R. S., et al., "Design, Construction and Performance of Asphalt Friction Courses in Pennsylvania", Transportation Research Record No. 659, 1977.
6. Greby, E., "Drainage Asphalt in Denmark", a paper presented at the Conference of International Symposium on Porous Asphalt, The Netherlands, 1976.
7. "Open-Graded Friction Courses for Highways", NCHRP Report No. 49, Transportation Research Board, Washington, D. C., 1978.
8. "Experience in the Northwest With Open-Graded Emulsified Asphalt Pavements", Implementation Package 74-3, U. S. Department of Transportation, Federal Highway Administration, July, 1974.
9. Gallaway, B. M. and Epps, J. A., "Mixture Design Concepts, Laboratory Tests and Construction Guides for Open-Graded Bituminous Overlay", Research Report 36-1F, Study No. 2-10-74-36, Texas Transportation Institute, October, 1974.
10. "Open-Graded Asphalt Friction Courses", Construction Leaflet No. 10, The Asphalt Institute, November, 1974.
11. "Sulfur: 1980's Shortage or Glut"? Chemical Engineering, September, 1976.

12. Schmidt, R. J., "A Practical Method for Determining the Resilient Modulus of Asphalt Treated Mixes", Highway Research Record No. 404, 1972.
13. Britton, S. C., et al., "Performance of Open-Graded Friction Courses", Research Report 234-1F, Study No. 2-9-79-234, Texas Transportation Institute, June, 1979.
14. "Mix Design Methods for Asphalt Concrete and Other Hot-Mix Types", The Asphalt Institute, Manual Series No. 2, 4th Edition, March, 1974.
15. Eager, W. L., "Construction and Performance of Plant-Mix Seal Coats", a paper presented at 1967 convention of AASHTO, Salt Lake City, Utah, October 18, 1967.
16. The Sulfur Data Book, Freeport Sulfur Company, McGraw-Hill Book Company, Inc., 1954, New York, p. 3.
17. Texas A&M Research Foundation Contract No. RF 3259, "The Extension and Replacement of Asphalt Cement With Sulfur".
18. "Beneficial Uses of Sulfur-Asphalt Pavements", Volume I-A, I-B, I-C, II and III, Final Reports on Texas A&M Research Project RF 983, January, 1974.
19. Deme, I., Hammond, R. and McManus, D., "The Uses of Sand-Asphalt-Sulfur Mixes for Road Base and Surface Applications", Proceedings, Canadian Technical Asphalt Association, Vol. XVI, November, 1971.
20. Deme, I., "Basic Properties of Sand-Asphalt-Sulfur Mixes", presented to the VII International Road Federation World Meeting, Munich, Germany, October, 1973.
21. Deme, I., "Processing of Sand-Asphalt-Sulfur Mixes", presented at the Annual Meeting of the Association of Asphalt Paving Technologists, Williamsburg, Virginia, February, 1974.
22. Sullivan, T. A. and McBee, W. C., "Studies of Sand-Sulfur-Asphalt Paving Materials", U. S. Bureau of Mines RI 8087, 1975.
23. Texas A&M Research Foundation Contract No. RF 3146, "Evaluation of Sulfur-Asphalt Emulsion Binder Systems for Road Building Reposes".

24. Garrigues, C. and Vincent, P., "Sulfur/Asphalt Binder for Road Construction", Advances in Chemistry Series 140, American Chemical Society, Washington, D. C., 1975.
25. Izatt, J. O., "Sulfur-Extended Asphalt Field Trials, MH 153 Brazos County, Texas", Texas Transportation Institute, December, 1979.
26. Pickett, D. E., et al., "Extension and Replacement of Asphalt Cement With Sulfur", Report No. FHWA-RF-78-95 Final Report, Texas Transportation Institute, March, 1978.
27. Metcalf, C. F., "Bituminous Road Surfacing Mass Containing Sulfur", German Patent No. 1,295,463 (U. C. 10c), September 11, 1979.
28. Kennepohl, G. J. H., et al., "Conventional Paving Mixes With Sulfur Asphalt Binders", a paper presented at the Annual Meeting of the Association of Asphalt Paving Technologists, February, 1975.
29. Fromm, H. J., et al., "Sulfur-Asphaltic Concrete - Three Ontario Test Roads", a paper presented to the Annual Meeting of Asphaltic Paving Technologists, Denver, Colorado, 1979.
30. McBee, W. C., et al., "Direct Substitution of Sulfur for Asphalt in Paving Materials", U. S. Bureau of Mines, Report of Investigation RI 8303, 1978.
31. Saylak, D., et al., "Recycling Old Asphalt Concrete Pavement", Proceedings of the Fifth Mineral Waste Utilization Symposium, Chicago, Illinois, April 13-14, 1976.
32. Benson, F. C. and Gallaway, B. M., "Field Trial of Sulfur-Extended-Asphalt Binder in Open-Graded Friction Course Loop 495. Nacogdoches, Texas Transportation Research Report 547-1, January 1981.
33. White, T. D., "Construction and Evaluation of Airfield PFC in the United States", Soils and Pavement Laboratory, U.S. Army Engineer Waterway Experiment Station, Vicksburg, Mississippi, a paper presented at the International Symposium on Porous Asphalt, The Netherlands, 1976.
34. McBee, et al., "An Overview of Sulfur Extended Asphalt Usage", Asphalt Pavement Construction: New Materials and Technologies, ASTM STP 724, J.A. Scherocman, Ed., ASTM, 1980, pp. 39-63.

35. Gandhi, P. M. K., "Asphalt Absorption of Lightweight Synthetic Aggregates", unpublished report prepared as a term paper for CE 685 at Texas A&M University, College Station, Texas.
36. AASHTO Designation: T85, American Association of State Highway and Transportation Officials, Part II, 11th Edition, 1974.
37. AASHTO Designation: T84, American Association of State Highway and Transportation Officials, Part II, 11th Edition, 1974.
38. AASHTO Designation: T96, American Association of State Highway and Transportation Officials, Part II, 11th Edition, 1974.
39. Test Designation: Tex-206-F, Part II, Texas Highway Department Manual of Testing Procedures.
40. AASHTO Designation: T167, American Association of State Highway and Transportation Officials, Part II, 11th Edition, 1974.
41. Test Designation: Tex-126-E, Texas Highway Department Manual of Testing Procedures.
42. Gallaway, B. M. and Saylak, D., "Beneficial Use of Sulfur in Sulfur-Asphalt Pavements", Vol. III of III, Final Report on Texas A&M Research Project RF 983-IA, Texas A&M Research Foundation, Texas A&M University, January, 1974.
43. The Asphalt Institute, Manual Series No. 2, 4th Edition, March, 1974.
44. Newcomb, D. E., "Analysis of Sulfur-Asphalt Field Trials in Texas", a thesis as partial requirement for the M. S. degree, Texas A&M University, August, 1979.
45. Gallaway, B. M. and Saylak, D., "Sulfur/Asphalt Mixture Design and Construction Details - Lufkin Field Trials", Study No. 1-10-75-512, Texas Transportation Institute, January, 1976.
46. The Asphalt Institute, Manual Series No. 2, 4th Edition, March, 1974, p. 57.

47. Button, Joe, et al., "Test Results on Laboratory Standard-Asphalt, Aggregates and Mixtures", Texas A&M University, January, 1977.
48. Britton, S. C., et al., "Performance of Open-Graded Friction Courses", Research Report 234-1F, Study No. 2-9-79-234, Texas Transportation Institute, June, 1979, Appendix C.
49. McBee, W. C., et al., "Role of Sulfur in SEA Pavements", Sulphur Research and Development, Vol. 4, 1981, pp. 5-12.
50. Gallaway, B. M., et al., "Mixture Design Concepts, Laboratory Tests and Construction Guides for Open-Graded Bituminous Overlay", Research Report 36-1F, Study No. 2-10-74-36, Texas Transportation Institute, October, 1974, p. 8.
51. Kenis, W. J., "Predictive Design Procedures, VESY's Users Manual", Report No. FHWA-RD-77-154, January, 1978.
52. Shapery, R. A., "A Theory of Crack Growth in Viscoelastic Media", Mechanics and Materials Research Center, Texas A&M University, College Station, Texas, 1973.
53. Paris, P. C. and Erdogan, F., "A Critical Analysis of Crack Propagation Laws", Transactions ASME, Journal of Basic Engineering, Series D, 35, No. 3, 1963.
54. Germann, F. P. and Lytton, R. L., "Methodology for Predicting the Reflection Cracking Life of Asphalt Concrete Overlays", Research Report 207-5, Texas Transportation Institute, Texas A&M Univ., Mara, 1979.
55. Monosmith, C. C. and Epps, J. A., "Asphalt Mixture Behavior in Repeated Flexure", Report No. TE 69-6, University of California, Berkley, 1969.
56. Pickett, D. E., Saylak, D., Lytton, R. L., Conger, W. W., Newcomb, D. and Shapery, R. A., "Extension and Replacement of Asphalt Cement with Sulfur", FHWA Report RD-78-95, March, 1975.
57. Little, D. N., Epps, J. A. and Holmgreen, R. J., "The Effect of Recycling Agents on the Structural Properties of Recycled Asphalt Concrete Materials", Vol. 49, Proceedings Association of Asphalt Paving Technologist, 1981.



APPENDIX A  
FEDERAL HIGHWAY ADMINISTRATION DESIGN METHOD  
FOR  
OPEN-GRADED ASPHALT FRICTION COURSES  
(MODIFIED FOR USE WITH  
SYNTHETIC AGGREGATES)

## Material Requirements

1.1. It is recommended that relatively pure carbonate aggregates or any aggregates known to polish be excluded from the coarse-aggregate fraction [material retained on the No. 8 (2.38mm) sieve]. In addition, the coarse-aggregate fraction should have at least 75 percent (by weight) of particles with at least two fractured faces and 90 percent with one or more fractured faces. The abrasion loss (AASHTO T 96) should not exceed 40 percent.

1.2. Recommended Gradation for Open-Graded Asphalt Friction Course.

<u>Sieve Size<sup>a</sup></u>	<u>Percent Passing<sup>b</sup></u>
1/2 in. (12.7mm)	100
3/8 in. (9.52mm)	95-100
No. 4 (4.76mm)	30-50
No. 8 (2.38mm)	5-15
No. 200 (0.074mm)	2-5

---

<sup>a</sup>U. S. sieve series.

<sup>b</sup>By volume. (This is the same as by weight unless specific gravities of aggregates being combined are different).

1.3 The recommended grades of asphalt cement are AC-10, AC-20 or AR-40, AASHTO M 226. For AC-10 and AC-20, the M 226 Table 2 requirements should apply where such asphalt is available. AR-40 requirements are given in Table 3 of M 226.

## Preliminary Data

2.1. Test coarse and fine aggregates as received for the project for gradation unless otherwise provided. If mineral filler is submitted as a separate item, it should also be tested for specification compliance. Analyze gradation results to determine if proportions of aggregates and batching operations proposed by the contractor will meet the job-mix formula and the specification limits of Step 1.2.

2.2. Determine bulk and apparent specific gravity for the coarse and fine aggregate fractions (retained and passing the No. 8 (2.38mm) sieve) for each type of material submitted. Additional specific gravity tests are not warranted when the only distinction between aggregates is size of grading. Using the information verified in Step 2.1., mathematically compute the bulk and apparent specific gravity for the coarse and fine aggregate fractions [retained and passing the No. 8 (2.38mm) sieve] for the proposed job-mix gradation.

2.3. Test the asphalt cement to be used for specification compliance (AASHTO M 226), viscosity-temperature data, and specific gravity at 77.0°F (25°C).

#### Asphalt Content

3.1. Determine the surface capacity of the aggregate fraction that is retained on a No. 4 (4.76mm) sieve in accordance with the following procedure (25).

Note: For highly absorptive aggregates, use the procedure described in Step 3.3.

$K_c$  is determined from the percent of SAE No. 10 oil retained, which represents the total effect of superficial area, the aggregate's absorptive properties and surface roughness.

3.1.1. Quarter out 105 g representative of the material passing the 3/8-in. (9.52mm) sieve and retained on the No. 4 (4.76mm) sieve.

3.1.2. Dry sample on hot plate or in  $230 \pm 9^\circ\text{F}$  ( $110 \pm 5^\circ\text{C}$ ) to constant weight and allow to cool.

3.1.3. Weight out 100.0 g and place in a metal funnel (top diam 3-1/2 in. (88.9mm), height 4-1/2 in. (114.3mm), orifice 1/2 in. (12.7 mm), with a piece of No. 10 sieve soldered to the bottom of the opening).

3.1.4. Completely immerse specimen in SAE No. 10 lubricating oil for 5 min.

3.1.5. Drain for 2 min.

3.1.6. Place funnel containing sample in 140°F (60°C) oven for 15 min. of additional draining.

3.1.7. Pour sample from funnel into tared pan; cool and reweigh sample to nearest 0.1 g. Subtract original weight and record difference as percent oil retained (based on 100 g of dry aggregate)

3.1.8. Use chart shown in Figure A-1 for determination of  $K_c$ .

(a) If specific gravity for the fraction is greater than 2.70 or less than 2.60 apply correction to oil retained, using formula at bottom of chart in Figure A-1.

(b) Start at the bottom of chart in Figure A-1 with the corrected percent of oil retained; follow straightedge vertically upward to intersection with the diagonal line; hold point and follow the straightedge horizontally to the left. The value obtained is the surface constant for the retained fraction and is known as  $K_c$ .

3.2 Determine the required asphalt content, which is based on weight of aggregate, from the following relationship (2):

$$\text{Percent asphalt} = (2.0 K_c + 4.0) \times \frac{2.65}{(SG)_{ca}}$$

Where  $K_c$  = surface constant

(SG)<sub>ca</sub> = apparent specific gravity of coarse aggregate; 3/8 in to No. 4 (9.5 mm to 4.76 mm)

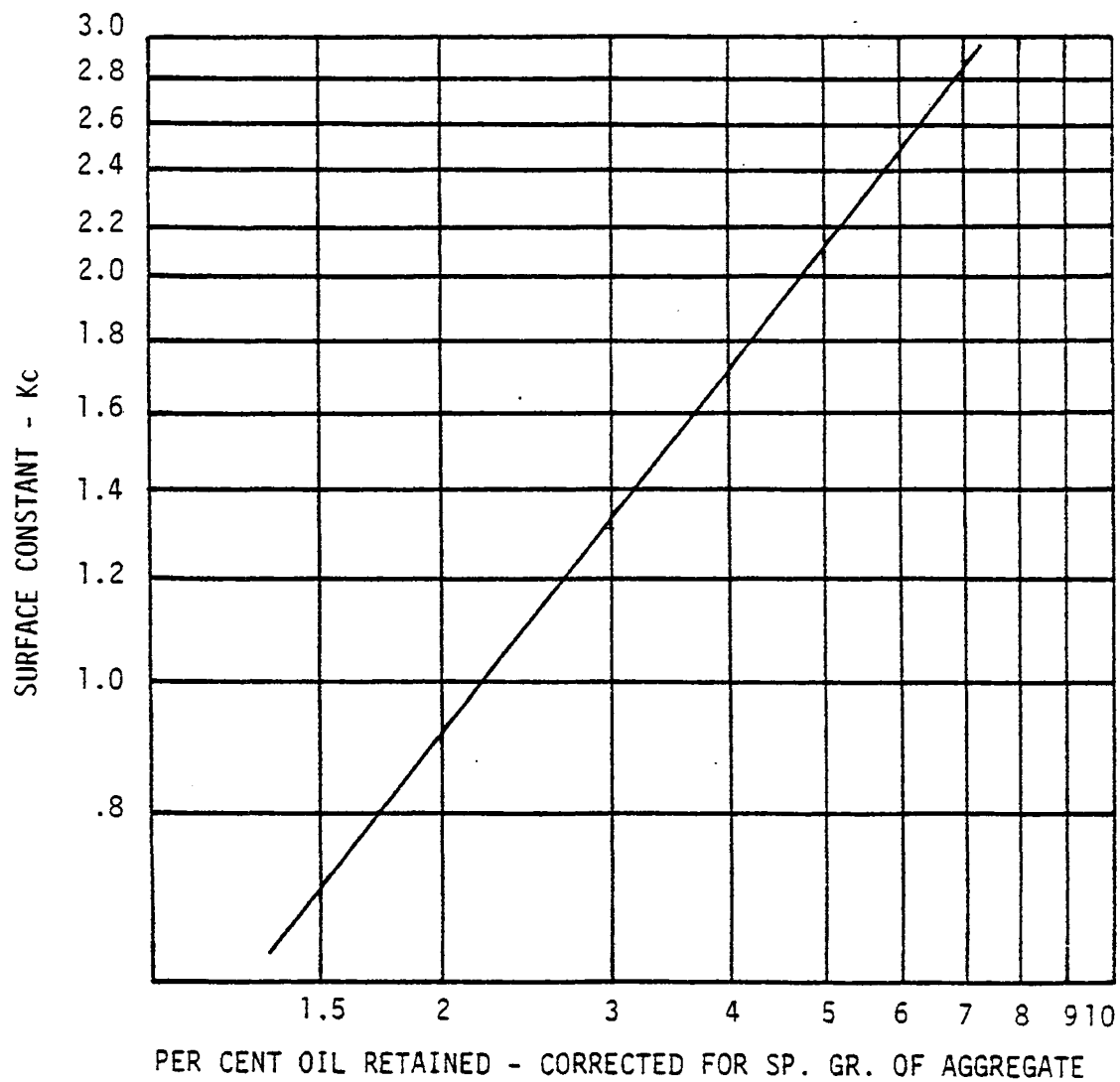
3.3 For highly absorptive aggregates, use the following procedure for determining  $K_c$  and asphalt content.

3.3.1. Follow the recommended design procedure from Step 3.1. through Step 3.1.3.

3.3.2. Follow the instructions in Step 3.1.4., except immerse the specimen for 30 min.

3.3.3. Follow the recommended procedure from Step 3.1.5. through Step 3.1.7.

3.3.4. Pour the sample onto a clean, dry, absorptive cloth; obtain a saturated surface dry condition; pour sample from cloth into a tared pan; reweigh sample to nearest 0.1 g. Subtract original weight of aggregate and record difference as percent oil absorbed (based on 100 g of aggregate).



Material Used: Aggregate - Passing 3/8", Ret. No. 4 Sieve  
Oil - SAE 10

$$\text{Oil Retained Corrected (\%)} = \text{Oil Retained (\%)} \times \frac{\text{"apparent" sp. gr. of Coarse Aggregate}}{2.65}$$

Figure A-1 Chart for determining surface capacity (Kc) of coarse aggregate

3.3.5. Subtract the percent oil absorbed value (see 3.3.4. above) from the percent oil retained value (see 3.3.3. above), and obtain the percent (free) oil retained value. This value represents the percent oil retained value that would have been obtained had the aggregate been a nonabsorptive type. The above technique allows one to evaluate the aggregate's surface and shape characteristics without the overwhelming influence of a large quantity of absorbed oil.

3.3.6. Follow the procedure recommended in Steps 3.1.8. and 3.2. The only exception is that the percent (free) oil retained value is used (from step 3.3.5.) to obtain  $K_s$ . Thus, the asphalt quantity determined is the "effective" asphalt content".

3.3.7. Follow the recommended procedure indicated through Sections 4 and 5. Because asphalt absorption is not presently included in the formula for the determination of fine aggregate content, it is particularly desirable that the effects of oil absorption be excluded in the case of highly absorptive aggregate.

3.3.8. Prepare a trial mixture using an asphalt content equal to or somewhat greater than (try to estimate amount that will be absorbed) the effective asphalt content determined in Step 3.3.6. and also using the aggregate gradation as determined in Step 3.3.7. Using a suitable technique, such as the test for maximum specific gravity of asphalt mixtures (AASHTO T 209), determine the actual quantity of asphalt absorbed (in percent, based on total weight of aggregate).

3.3.9. Determine the total asphalt content of the subject mixture by adding the effective asphalt content (from Step 3.3.6.) to the absorbed asphalt content (from Step 3.3.8.).

3.3.10. Follow the recommended procedure indicated in Sections 6 and 7, using the total asphalt content for all subsequent computations and trials (from Step 3.3.9.).

#### Void Capacity of Coarse Aggregate

4.1. Use the following procedure to determine the vibrated unit weight and void capacity of the coarse-aggregate fraction (material retained on a No. 8 (2.38mm) sieve) of the proposed job-mix gradation (26).

##### 4.1.1. Apparatus

Rammer - A portable electromagnetic vibrator as shown in Figure A-2, having a frequency of 3,600 cycles per min, suitable for

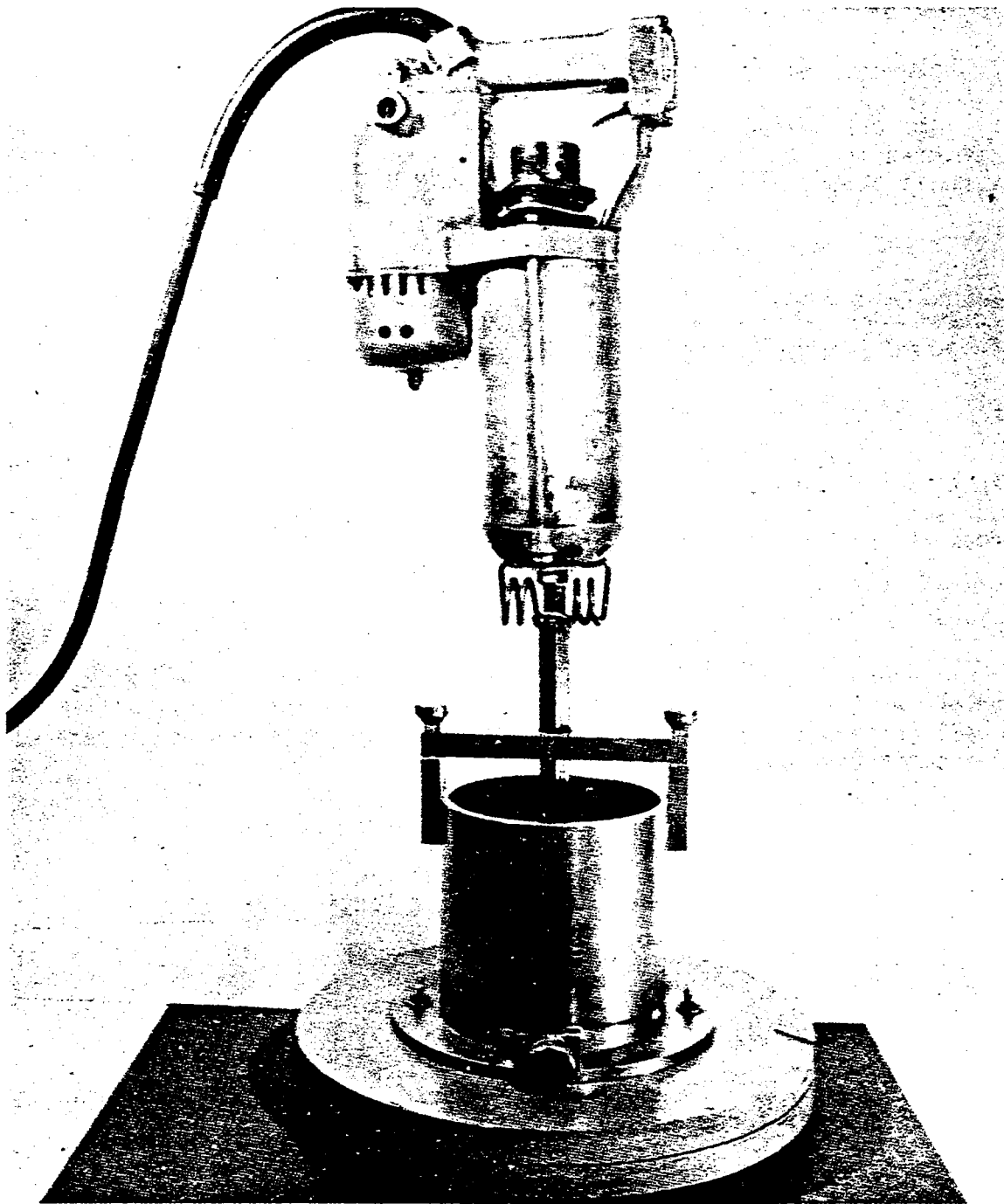


Figure A-2. FHWA vibratory compaction apparatus. (Reference 2)

use with 115-v ac. The rammer shall have a tamper foot and extension as shown in Figure A-3.

Mold - A solid-wall metal cylinder with a detachable metal base plate and a detachable metal guide-reference bar as shown in Figure A-4.

Wooden Base - A plywood disc 15 in. (381.0mm) in diam., 2 in. (50.8mm) thick, with a cushion of rubber attached to the bottom. The disc shall be constructed so it can be firmly attached to the base plate of the compaction mold.

Timer - A stopwatch or other timing device graduated in divisions of 1,0 sec. and accurate to 1.0 sec. and capable of timing the unit for up to 2 min. An electric timing device or electrical circuits to start and stop the vibratory rammer may be used.

Dial Indicator - A dial indicator graduated in 0.001-in. (.025mm) increments and having a travel range of 3.0 in. (76.2mm).

4.1.2. Sample: Select a 5-lb (2.269kg) sample of the coarse-aggregate fraction from the proposed job-mix formula as verified in Step 2.1.

#### 4.1.3. Procedure

(a) Pour the selected sample into the compaction mold and place the tamper foot on the sample.

(b) Place the guide-reference bar over the shaft of the tamper foot and secure the bar to the mold with the thumb screws.

(c) Place the vibratory rammer on the shaft of the tamper foot and vibrate for 15 sec. During the vibration period, the operator must exert just enough pressure on the hammer to maintain contact between the sample and the tamper foot.

(d) Remove the vibratory rammer from the shaft of the tamper foot and brush any fines from the top of the tamper foot. Measure the thickness (t) of the compacted material to the nearest 0.001 in (.025mm).

Note: The thickness (t) of the compacted sample is determined by adding the dial reading, minus the thickness of the tamper foot, to the measured distance from the inside bottom of the mold and the end of the dial gauge when it is seated on the guide-reference bar with stem fully extended.



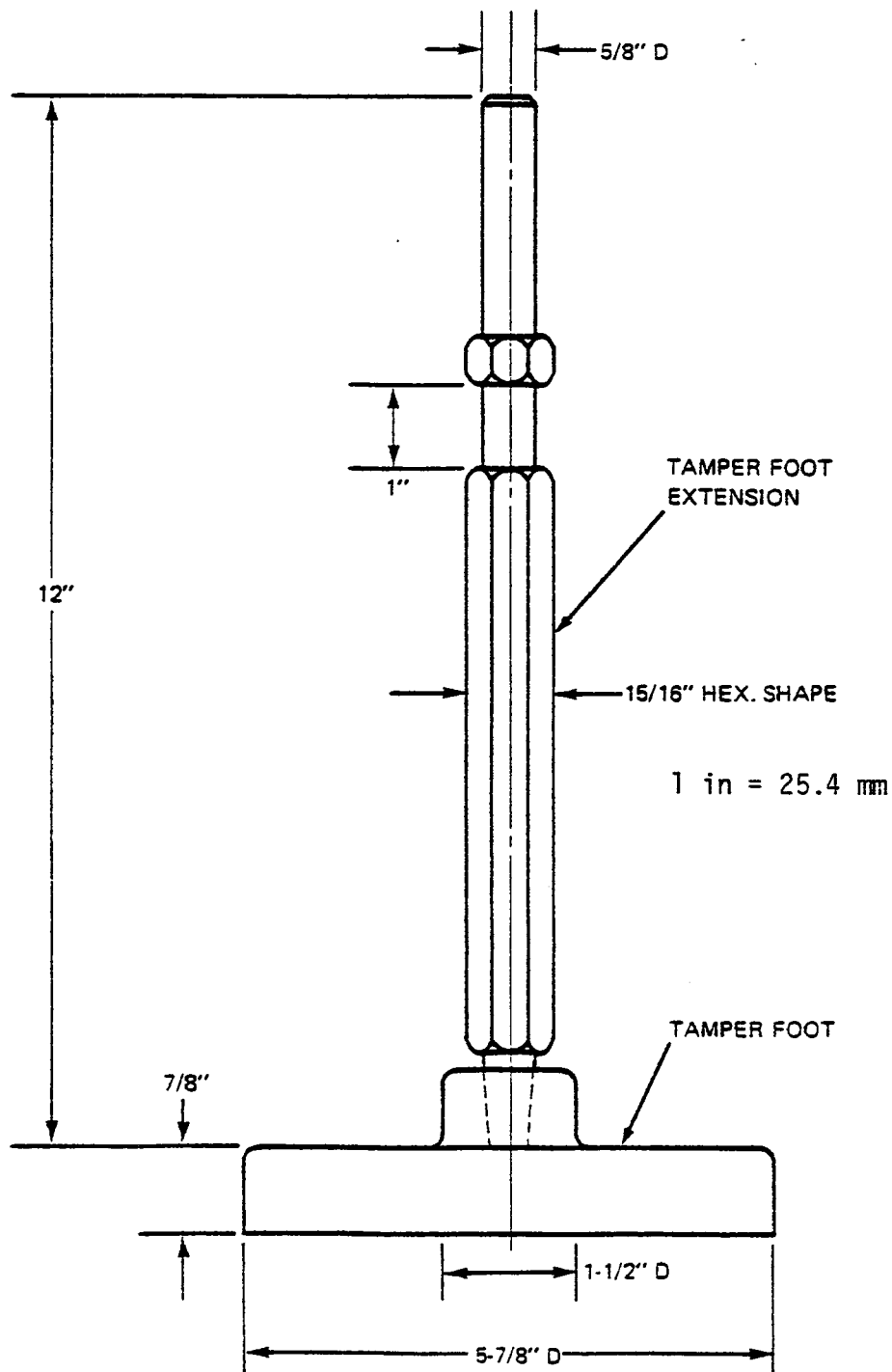


Figure A-3. Tamper foot and extension. (Reference 2)



#### 4.1.4. Calculations.

Calculate the vibrated unit weight (X) as follows:

$$X = 6912(w)/\pi(d)^2t(\text{lb/ft}^3)$$

Where w = wt of coarse-aggregate fraction (lb)

d = diam. of compaction mold (in.)

If w = 5 lb. and d = 6 in:

$$X = 305.58/t (\text{lb/ft}^3)$$

where t is in inches

Determine the void capacity (VM) as follows:

$$\text{VMA} = 100(1 - X/U_c) \text{ (in percent)}$$

Where  $U_c$  = bulk solid unit weight ( $\text{lb/ft}^3$ ) of the coarse-aggregate fraction.  $U_c$  is calculated from bulk specific gravity, as determined in Step 2.2<sup>c</sup>, multiplied by 62.4  $\text{lb/ft}^3$ .

#### Optimum Content of Fine Aggregate

5.1. Determine the optimum content of fine-aggregate fraction using the following relationship:

$$Y = \frac{[\% \text{ VMA} - V] - [(\% \text{ AC}) (X)/U_a]}{[(\% \text{ VMA} - V)/100] + [(X)/U_f]}$$

Where:

Y = Percent passing the No. 8 (2.38mm) sieve by weight

X = Actual vibrated unit weight of coarse aggregate (retained on the No. 8 (2.38mm) sieve)

$U_f$  = Theoretical bulk dry solid unit weight of fine aggregate (passing the No. 8 (2.38 mm) sieve.

$U_a$  = Unit weight of asphalt cement

% AC = Percent asphalt by total weight of aggregate

$$(2.0 K_c + 4.0) \frac{2.65}{(\text{SG})} \text{ ca}$$

$V$  = Design percent air voids (15.0 percent)

% VMA = Percent voids mineral aggregate of the coarse aggregate (retained on the No. 8 (2.38mm) sieve), which is  $100 - (100)(X)/U_c$

$U_c$  = Theoretical bulk dry solid unit weight of coarse aggregate (retained on the No. 8 (2.38mm) sieve).

Note:  $X$ ,  $U_a$ ,  $U_c$ , and  $U_f$  are in pounds per cubic foot.

In the above relationship, asphalt absorption by aggregate has been assumed to be negligible. Because asphalt absorption requirements are considered in the test for  $K_c$  (see Step 3.1.), the estimated air voids of 15 percent in the mixture will actually be greater by an amount equivalent to the volume of asphalt absorbed, in percent. This condition provides, if anything, an additional safety factor.

If the value thus obtained for fine-aggregate content is greater than 15 percent, a value of 15.0 percent shall be used.

5.2. Compare the optimum fine-aggregate content ( $Y$ ) determined in Step 5.1. to the amount passing the No. 8 (2.38mm) sieve of the contractor's proposed job-mix formula. If these values differ by more than plus or minus 1 percentage point, reconstruct a revised or adjusted job-mix formula using the value determined for optimum fine-aggregate content. Recompute the proportions of coarse and fine aggregates (as received) to meet the revised job-mix formula for submission to the contractor.

Note: If the proposed and revised job-mix gradations are significantly different, it may be necessary to rerun portions of this procedure.

### Optimum Mixing Temperature

6.1. Prepare a 1,000-g (2.20 lb.) sample of aggregate in the proportions determined in Section 5. Mix this sample at the asphalt content determined in Step 3.2. at a temperature corresponding to an asphalt viscosity of 800 centistokes determined in Step 2.3. When the mixture is completely coated, transfer it to a 8 to 9 in (203.2 to 228.6 mm) diameter pyrex glass plate and spread the mixture with a minimum of manipulation. Return it to the oven at the mixing temperature. Observe the bottom of the plate after 15 and 60 min. A slight puddle at points of contact between aggregate and glass plate is

suitable and desirable. Otherwise, repeat the test at a lower mixing temperature, or higher if necessary.

Note: If asphalt drainage occurs at a mixing temperature that is too low to provide for adequate drying of the aggregate, an asphalt of a higher grade should be used.

#### Resistance to Effects of Water

7.1. Conduct the Immersion-Compression Test (AASHTO T 165 and T 167) on the designed mixture. Prepare samples at the optimum mixing temperature determined in Step 6.1. Use a molding pressure of 1,000 psi (6,900 kPa) rather than the specified value of 3,000 psi (20,700 kPa).

After a four-day immersion at 120°F (48.9°C), the index of retained strength shall not be less than 50 percent unless otherwise permitted.

Note: Additives to promote adhesion that will provide adequate retained strength may be used when necessary.

## Background

In the preparation of sulfur-extended asphalt mixtures, sulfur is usually handled in liquid form. There are several potential hazards associated with handling liquid sulfur, such as fires, explosion and burns, but these hazards are similar to those associated with handling hot molten asphalt. However, one of the major concerns of the pavement industry in the development of the sulfur-asphalt mixtures has been the potential hazards created at the operation site due to the evolution of toxic gases such as hydrogen sulfide ( $H_2S$ ), and sulfur dioxide ( $SO_2$ ) and particulate elemental sulfur. The reason for the concern is the fatal effects on human if either or both of these toxic gases are inhaled in excess of their maximum allowable concentration (MAC) amounts. In the case of particulate sulfur, these airborne dusts can cause irritation to the eyes on contact. A separate discussion of each of these gases and particulate sulfur are given below.

## Hydrogen Sulfide Gas

Hydrogen sulfide possesses a characteristic "rotten-egg" odor which can be detected at a level of concentration as low as 0.02 ppm (by volume) (Reference B-1). However, odor is not a good indicator of concentration level because a high concentration of hydrogen sulfide can have a paralyzing effect on the sense of smell (Reference B-2). Consequently, high concentrations of  $H_2S$  can escape olfactory detection. Nevertheless, the inhalation of a high concentration of hydrogen sulfide gas can produce a poisoning effect with complete arrest of respiration. At lower concentrations, the symptoms may be stomach distress, coughing, headache, irritation of the eyes, nose, throat and blistering of the lips. Irritation may also result from skin contact with hydrogen sulfide (Reference B-3). The relative toxicity of  $H_2S$  and the corresponding physiological responses are presented in Tables B-1 and B-2 (Reference B-4). Over the years Shell Canada has monitored  $H_2S$  gas in the laboratory. Shell indicated that as long as the temperature of the mixture is kept below 300°F (149°C) the concentration of  $H_2S$  produced is well below the maximum allowable concentration (MAC) of 5 ppm (by volume) as suggested by the American Conference of Government Industrial Hygienists (ACGIH). Similar studies at Texas Transportation Institute and the Bureau of Mines support this claim (Reference B-5).

There are several areas in the laboratory that tend to be more susceptible to higher concentrations of  $H_2S$ , such as the storage area for liquid sulfur, the vicinity of the mixing bowl and compaction equipment, the area directly above the hot mixtures, and possibly the

Table B-1. Physiological Symptoms at Various Levels of Exposure to H<sub>2</sub>S.

Concentration, ppm	Physiological Symptoms
0.02	Odor threshold
0.1	Beginning eye irritation
5-10	Suggested maximum allowable concentration (MAC)
50-100	Slight conjunctivitis and respiratory tract irritation after 1 hour of exposure
100	Coughing, eye irritation, loss of sense of smell after 2 to 15 minutes  Altered respiration, pain in the eyes, drowsiness after 15 to 30 minutes, followed by irritation after 1 hour  Several hours exposure results in gradual increase in severity of these symptoms and death may occur within the next 48 hours
200-300	Marked conjunctivitis and respiratory tract irritation after 1 hour of exposure
500-700	Loss of consciousness and possibility of death in 1/2 to 1 hour
700-1000	Rapid unconsciousness, cessation of respiration and death
1000-2000	Unconsciousness at once, with early cessation of respiration and death in a few minutes (death may occur even if the individual is removed to fresh air at once).

Table B-2. Olfactory Responses to Various Concentrations of H<sub>2</sub>S.

Concentration, ppm	Olfactory Response
<0.02	No odor
0.13	Minimal perceptible odor
0.77	Faint but readily perceptible odor
4.6	Easily detectable, moderate odor
27.0	Strong, unpleasant odor, but not intolerable
<90.0	Can no longer smell H <sub>2</sub> S; paralysis of olfactory nerves



interior of the oven in which SEA mixtures are sometimes reheated.

In the field, toxic and explosive quantities of hydrogen sulfide gas can collect in the domes of transports and in the domes of tanks containing sulfur (Reference B-4). Hydrogen sulfide gas may also be detected around the mixing unit and above the pugmill. Above the pugmill, these fumes of gas may be visible as plumes of smoke during the batch loading of trucks. Additionally, smoke and fumes containing  $H_2S$  gas can also be found on an overheated paver screed.

#### Sulfur Dioxide Gas

Sulfur dioxide is a colorless gas with a pungent odor which unlike  $H_2S$  gives ample warning of its presence. The major health hazard from  $SO_2$  comes from the inhalation of excessive quantities above its MAC of 5 ppm (by volume). Sulfur dioxide is highly irritating to the eyes, nose, throat and lungs in concentrations greater than 6 to 20 ppm. At 150 ppm, exposure is almost unbearable. Concentrations in excess of 500 ppm can result in suffocation. The National Institute for Occupational Safety and Health (NIOSH) and the Manufacturing Chemists Association have documented the relationships between  $SO_2$  concentrations and the corresponding human effects (Reference B-6, B-7) and they are shown in Table B-3. Shell Canada has also stated that the level of concentration of  $SO_2$  is well below the MAC if the temperature of the mix is maintained below 300°F (149°C).

As in the case of  $H_2S$ , high concentrations of  $SO_2$  may be found, depending on the temperature, around the storage area for liquid sulfur, in the vicinity of the mixing bowl and compaction equipment, directly above the hot mixtures, and also inside the oven in which SEA mixtures are sometimes reheated. In the field, sulfur dioxide gas may be found in the same areas where hydrogen sulfide gas occurs.

#### Particulate Sulfur

A certain amount of undissolved and unreacted sulfur is contained in the vapor which is given off during mixing and compaction. These sulfur vapors, upon contact with cool air, can be crystallized into small particles which are dispersed in the air in a manner similar to dust and fine sands. The principle problems associated with sulfur dust lie in its contact with eyes. It is capable of irritating the inner surface of the eyelids. This problem can be minimized if goggles are worn in work areas subject to this pollutant.

Table B-3. Toxicity of Sulfur Dioxide.

Concentration, ppm	Effects
0.3-1	Detected by taste
1	Injurious to plant foliage
3	Noticeable odor
5	Maximum allowable concentration (MAC)
6-12	Immediate irritation of nose and throat
20	Irritation to eyes
50-100	MAC for 30-60 minutes exposure
400-500	Immediately dangerous to life

## Summary and Recommendations

The hazards in handling liquid sulfur are most likely to be the same as those associated with hot molten asphalt, such as fires, explosions, and burns. Liquid sulfur can be handled safely with the realization that it is hot, it can burn, and the vapor is irritating to the eyes.

The primary concerns of the industry throughout the development of the sulfur-asphalt mixtures has been the potential hazards created due to the evolution of toxic gases ( $H_2S$  and  $SO_2$ ) and particulate sulfur. Inhalation of a high concentration of hydrogen sulfide gas can produce a serious poisoning, symptoms may be stomach distress, headache and irritation of the eyes, nose and throat. Exposure to sulfur dioxide gas results in immediate and severe irritation to eyes, nose and throat. However, neither systemic effects nor chronic effects are known. The National Institute for Occupational Safety and Health (NIOSH) and the Manufacturing Chemists Association have documented the relationship between  $H_2S$  and  $SO_2$  concentration in ppm and human effects. On the basis of these effects, a maximum allowable concentration (MAC) value of 5 ppm (by volume) is normally specified as the upper limit for continuous exposure to both  $H_2S$  and  $SO_2$  emissions. A two-year study on the environmental and safety aspects of the use of sulfur in highway pavements have been made by Saylak, et al., at Texas Transportation Institute under FHWA Contract No. DOT-FH-11-9457 and the results of the study are about to be published by FHWA (Reference B-4). The study indicated that if the temperature of the mix is maintained below 300°F (149°C),  $H_2S$  and  $SO_2$  emissions are generally sufficiently lower than the maximum allowable concentration (MAC) of 5 ppm in those work areas where construction personnel are expected to be present. Therefore, it is recommended that sulfur and its mixtures are handled below 300°F (149°C) at any time. In addition, all work areas should be well ventilated.

Particulate sulfur comes from the crystallization of sulfur vapors which come into contact with cool air. These sulfur particles, upon contact with eyes, are capable of irritating the inner surfaces of the eyelids. Therefore, it is suggested that safety goggles be worn by all personnel in areas subject to this pollutant.

#### REFERENCES

- B-1. Elkins, Harvey B., The Chemistry of Industrial Toxicology, John Wiley and Sons, Inc., New York, 1950.
- B-2. Stoleman, A., Editor, Progress in Chemical Toxicology, Academic Press, New York, Vol. 1, 1965.
- B-3. Hydrogen Sulfide, Chemical Safety Data Sheet DS-36, Manufacturing Chemist Association, Washington, D. C., (Revised 1968).
- B-4. Saylak, D., et al., "Environmental and Safety Aspects of the Use of Sulfur in Highway Pavements", Vol. II of III, FHWA Contract No. DOT-FH-11-9457, Texas Transportation Institute, August, 1980.
- B-5. Izatt, J. O., et al., "Sand-Asphalt-Sulfur Pavements Experimental Project Highway U. S. 77, Kenedy County Texas - A Construction Report", Texas Transportation Institute, April, 1977.
- B-6. Occupational Exposure to Sulfur Dioxide, Criteria Document, U. S. Department of Health, Education and Welfare Document published by Public Health Service Center for Disease Control, National Institute for Occupational Safety and Health, 1974.
- B-7. Sulfur Dioxide, Chemical Safety Data Sheet, SD-52, Manufacturing Chemist Association, Washington, D.C., 1959.

APPENDIX C  
TEXAS STATE DEPARTMENT OF HIGHWAYS  
AND PUBLIC TRANSPORTATION  
TEXAS METHOD DESIGNATION TEX-206-F, PART II,  
MOTORIZED GYRATORY-SHEAR MOLDING  
PRESS OPERATING PROCEDURE

Note: It is suggested that laboratory personnel should contact the Materials and Tests Division for instructions concerning proper operation and maintenance of the motorized press prior to putting it into service.

1. Combine aggregates and prepare laboratory bituminous mixtures as described in Test Method Tex-205-F. Hot mix asphaltic concrete mixtures which contain asphalt cement are mixed and compacted into test specimens at a temperature of 250°F (121°C) plus or minus 5°F (3°C). Place hot mix cold laid mixtures and rock asphalt mixtures in an oven, cure to constant weight at a minimum temperature of 140°F (60°C) to remove moisture and/or hydrocarbon volatiles and mold at a temperature of 100°F (38°C) plus or minus 5°F (3°C). Curing to a "constant weight" may be accomplished by drying for a specific period of time that has proved by experiment to be adequate or drying to the point that by observation, based on experience the material is sufficiently dry for testing. Drying should be accompanied by frequent stirring. On some projects that specify Item 350, "Hot Mix Cold-Laid Asphaltic Concrete", the Engineer may choose to place the mixture "hot", omitting the addition of water and primer. In these instances it is suggested that the Hveem specimens be molded at 250°F  $\pm$  5°F (121°C  $\pm$  3°C) rather than the lower temperature specified previously for cold mix asphaltic concrete mixtures.

2. If the design mixture prepared in the laboratory or the mixture obtained from an asphaltic concrete plant contains aggregate larger than 7/8 inch (22.2mm), separate the large size aggregate from the sample by means of 7/8 inch sieve (or a 1-inch round opening screen). Use the towel to rub the material through the sieve and scrape off as much of the fines clinging to oversize particles as possible.

3. Pre-heat the mold and base plate in an oven to 140°F (60°C) to 200°F (93°C). For "premixes" heat mold to 100°F (38°C). Making certain that the platen is free to turn, connect the motorized gyratory-shear molding press to a 110v. ac outlet, and push the reset button, allowing the press to go through one set of gyrations.

4. Remove from oven and wipe the inside of the mold with a rag lightly moistened with kerosene or light lube oil. Insert the base plate into the mold with the large diameter up, and place a paper gasket over the base plate.

5. By means of the bent spoon and wide mouthed funnel, transfer the laboratory mixtures or a weighted quantity of plant-mixed material, heated to the proper molding temperature, into the mold. Place approximately 1/3 of the mixture into the mold and use the spoon to press the material down lightly. Use the small spatula to move

any large aggregate which might be touching the side of the mold. Add another 1/3 of the material, press down and move large aggregate away from the edge. Place the remainder of the sample into the mold, move the large particles a small distance away from the mold wall as before, and level the surface of the specimen while pressing the material down. Place a paper gasket on top of the mixture. Be careful to avoid loss of material and segregation of particles while placing the mixture into the mold. The vertical side of the specimen should be smooth to prevent damage to diaphragm of the stabilometer; since the top and bottom surface need not be exceptionally smooth, do not arbitrarily place fine material on the bottom or top of the sample.

6. Quickly place a small amount of light-weight oil in the center of the motorized press platen and a drop or two on the surface of the lower bearing. (This is the bearing that "cocks" the mold and creates the gyratory action).

7. Squirt a small ring of oil around the periphery of the mold on the top surface of the hardened steel ring. This ring of oil should be in the path that the upper bearing will follow during gyration. Do not use an excessive amount of oil in making this ring.

When molding a number of Hveem specimens, Steps 6 and 7 should be repeated every 10 to 15 specimens or as appears necessary when wearing surfaces become dry.

8. Steps 6 and 7 should be done quickly without delay. Then slide the hot mold and contents to the edge of the work table, and with gloved hand holding the base plate in place transport the mold to the platen of the press.

9. Slide the mold onto the platen and center it in molding position beneath the ram of the press.

10. Move the lever on the control valve to the forward or positive position, and pump the ram down into the center of the mold. Continue pumping until the low pressure gauge registers 50 psi (345 kPa).

11. Immediately pull the handle of the cam-lever down to the horizontal position, cocking the mold to the proper angle of gyration. Be certain that the cam-lever is pulled all the way down. The pump handle should be all the way up.

12. Push the reset button and then the start button. The mold will then gyrate three times and stop.

13. As soon as the mold stops gyrating, raise the cam-lever handle to the vertical position, leveling the mold, and immediately make one full stroke with the pump handle. No time should be wasted in the leveling of the mold and stroke of the pump. It should be two smooth, consecutive motions. (The full stroke of the pump is important, for it serves as an end point for the procedure. When one full stroke of the pump causes the low pressure gauge to surge to 150 psi (1035 kPa) or more, the gyrating portion of the molding procedure is complete).

14. Once again adjust the pressure to 50 psi (345 kPa) lower the cam-lever to the horizontal position, push the reset button, and then push the start button.

Experience has revealed that the smoothest operating procedure, and certainly the safest, is for the operator to keep the right hand on the pump handle at all times while operating the cam-lever, push buttons, and control valve with the left hand.

15. Continue Steps 12 through 14 until one smooth, but not violent, stroke of the pump handle will cause the low pressure gauge to indicate a pressure of 150 psi (1035 kPa) or more.

During molding when one stroke of the pump handle causes the gauge to come to rest between 50 (345) and 150 psi (1035 kPa), drop the pressure below 50 psi (345 kPa) by shifting the lever on the control valve all the way back and immediately returning it to the forward position. Then pump the pressure back up to 50 psi (345 kPa). Continue Steps 12 through 15 until the end point is reached.

16. When this end point of 150 psi (1035 kPa) is reached, bring the pump handle down slowly until the automatic gauge protector valve cuts the low pressure gauge out of the system. Now, at approximately one stroke per second, pump the pressure up to 2,500 psi (17,250 kPa) as measured on the high pressure gauge.

17. As soon as the gauge registers 2,500 psi (17,250 kPa), stop pumping with the right hand, and with the left hand very carefully release the pressure by slowly reversing the lever on the control valve to the backward position. Watch the large capacity gauge due to sudden, violent release of pressure.

18. Then pump the ram up and out of the mold.

19. Slide the mold out of the press, remembering to place a gloved hand beneath the mold to keep the base plate from falling out.



20. Allow the base plate to drop out of the mold onto the work table and remove the specimen from the mold with a converted arbor press or similar device.

21. Clean the mold on the inside with a kerosene rag before molding another specimen. It is to be emphasized that this motorized press must be kept clean. Attention must be given to the cleanliness of the press during and after molding.

22. When all the molding is completed, disconnect the press from the electric outlet, clean the unpainted parts of the press, the mold and base plate with a lightly moistened kerosene rag and coat with a thin coating of light-weight oil. This cleaning and oiling is an absolute necessity if the press is expected to continue functioning properly. Wipe the painted parts of the press with a clean dry rag.

#### Proposed Modifications

Step 1. The OGFC mixture will be prepared according to the method outline in Chapter III.

Step 2-9. No change in the described procedure.

Step 10. A pressure of 25 psi (173 kPa) should be used in place of the 50 psi (346 kPa) specified in the text.

Step 11. The angle of gyration should be set at 1°.

Step 12. No change in the described procedure.

Step 13. 100 psi (690 kPa) should be used to replace the 150 psi (1035 kPa) mentioned in this step.

Step 14. The pressure should be adjusted to 25 psi (173 kPa) instead of 50 psi (346 kPa).

Step 15. 25 psi (173 kPa) and 100 psi (690 kPa) should be used to replace 50 psi (346 kPa) and 150 psi (1035 kPa) respectively.

Step 16. 100 psi (690 kPa) is suggested to be the end point, and 1,500 psi (10,350 kPa) is recommended for leveling pressure.

Step 17. 2,500 psi (17,250 kPa) should be replaced by 1,500 psi (10,350 kPa).

Step 18-22. No change in the described procedure.

APPENDIX D  
AMERICAN ASSOCIATION OF STATE HIGHWAY  
AND TRANSPORTATION OFFICIALS  
TEST METHOD T 167  
COMPRESSIVE STRENGTH OF BITUMINOUS MIXTURES

## MOLDING AND CURING TEST SPECIMENS

Laboratory prepared mixtures shall be allowed to cool to molding temperature as quickly as possible after mixing. Mixtures from field projects shall be brought to molding temperature by careful, uniform heating immediately prior to molding. (Note) Molding temperatures shall be  $255 \pm 5^{\circ}\text{F}$  ( $124 \pm 3^{\circ}\text{C}$ ) for hot mixtures containing asphalt, and  $220 \pm 5^{\circ}\text{F}$  ( $104 \pm 3^{\circ}\text{C}$ ) for those containing road tar. Molding temperature shall be considered the temperature when the molding is first applied. As soon as the materials have been thoroughly mixed and have reached a suitable temperature within the specified range, approximately one half of the mixture shall be placed in the molding cylinder which, together with the top and bottom plunger, shall have been preheated in the water bath maintained at a temperature just under the boiling point. The molds and plungers should be wiped with a clean cloth that has a few drops of oil on it. With the bottom plunger in place and the molding cylinder supported temporarily on the two steel bars, the mixture shall be spaded vigorously twenty-five times with a heated spatula with fifteen of the blows being delivered around the inside of the mold to reduce honey comb, and the remaining ten at random over the mixture. The remaining half of the mixture shall then be quickly transferred to the molding cylinder and a similar spading action repeated. The spatula should penetrate the mixture as deeply as possible. A spatula having a slightly curved cross-section has been used to advantage by some laboratories. The top of the mixture should be slightly rounded or cone-shaped to aid in firm seating of the upper plunger. The mixture shall be compressed between the top and bottom plungers under an initial load of about 150 psi (1.0 MPa) to set the mixture against the sides of the mold. The support bars shall then be removed to permit full double-plunger action and the entire molding load of 20.7 MPa (3000 psi) shall be applied for 120 seconds. The specimen shall be removed from the mold with an ejection device that provides a smooth, uniform rate of travel for the ejection head.

After removal from the mold, specimens shall be oven cured 24 h at  $140^{\circ}\text{F}$  ( $60^{\circ}\text{C}$ ) and thereafter brought to test temperature,  $77^{\circ}\text{F}$  ( $25^{\circ}\text{C}$ ), by storing in the air bath at this temperature for not less than 5 h before testing. In case specimens are to be stored dry for more than 24 hours from completion of oven curing to compression testing, they shall be protected from exposure to the air by sealing them in closely fitting, airtight containers.

### Proposed Modifications

The molding temperature of the mixture should be 5°F (3°C) below the mixing temperature determined in the drainage test described in Appendix A. The molding pressure of 3000 psi (20,700 kPa) as described in the text should be replaced by a pressure of 1000 psi (6900 kPa). In the case where lightweight aggregate is used, a reduced pressure of 750 psi (6175 kPa) should be applied to the mixture to avoid any excessive crushing of the aggregates. The loading should be increased steadily, since any abruptly increase will result in a non-uniformly compacted mixture. The rest of the procedure is the same as that described in the original molding procedure.

APPENDIX E  
TEXAS STATE DEPARTMENT OF HIGHWAYS  
AND PUBLIC TRANSPORTATION  
TEXAS METHOD DESIGNATION TEX-126-E. PART II  
MOTORIZED GYRATORY-SHEAR MOLDING  
PRESS OPERATING PROCEDURE

1. When the mold has been loaded, then level the fines on top with any convenient hand tool and insert a thermometer in the mix. Record the temperature on the data sheet. The temperature should be  $260 \pm 20^{\circ}\text{F}$  ( $127 \pm 11^{\circ}\text{C}$ ). Mixes usually lose about  $100^{\circ}\text{F}$  ( $38^{\circ}\text{C}$ ) during mixing and loading, and it is desirable that the electric hot plate have some means of temperature control.

2. Remove the thermometer, place another 6 inch (152.4mm) filter paper on top of the mix, then a thin 6 inch (152.4mm) metal disk and remove the dolly with mold to the gyratory press.

3. Slide the mold, with base plate, onto the platen of the compactor. The platen must have a generous coat of good lubricant or the platen and base plate can be damaged. Center the mold, lower the compactor head on the material, and turn the lift cam down to give the mold its  $5^{\circ}$  lift angle.

4. Using the machine controls, place a load of 35 psi gauge<sup>1</sup> (20 psi specimen) with the loading ram on the specimen, and turn on the machine. Gyrate the specimen for two minutes at 35\* psi gauge loading.

5. At the end of 2 minutes, increase the load to 69\* psi gauge (40 psi specimen) with the loading ram on the specimen and turn on the machine. Gyrate the specimen for two minutes at 35\* psi gauge loading.

5. At the end of 2 minutes, increase the load to 69\* psi gauge (40 psi specimen) and continue gyrating 2 minutes.

6. The increase the load on the ram to 104\* psi gauge (60 psi specimen), and continue gyration until the gauge needle will stand steady for 5 revolutions of the platen. This means that there has been no appreciable shortening or densification in the five revolutions. Turn the gyratory press off.

7. Release the pressure from the top of the specimen slightly, and using the handle provided, return the cam lift to its original position, and reduce the angle of lift to zero.

---

\* To nearest pound per square inch on gauge

<sup>1</sup> Gauge readings are given in English units in this Appendix to correspond with units on instrument.

8. Place 35 psi gauge pressure on the specimen, and turn the machine on for a few revolutions. This tends to square-up the specimen. Turn the machine off.

9. Wipe off any oil on the platen, and place 865 psi gauge pressure on the specimen. This is 500 psi (3450 kPa) on the cross-section of the specimen.

10. Place the pre-set measuring stand in position to measure the height of specimen. Hold the load on the specimen until the rate of consolidation is 0.005 inches (.127mm) or less in 5 minutes.

11. Observe the dial reading, and record the net height of specimen only, making allowance for thickness of the metal disks.

12. Remove the measuring device, and then the load on the specimen. Raise the ram out of the mold, and remove the mold from the machine platen to the dolly.

13. Slide the mold with base plate on the platen of the ejection press and eject the specimen up and out of the mold.

#### Proposed Modifications

Material for the base can be any standard laboratory aggregate with a grading in compliance with "Type D" gradation in the Texas Highway Department specification. The asphalt cement can either be AC-10 or AC-20. The base should have a height of 5 to 7 inches (127 to 177.8mm). The original procedure described in this appendix should be used, without any modifications, to mold the base.

But for the OGFC materials, some minor changes have to be made to the standard procedure. They are as follows:

Step 1. The molding temperature should be 5°F (3°C) below the mixing temperature determined in the drainage test. The temperature of the base should be brought up to about 100°F (38°C) before OGFC material is placed on the top of it in the mold.

Step 2. No change in the described procedure.

Step 3. 1° angle of gyration should be used to replace the 5° lift angle.

Step 4-10. No change in the described procedure.

Step 11. This step may be discarded.

Step 12-13. No change in the described procedure.

## Permeability Test Apparatus

The permeability apparatus developed for this study (see Figure F-1) was similar to the one used by the U. S. Army Waterways Experiment Station. The permeameter assembly basically consisted of a plastic standpipe with a 6.98 cm (2.75 in) inside diameter and a 17.78 cm (7 in.) diameter metal standpipe base (see Figure F-2).

For testing purposes, a support assembly was constructed to hold the specimen and support the permeameter assembly. A "silicone-sponge rubber" gasket, unlike the gasket used by the Waterways Experiment Station, was developed and used as a seal between the standpipe base and the pavement surface (see discussion in next section). Compression springs were used to apply a load to the permeameter assembly in order to create a better seal between surfaces and eliminate surface flow. A schematic of the permeability test set-up procedure is shown in Figure F-3.

### "Silicone-Sponge Rubber" Gasket

The intention was to use a gasket which would prohibit all surface flow, and this required the use of a material which could conform to the macrotexture of the pavement surface. Some of the different types of materials experimented with were "bowl wax", "permagum", and "sponge rubber". The bowl wax and permagum gaskets did seal off the surface, but they also tended to flow into and seal portions of the internal drainage system. The sponge rubber gasket appeared to conform well to the surface of the specimen, however leaks developed between the gasket and the standpipe base. It should also be noted that poor repeatability was evidenced for these three types of gaskets.

The gasket finally developed for use in this study was a combination of two materials (1) a 0.64 cm (0.25 in.) layer of silicone rubber (General Electric RTV 11) and (2) a 0.64 cm (0.25 in.) layer of silicone sponge rubber (Connecticut Hard Rubber Co.)\*. The silicone sponge rubber layer was placed in contact with the surface of the specimen because it was capable of conforming to the macrotexture. The silicone rubber layer was comparatively stiff, yet flexible enough to provide a good seal between it and the metal standpipe base.

A mold was constructed according to the dimensions of the desired gasket [15.24 cm (6 in.) outside diameter and a 6.98 cm (2.75 in.) inside diameter]. The silicone rubber was poured into the mold to a depth of 0.64 cm (0.25 in.). Next, the silicone sponge rubber, cut to the dimensions shown above, was placed on top of the silicone rubber layer. The curing process causes an adhesion between the two surfaces.



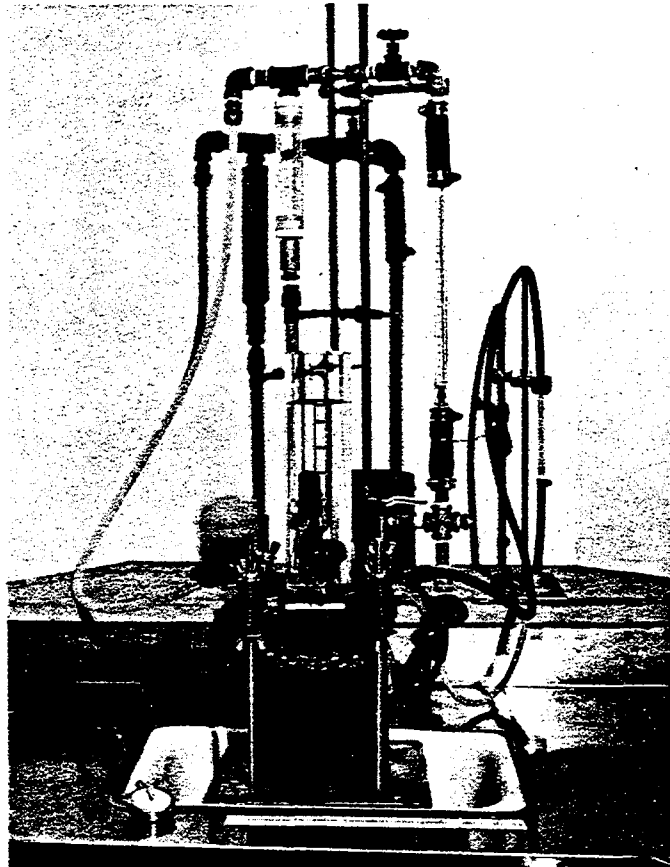


Figure F-1. Permeability Test Apparatus

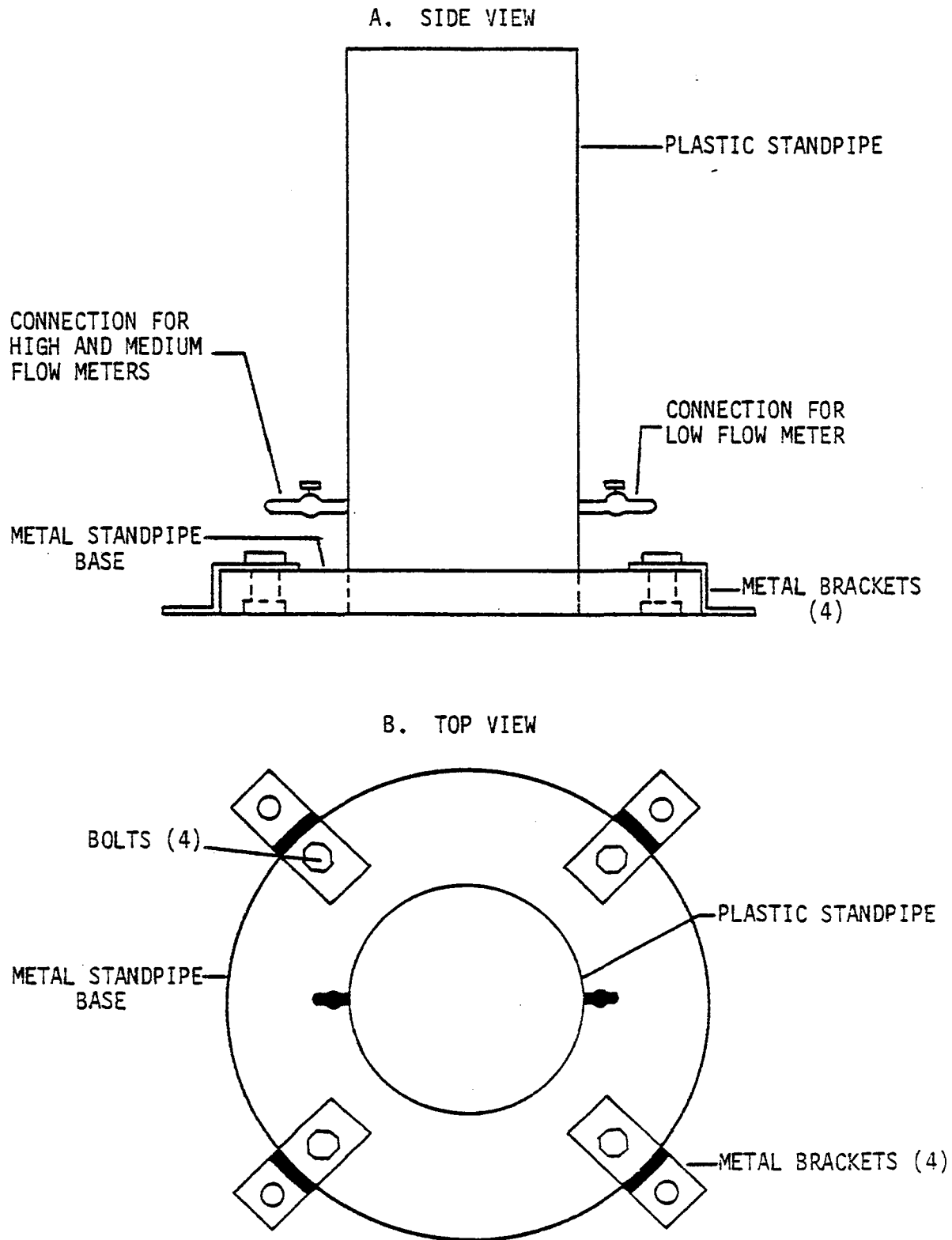


Figure F-2. Schematic of Permeameter Assembly

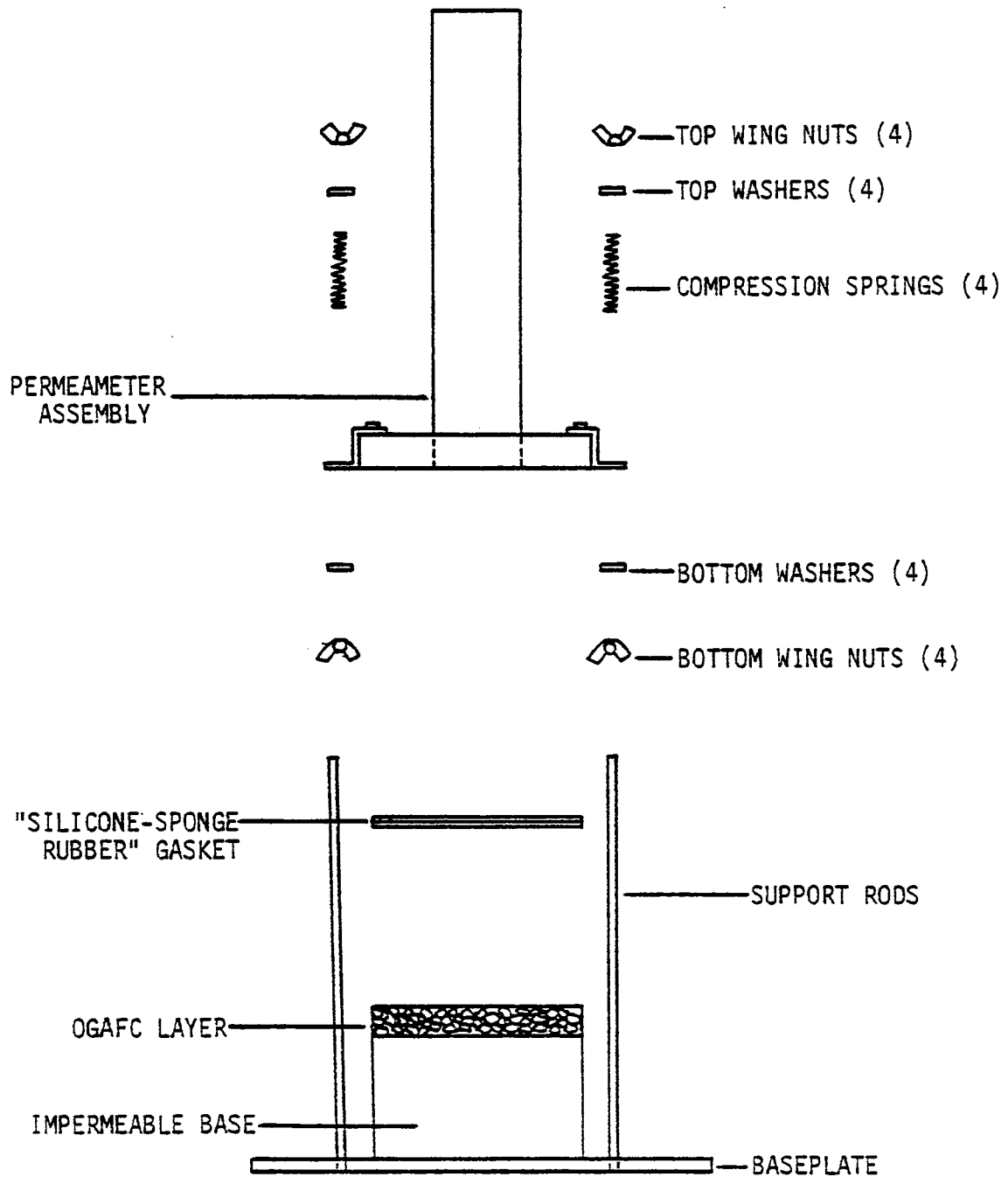


Figure F-3. Schematic of Permeability Test Set-Up Procedure

Note that it may be necessary to allow the silicone rubber to cure for a while before applying the silicone sponge rubber.

### Permeability Test Procedures

The permeability apparatus developed for this study was used to run variable head tests. The recommended test procedure for the variable head test is described in Table F-1.

### Formulas for Calculating Permeability

The permeability measurements determined from the variable head tests were recorded as "time to fall". From these values the coefficients of permeability  $k$  (cm/sec) were computed from the following equations.

$$\text{Variable Head Test: } k_v = 2.3 \frac{aL}{A(t)} \log_{10} \frac{h_0}{h_1} \quad \text{Equation F-1}$$

in which:

$a$  = cross-sectional area of standpipe,  $\text{cm}^2$

$L$  = length of flow path, cm

$A$  = area perpendicular to flow path,  $\text{cm}^2$

$t$  = time for water level to fall from  $h_0$  to  $h_1$ , sec

$h_0, h_1$  = the heads between which the permeability is determined, cm

In order to apply these equations, several assumptions had to be made in regard to the flow path. First, it was assumed that the water initially flowed vertically downward and then, radially out of the sample. It was also assumed that a single flow path through the specimen has both a vertical and a horizontal component. These flow path assumptions are shown in Figure F-9. Since the ratio of length of flow path to perpendicular area of flow path ( $L/A$ ) exists in both equations, a formula was derived by which to calculate this ratio based on the thickness of the OGAF layer.

The average length of the flow path was determined based on the following equation:

Table F-1. Tests Procedures for Laboratory Permeability Measurements  
Variable Head Test.

Procedure	Remarks
1. Record specimen number, give a brief description of OGAFc layer (i.e. surface irregularities, damaged surface, macrotexture, density, etc.), and determine the average thickness of the OGAFc layer.	This information will serve as a quick reference and may provide possible reasons for errors in the event the results appear questionable.
2. Place a 15.24 cm (6 in.) diameter core on the base plate of the support assembly (Figure F-3).	
3. Place "silicone-sponge rubber" gasket on surface of OGAFc layer making certain it is centered on surface (Figure F-4).	Several gaskets should be on hand and used on a rotation basis due to the temporary deformation of the sponge rubber.
4. Lower the permeameter assembly onto the specimen making certain it is aligned with the gasket (Figure F-5).	
5. Slide the 4 compression springs and washers onto the support rods. (Figure F-6).	These springs should be pre-calibrated to determine their compressed length under a 445 N (100 lbs.).
6. Apply a 445 N (100 lbs.) at each spring by tightening the upper wing nuts until the spring is compressed to the pre-determined length. (Figure F-7).	Loads in excess of this may cause damage to samples.
7. Screw the lower wing nuts up until they just come into contact with the metal brackets.	
8. Make certain the apparatus is level.	Set level on surface of metal standpipe base and adjust the complete apparatus until level.

Table F-1. (Cont'd).

Procedure	Remarks
9. Open valve and allow water to flow into standpipe and through specimen for approximately 2-3 minutes before beginning test.	This was done in order to wet exposed surfaces and flow paths. Also check to make sure there is a good surface seal.
10. Locate and measure the heads $h_0$ and $h_1$ .	The values used for this study were $h_0 = 10$ in. (25.40 cm) and $h_1 = 3.41$ in (8.65 cm).
11. Fill the permeameter tube with water to an elevation slightly above $h_0$ .	
12. Close main valve; start the stopwatch when the water level is at $h_0$ and record the elapsed time when water level reaches $h_1$ . Make three determinations and compute the average (Figure F-8).	If the elapsed time is greater than 10 minutes, then two determinations will be sufficient.
13. Measure temperature of the water and record.	For this study, the temperature of the water was maintained at approximately 77°F (25°C) (fluctuations were insignificant as far as changes in viscosity were concerned).

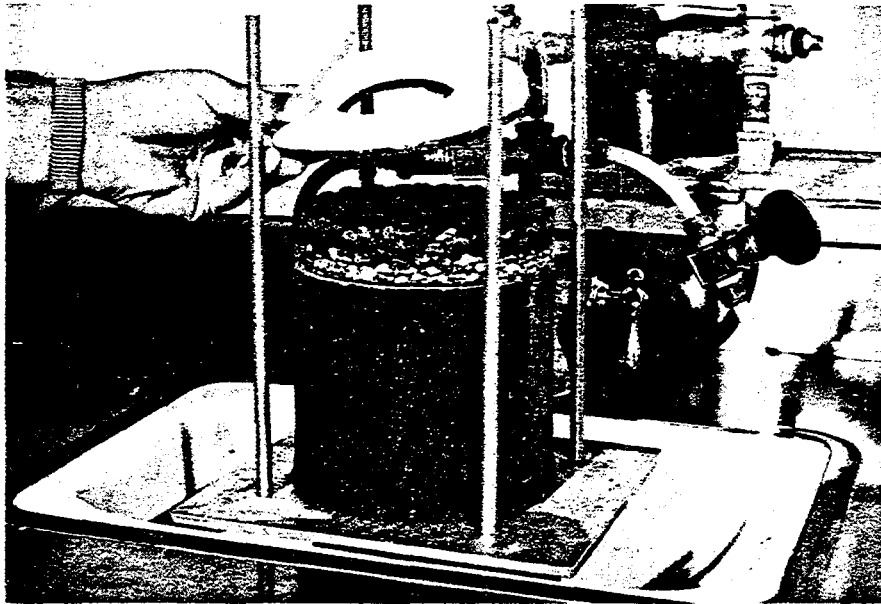


Figure F-4. Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

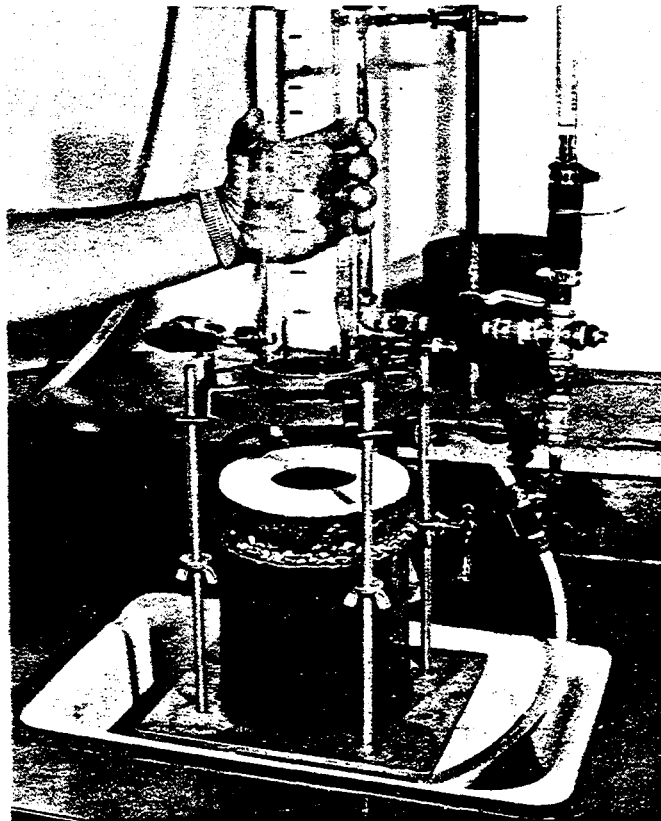


Figure F-5. Gasket in Place and Permeameter Assembly Being Lowered Into Position

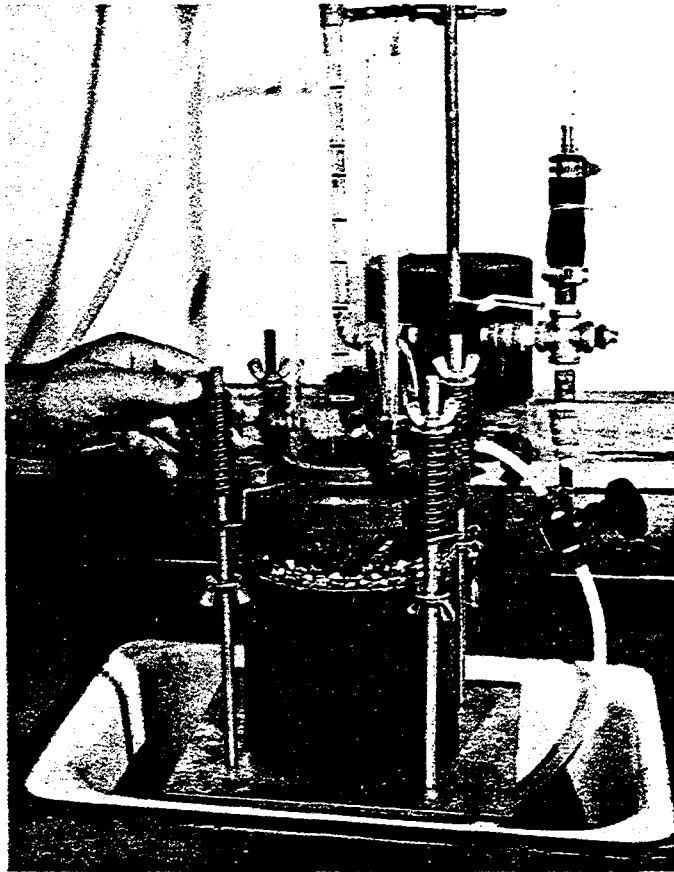


Figure F-6. Compression Springs Being Positioned on Support Rods



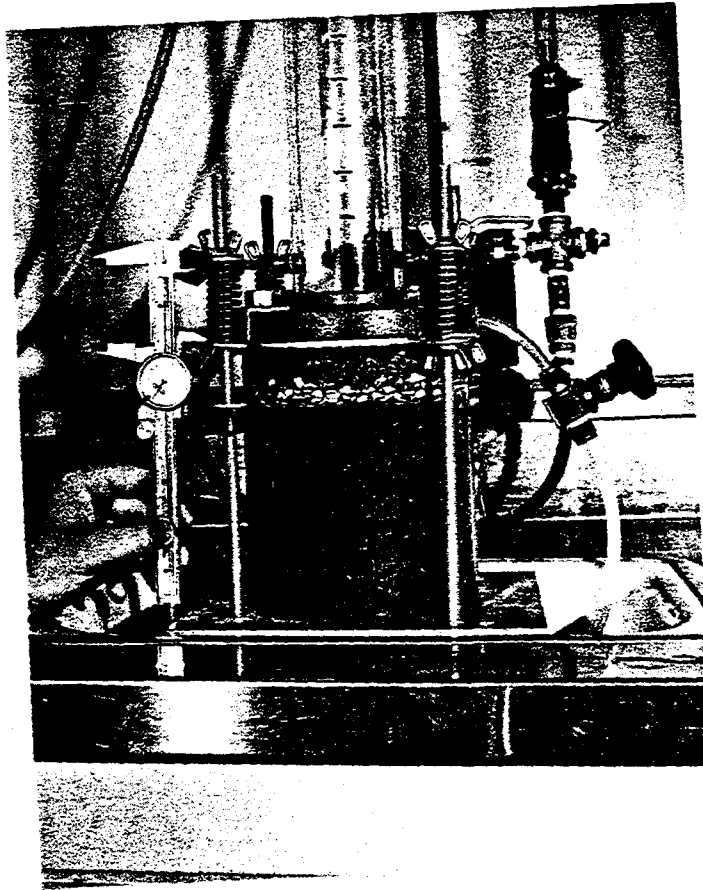


Figure F-7. Compression Springs in Place and Load Being Applied

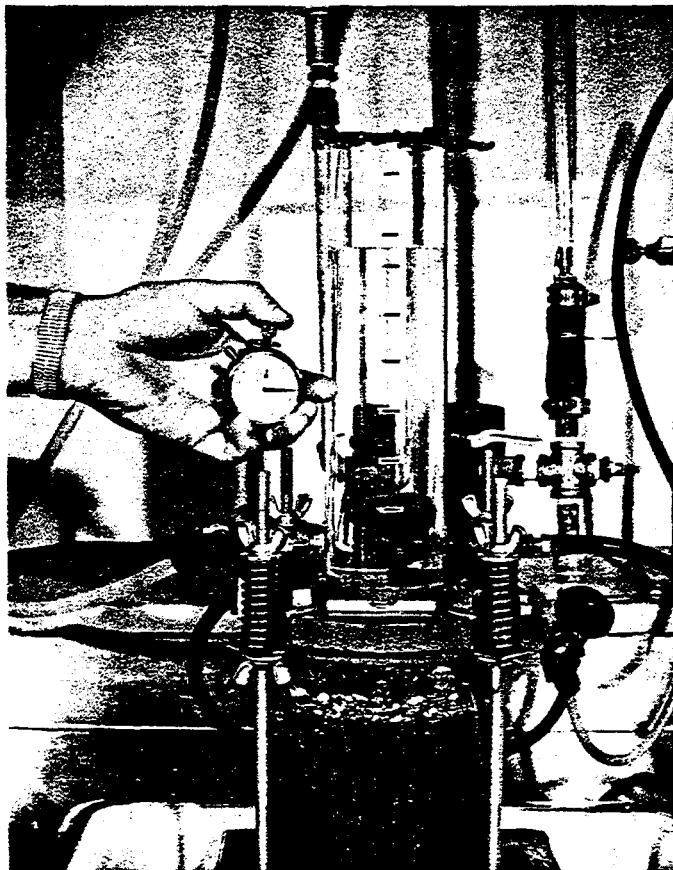


Figure F-8. Variable Head Permeability Test in Progress

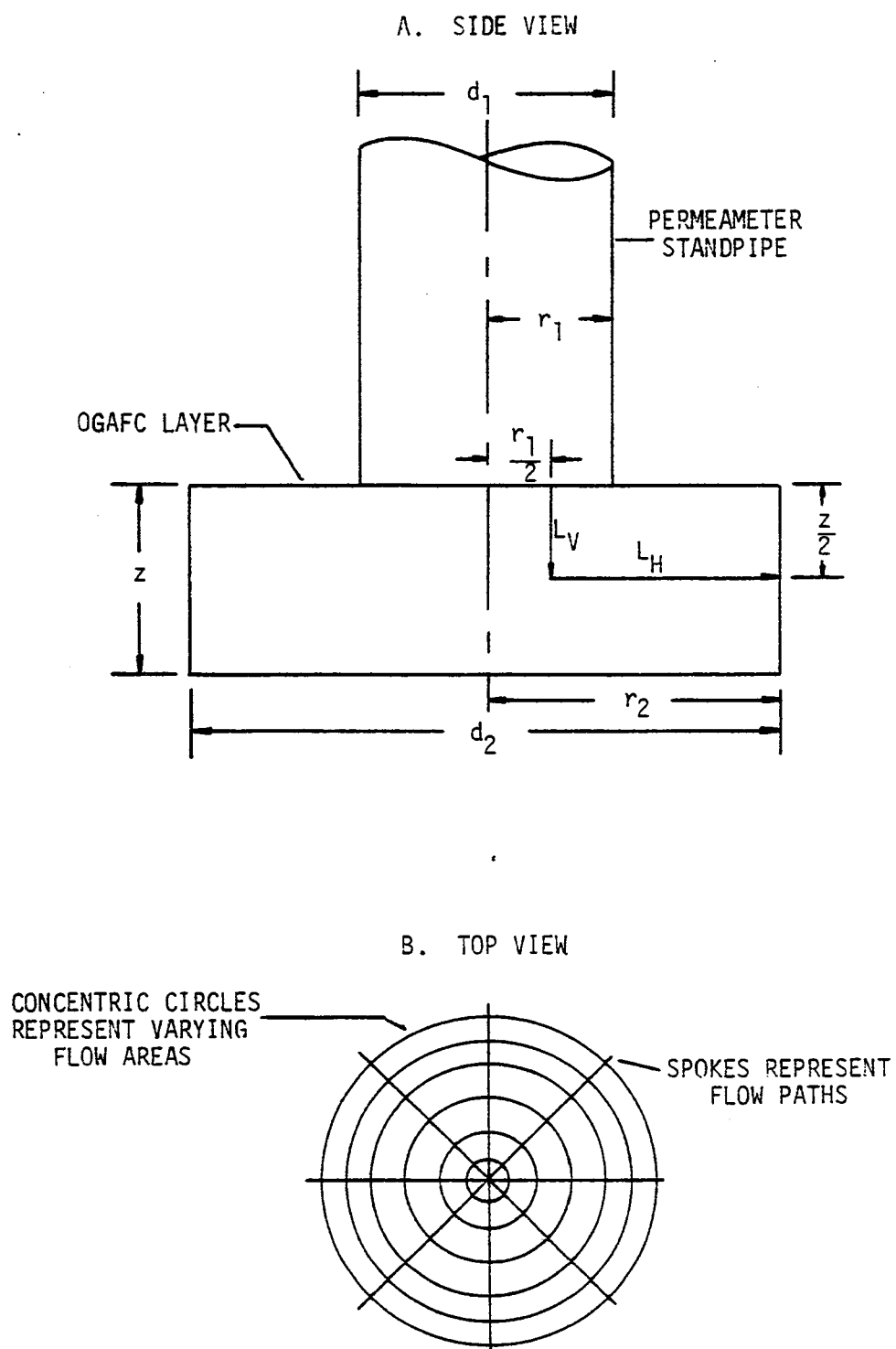


Figure F-9. Schematics Showing Flow Assumptions

$$L_{AVG} = L_V + L_H$$

Equation F-3

where:

$L_{AVG}$  = average length of the flow path

$L_V$  = average length of the vertical component

$L_H$  = average length of the horizontal component

$L_V$  was assumed to be equal to half the thickness ( $z$ ) of the OGAF layer ( $L_V = z/2$ ) and  $L_H$  was assumed to be equal to the radius [ $r_2 = 3$  in. (7.62 cm)] of the specimen minus half the radius [ $r_1 = 1.37$  in. (3.49 cm)] of the standpipe ( $L_H = r_2 - r_1/2$ ). By setting  $r_1/2 = r_0$  and substituting these values into Equation E-3, it gives the following expression:

$$L_{AVG} = z/2 + (r_2 - r_0)$$

Equation F-4

The flow area ( $A_V$ ) perpendicular to  $L_V$  is equal to the area of the standpipe.

$$A_V = r_1^2 = (3.49 \text{ cm})^2$$

$$A_V = 38.3 \text{ cm}^2 \text{ or } (5.94 \text{ in.}^2)$$

The flow area ( $A_H$ ) perpendicular to the horizontal flow path varies as the distance ( $r_1$ ) varies from  $r_1/2$  or  $r_2$ .

The general equation for determining  $A_H$  is:

$$A_{H_{r_i}} = 2 r_i z$$

Equation F-5

Figure F-10 shows how the flow area varies with the length of the flow path and Figure F-11 shows how the ratio  $L/A$  varies with the length of the flow path for a 1.00 in. (2.54 cm) thick OGAF layer.

Since the ratio  $L/A$  increases as the length of the flow path increases it was necessary to determine an average value for  $L/A$  to be used in calculating permeability. In order to do this it was

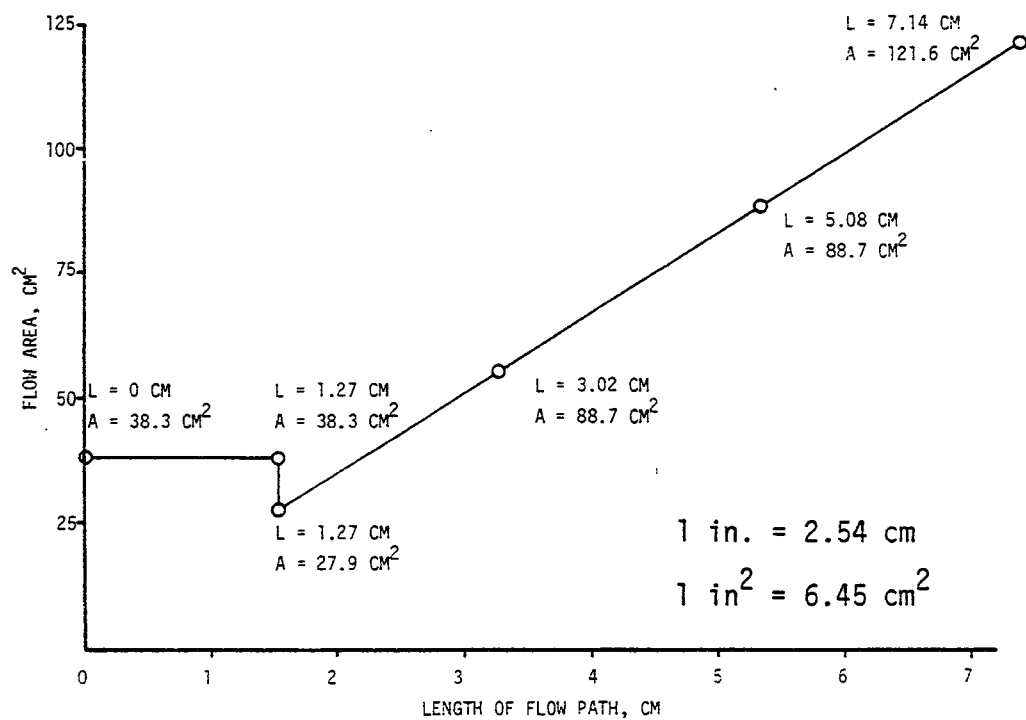


Figure F-10. Flow Area vs. Length of Flow Path for 6 in. (15.24 cm) Diameter, 1 in. (2.54 cm) Thick Specimen

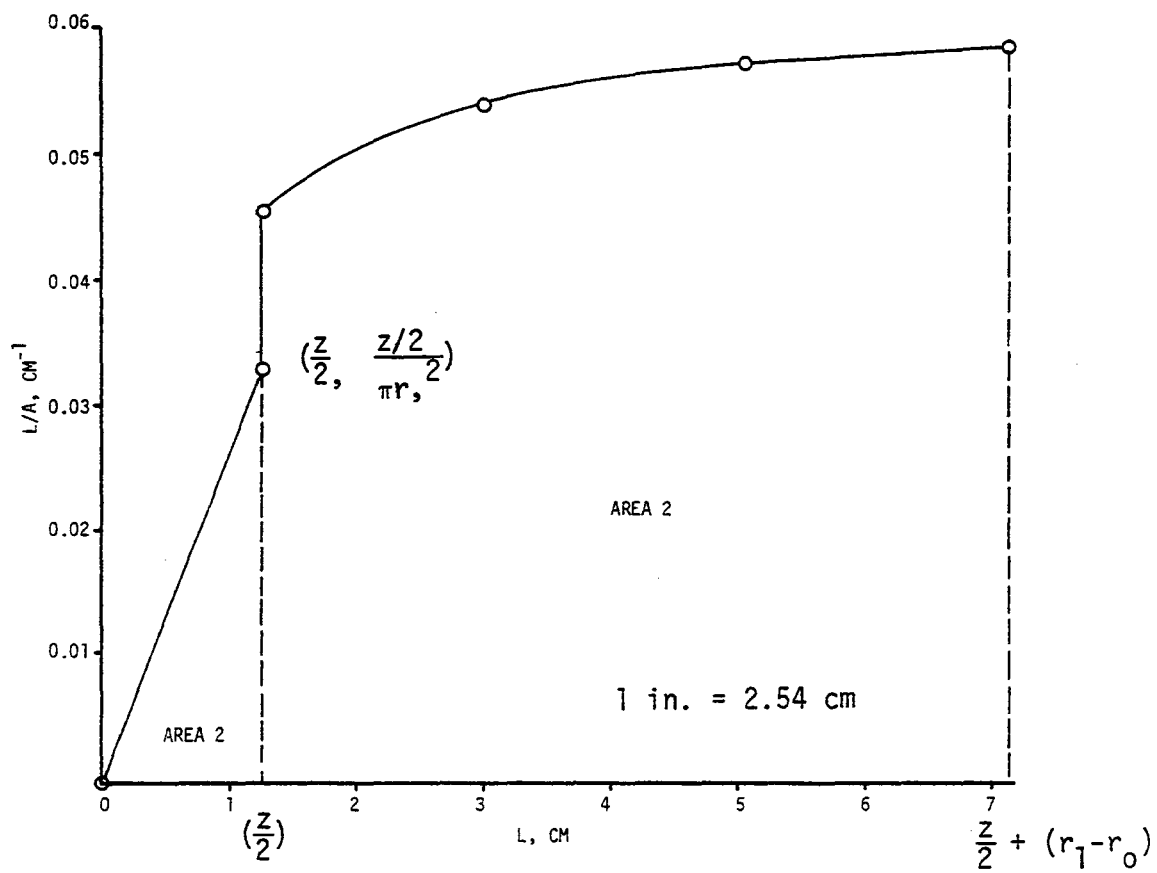


Figure F-11.  $L/A$  vs.  $L$  a 6 in. (15.24 cm), 1 in. (2.54 cm) Thick OGAFc Specimen

necessary to determine the area under the curve in Figure F-11. The equations given below are in general form and be used for any thickness.

$$\begin{aligned} \text{Area 1} &= \frac{1}{2}(z/2)\left(\frac{z/2}{r_1}\right)^2 \\ \text{Area 1} &= \frac{1}{8} (z/r_1)^2 \end{aligned} \quad \text{Equation F-5}$$

$$L = z/2 + (r_2 - r_o)$$

$$\text{Area 2} = \int L/A \, dL \quad \text{Equation F-6}$$

$$L = z/2$$

Equation F-3 can be written in the general form:

$$L = z/2 + (r_i - r_o)$$

and by rearranging terms:

$$r_i = L + r_o - z/2$$

Substituting this into Equation F-4 gives:

$$A_{H_{r_i}} = 2 (L + r_o - z/2)z$$

Therefore, Equation F-6 reduces to:

$$\begin{aligned} \text{Area 2} &= \int \frac{L}{L + r_o - z/2} \, dL \\ L &= z/2 \end{aligned}$$

which is of the form:

$$\frac{x}{(ab + b)} \, dx, \text{ where } a = 1, b = r_o - t/2, \text{ and } x = L.$$

Integrating this expression gives the following result:

$$\text{Area 2} = 1/2 z [L - (r_0 - z/2) \ln(L + r_0 - z/2)] \quad \begin{matrix} L = z/2 + r_2 - r_0 \\ L = z/2 \end{matrix}$$

$$\text{Area 2} = 1/2 z [(r_2 - r_0) - (r_0 - z/2) \ln r_0/r_2] \text{ Equation 7}$$

Equations F-5 and F-7 can then be used to compute an average value for L/A:

$$\begin{aligned} (L/A)_{\text{AVG}} &= \frac{\text{Area 1} + \text{Area 2}}{L_V + L_H} \\ &= \frac{1/8 (z/r_1)^2 + 1/2 z [(r_2 - r_0) + (r_0 - z/2) \ln r_0/r_2]}{z/2 + (r_2 - r_0)} \end{aligned}$$

Substituting  $r_1 = 3.48$  cm,  $r_0 = 1.75$  cm, and  $r_2 = 7.62$  cm, this equation reduces to the following:

$$(L/A)_{\text{AVG}} = \frac{0.00326z^2 + 0.525/z + 0.117}{(z/2 + 5.87)} \text{ cm}^{-1}$$

This was the equation used for computing the L/A term in the variable head permeability formula.



APPENDIX G  
A TENTATIVE PROCEDURE FOR DETERMINING  
RESISTANCE OF OPEN-GRADED FRICTION COURSES (OGFC'S)  
TO RAPIDLY REPEATED CYCLES OF FREEZING AND THAWING IN WATER  
AND  
ALTERNATE METHOD FOR TEST SAMPLES  
REF. AASHTO T135

## Freezing and Thawing Apparatus

1. A suitable chamber shall be furnished with the appropriate cooling and heating equipment which operates automatically to produce continuous repeated cycles within the pre-set temperature limits. In the event that automatic equipment is not available, either the equipment is manually operated on a continuously 24-hour-a-day basis or the specimen stored in a frozen condition when the equipment is not in operation.

2. When the equipment is in operation, the temperature throughout the chamber, as well as the specimens should be uniform and within 6°F (3.3°C) at any given time. However, the transition period between freezing and thawing and vice versa, can be exempted from the above requirement.

## Specimen Holding Apparatus

The dimension of the pan, in which two 6-in (15.24 cm) diameter specimens are placed is 18 in. (45.72 cm) long, 8 in. (20.32 cm) wide, and 4 in (10.16 cm) deep. Within each pan, the specimen shall be placed on a pedestal and inside a confining collar which is made of copper. The inner diameter of the collar should be approximately 1/16 (0.16 cm) to 3/16 in (0.48 cm) larger than the diameter of the OGFC layer and the thickness of the collar shall be about 1/16 in (0.16 cm). The depth of the collars should be slightly larger than the height of the specimen being tested. A schematic diagram of the apparatus is shown in Figure G-1.

## Temperature Measuring Equipment

The temperature-measuring equipment shall consist of thermometers, resistance thermometers, or thermocouples, capable of measuring the temperature at various points within the specimen chamber and adjacent to control specimens to within 2°F (1.1°C).

## Freezing and Thawing Cycle

Freezing and thawing tests shall be started at the beginning of the thawing phase of the cycle. The upper and lower temperature bounds are 40°F (4.4°C) and 0°F (-17.8°C) respectively. A complete freezing and thawing cycle shall consist of raising the temperature from 0°F (-17.8°C) to 40°F (4.4°C) in a period of not more than one hour and lowering the temperature from 40°F (4.4°C) to 0°F (-17.8°C) in a

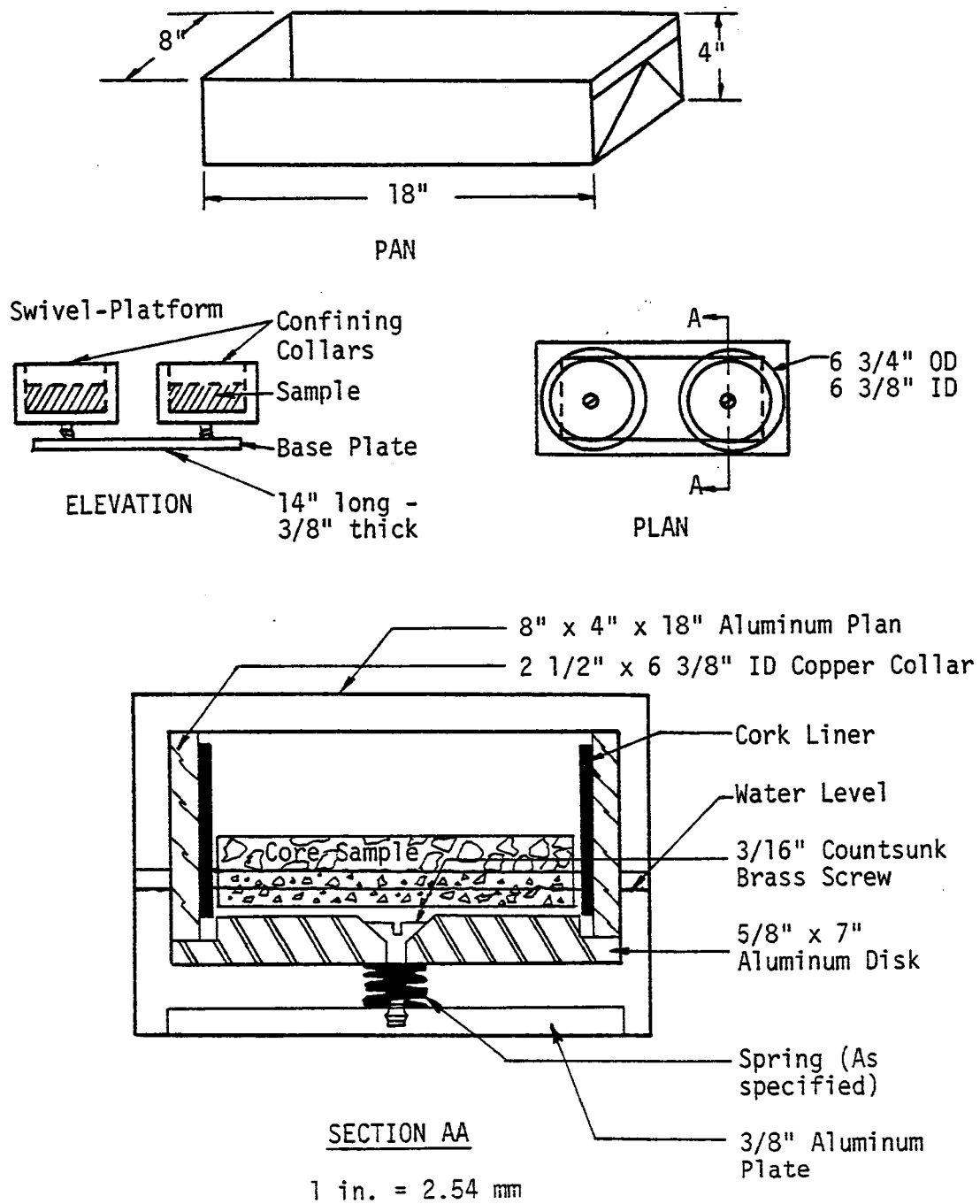


Figure G-1. Specimen Holding Apparatus

period of not more than one hour and lowering the temperature from 40°F (4.4°C) to 0°F (-17.8°C) in a period of not more than three hours. At the end of the cooling and heating phase in the cycle, the temperature of every specimen should be within  $\pm 3^\circ\text{F}$  (1.7°C) of their respective temperature requirements. No specimen shall, at anytime, reach a temperature higher than 43°F (6.1°C) nor lower than -3°F (-19.4°C). The time required for the temperature of any single specimen to be reduced from 37 to 3°F (2.8 to -16.1°C) shall be not less than half the length of the cooling period, and the time required for the temperature of any single specimen to be raised from 3 to 37°F (-16.1 to 2.8°C) shall be not less than half the length of the heating period.

The period of transition from freezing to the thawing phase of the cycle or vice versa shall not exceed 10 minutes.

### Test Specimen

The test specimens shall be obtained by sawing the OGFC layer off the top of the core at a point about 1/2-in (1.27cm) below the bottom of the 6-in (15.24cm) diameter OGFC. The diameter of the OGFC should be within  $6 \pm .20$  in (15.24 cm  $\pm$  .51cm) and the total height of the specimen (i.e., OGFC layer plus the thin layer of asphaltic concrete left to act as a base for the layer to be tested) should be in the range of 1.25 to 1.75 in (3.18 to 4.45 cm). Specimens shall be tested in triplicates.

The test specimens must be stored in a cool, (40 to 70°F (4.4 to 21.1°C)), dry place at all times prior to the test.

### Procedure

1. An initial visual inspection of each specimen shall be performed, noting any defects or obvious damages resulting from improper compaction or handling. Information about the mix design should be recorded.

2. Obtain and assemble the parts of the specimen-holding apparatus. Each apparatus shall hold 2 specimens.

3. A base plate, with its two swivel-platforms and two specimens on the top of it, shall be placed in each pan. Each pan shall then be filled with water to a depth of approximately 2.5 in (6.35 cm) and placed in the freezing and thawing cabinet. The height of each swivel-platform shall be adjusted individually so as to place the water surface at slightly below the mid-point of the OGFC layer. The con-

fining collars shall then be placed around the specimens. After placing the collars, one shall check to insure the surface is level and that the water surface is at about the mid-point of the OGFC layer. Any necessary correction shall be made before proceeding to Step 5. In the event that swivel-platforms are not available, every effort should be made to insure that the condition described above is satisfied.

4. A pan filled with wet sand shall be placed at the center position in the freezing and thawing cabinet. The cabinet's temperature sensors shall then be embedded in the sand at about the center of volume of the pan. The specimen-holding pans shall be positioned symmetrically on either side of the sensor pan. One 60-A (500 W) heating element shall be placed on each side of the individual pans. After the test has started, temperature of each specimen shall be checked to insure complete freezing and thawing, and any modifications on the arrangement shall be made if necessary.

The equalizing of the sensor's pan and specimen pans can be accomplished by adjusting the amount of sand and/or water.

5. The test shall be started at the beginning of the thawing phase of the cycle. At the end of 25, 50 and 75 cycles of freezing and thawing, the specimens shall be removed from the apparatus examined closely and returned to the apparatus. A detail description of the specimen condition shall be recorded. After each specimen has been subjected to 100 cycles, a final description shall be recorded. After the final visual inspection has been completed, other tests, such as wire-brush test, may be performed on the test specimens.

#### Report of the Data

The visual observations shall be reduced to numerical values by using two-part code numbers being separated by a dash. The first number tells the type of damage as follows:

- 0 = No damage visible
- 1 = Used only for beginning observation-aggregate not coated completely, damage not due to wear.
- 2 = Used where cracks or breaks were in the aggregate, and aggregate loss is visible, and
- 3 = Used where aggregate loss in part of total has occurred.

The second part of code-number is the percent of the aggregate affected. Example: 2-3 is fractured aggregate with 3 percent affected.

## ALTERNATE METHOD FOR TEST SAMPLES

REF. AASHO T135

### Scope

1. These specifications cover test on 6-in (15.24cm) diameter asphalt core samples which have been subjected to freeze and thaw as a means to aid in determining damage.

### Apparatus

2. (a) Scales - with 1.1 lb capacity to 0.000022 lb (500 g capacity  $\pm .01g$ )

(b) Wire Scratch Brush - A wire scratch brush made of 2 by 1/16 in. (5.08 by .16cm) flat No. 26 gage wire bristles assembled in 50 groups of 10 bristles each and mounted to form 5 longitudinal rows and 10 transverse rows of bristles on a 7 1/2 by 2 1/2 in (19.05 by 6.35cm) hardwood block.

### Procedure

After removing samples from freeze thaw and when all observations have been made, weigh and record weights of each core to nearest .1g. Hold or secure sample by any convenient means that will not in itself cause damage to the sample. Then with "Scratch Brush" apply 3 overlapping strokes to surface of core being tested. The force on the brush is to be 3 lbs. (13.35N). This can be achieved by adding the required weight to the brush to equal 3 lbs. (13.35N). The brush then only need be pushed or pulled across the core with no downward force. Record final weight and subtract from original weight for test value.

APPENDIX H  
A PROPOSED MIX-DESIGN PROCEDURE AND LABORATORY  
EVALUATION TEST FOR SULFUR-EXTENDED ASPHALT  
OPEN-GRADED FRICTION COURSES

## Material Requirements

1.1. It is recommended that relatively pure carbonate aggregates or any aggregates known to polish be excluded from the coarse-aggregate fraction (material retained on the No. 8 (2.38 mm) sieve). In addition, the coarse-aggregate fraction should have at least 75 percent (by weight) of particles with at least two fractured faces and 90 percent with one or more fractured faces. The abrasion loss (AASHTO T96) should not exceed 40 percent.

1.2. Recommended gradation for Sulfur-Extended Asphalt Open-Graded Friction Course.

<u>Sieve Size<sup>a</sup></u>	<u>Percent Passing<sup>b</sup></u>
1/2 in. (12.7 mm)	100
3/8 in. (9.52 mm)	95-100
No. 4 (4.76 mm)	30-50
No. 8 (2.38 mm)	5-15
No. 200 (0.074 mm)	2-5

---

<sup>a</sup>U. S. sieve series

<sup>b</sup>By volume. (This is the same as by weight unless specific gravities of aggregates being combined are different).

If the coarse and fine aggregate fractions are separated using a No. 10 (2.00 mm) sieve, the recommended grading should be changed as follows:

<u>Sieve Size</u>	<u>Percent Passing</u>
1/2 in. (12.7 mm)	100
3/8 in. (9.52 mm)	90-100
No. 4 (4.76 mm)	40-60
No. 10 (2.00 mm)	8-12
No. 200 (0.074 mm)	2-5



1.3. The recommended grade of asphalt cement is AC-10, AC-20, or AC-40, AASHTO M 226. For AC-10, AC-20 and AC-40, the M 226 Table 2 requirements should apply where such asphalt is available.

1.4. It is recommended that any commercially available elemental sulfur may be used.

#### Preliminary Data

2.1. Test coarse and fine aggregates as received for the project for gradation unless otherwise provided. If mineral filler is submitted as a separate item, it should also be tested for specification compliance. Analyze gradation results to determine if proportions of aggregates proposed by the contractor will meet the job-mix formula and the specification limits of Step 1.2.

2.2. Determine bulk specific gravity and water absorption capacity for the coarse and fine aggregate fractions (retained and passing the No. 8 (2.38mm) sieve) for each type of material submitted. Additional specific gravity tests are not warranted when the only distinction between aggregates is size of grading. Using the information verified in Step 2.1, mathematically compute the bulk specific gravity for the coarse and fine aggregate fractions (retained and passing the No. 8 (2.38 mm) sieve) for the proposed job-mix gradation.

2.3. Test the asphalt cement to be used for specification compliance (AASHTO M 226), viscosity-temperature data, and specific gravity at 77°F (25°C).

#### Asphalt Content

3.1. Determine the surface capacity of the aggregate fraction that is retained on a No. 4 (4.76 mm) sieve in accordance with the following procedure.

Note: For highly absorptive aggregates, use the procedure described in Step 3.3.

$K_s$  is determined from the percent of SAE No. 10 oil retained, which represents the total effect of superficial area, the aggregate's absorptive properties and surface roughness.

3.1.1. Quarter out 0.23 lb. (105 g) representative of the material passing the 3/8 in (9.52 mm) sieve and retained on the No. 4 (4.76 mm) sieve.

3.1.2. Dry sample on hot plate or in  $230 \pm 9^{\circ}\text{F}$  ( $110 \pm 5^{\circ}\text{C}$ ) weight and allow to cool.

3.1.3. Weigh out 0.22 lbs. (100.0 g) and place in a metal funnel (top diam. 3 1/2 in. (88.9 mm), height 4 1/2 in. (114.3 mm), orifice 1/2 in. (12.7 mm), with a piece of No. 10 (2 mm) sieve soldered to the bottom of the opening).

3.1.4. Completely immerse specimen in SAE No. 10 lubricating oil for 5 min.

3.1.5. Drain for 2 min.

3.1.6. Place funnel into tarred pan; cool, and reweigh sample to nearest  $22 \times 10^{-6}$  lb (0.1g). Subtract original weight and record difference as percent oil retained (based on 0.22 lb. (100 g) of dry aggregate).

3.1.8. Use chart shown in Figure A-1 for determination of  $K_c$ .

- (a) If specific gravity for the fraction is greater than 2.70 or less than 2.60, apply correction to oil retained, using formula at bottom of chart in Figure A-1 (Appendix A).
- (b) Start at the bottom of chart in Figure A-1 with the corrected percent of oil retained; follow straight-edge vertically upward to intersection with the diagonal line; hold point, and follow the straight-edge horizontally to the left. The value obtained is the surface constant for the retained fraction and is known as  $K_c$ .

3.2. Determine the required asphalt content, which is based on weight of aggregate, from the following relationship (2):

$$\text{Percent asphalt} = (2.0 K_c + 4.0) \times \frac{2.65}{(\text{SG})_{\text{ca}}}$$

Where  $K_c$  = surface constant

(SG)<sub>ca</sub> = apparent specific gravity of coarse aggregate (3/8 in. (9.52 mm) to No. 4 (4.76 mm)).

3.3. For highly absorptive aggregates, use the following procedure for determining  $K_c$  and asphalt content.

3.3.1. Follow the recommended design procedure from Step 3.1. through Step 3.1.3.

3.3.2. Follow the instructions in Step 3.1.4., except immerse the specimen for 30 min.

3.3.3. Follow the recommended procedure from Step 3.1.5 through Step 3.1.7.

3.3.4. Pour the sample onto a clean, dry, absorptive cloth; obtain a saturated surface dry condition; pour sample from cloth into a tared pan; reweigh sample to nearest  $22 \times 10^{-2}$  (0.1g). Subtract original weight of aggregate and record difference as percent oil absorbed (based on 0.22 lb. (100 g) of aggregate).

3.3.5. Subtract the percent oil absorbed value (see 3.3.4. above) from the percent oil retained value (see 3.3.3. above), and obtain the percent (free) oil retained value. This value represents the percent oil retained value that would have been obtained had the aggregate been a nonabsorptive type. The above technique allows one to evaluate the aggregate's surface and shape characteristics without the over-whelming influence of a large quantity of absorbed oil.

3.3.6. Follow the procedure recommended in Steps 3.1.8. and 3.2. The only exception is that the percent (free) oil retained value is used (from Step 3.3.5.) to obtain  $K_a$ . Thus, the asphalt quantity determined is the "effective" asphalt content.

3.3.7. Follow the recommended procedure indicated through Sections 4 and 5. Because asphalt absorption is not presently included in the formula for the determination of fine aggregate content, it is particularly desirable that the effects of oil absorption in the  $K_a$  test be excluded in the case of the highly absorptive aggregate.

3.3.8. Prepare a trial mixture using an asphalt content equal to or somewhat greater than (try to estimate amount that will be absorbed) the effective asphalt content determined in Step 3.3.6. and also using the aggregate gradation as determined in Step 3.3.7. Using a suitable technique, such as the test for maximum specific quantity of asphalt absorbed (in percent, based on total weight of aggregate).

3.3.9. Determine the total asphalt content of the subject mixture by adding the effective asphalt content (from Step 3.3.6.) to the absorbed asphalt content (from Step 3.3.8.).

3.3.10. Follow the recommended procedure indicated in Sections 6 and 7, using the total asphalt content for all subsequent computations and trials (from Step 3.3.9.).

#### Void Capacity of Coarse Aggregate

4.1. Use the following procedure to determine the vibrated unit weight and void capacity of the coarse-aggregate fraction (material retained on a No. 8 (2.76 mm) sieve) of the proposed job-mix gradation.

##### 4.1.1. Apparatus.

Rammer - A portable electromagnetic vibrating rammer as shown in Figure A-2, having a frequency of 3,600 cycles per min., suitable for use with 115-V ac. If this is not available, a rammer with a frequency of 2200 cycles per min. can also be used. The rammer shall have a tamper foot and extension as shown in Figure A-3.

Mold - A solid-wall metal cylinder with a detachable metal base plate and a detachable metal guide-reference bar as shown in Figure A-4.

Wooden Base - A plywood disc 15 in. (381 mm) in diameter, 2 in. (50.8 mm) thick, with a cushion of rubber hose attached to the bottom. The disc shall be constructed so it can be firmly attached to the base plate of the compaction mold.

Timer - A stopwatch or other timing device graduated in divisions of 1.0 sec. and accurate of 1.0 sec. and capable of timing the unit for up to 2 min. An electric timing device or electrical circuits to start and stop the vibratory rammer may be used.

Dial Indicator - A dial indicator graduated in 0.001 in.<sup>1</sup> increments and having a travel range of 3.0 in. (76.2 mm).

4.1.2. Sample: Select a 5 lb. (2268 gm) sample of the coarse-aggregate [3.22 lb. (1461 gm)] sample to lightweight aggregate) fraction from the proposed job-mix formula as verified in Step 3.1.

---

<sup>1</sup>Units are maintained to be consistent with instrument.

#### 4.1.3. Procedure.

- (a) Pour the selected sample into the compaction mold and place the tamper foot on the sample.
- (b) Place the guide-reference bar over the shaft of the tamper foot and secure the bar to the mold with the thumb screws.
- (c) Place the vibratory rammer on the shaft of the tamper foot and vibrate for 15 sec. (25 sec. in the case where a 2200 Hz frequency rammer is used). During the vibration period, the operator must exert just enough pressure on the hammer to maintain contact between the sample and the tamper foot.
- (d) Remove the vibratory rammer from the shaft of the tamper foot and brush any fines from the top of the tamper foot. Measure the thickness (t) of the compacted material to the nearest 0.001 in. (.025 mm). Note: The thickness (t) of the compacted sample is determined by adding the dial reading, minus the thickness of the tamper foot, to the measured distance from the inside bottom of the mold and the end of the dial gauge when it is seated on the guide-reference bar with stem fully extended.

#### 4.1.4. Calculations.

Calculate the vibrated unit weight (X) as follows:

$$X = 6912(w) / (d)^2 t (\text{lb/ft}^3)$$

Where w = wt. of coarse-aggregate fraction (lb.)

d = diam. of compaction mold (in.)

If w = 5 lb. and d = 6 in:

$$X = 305.58 / t (\text{lb/ft}^3)$$

where t is in inches.

Determine the void capacity (VMA) as follows:

$$VMA = 100 (1 - X/U_c) (\text{in percent})$$

Where  $U_c$  = bulk solid unit weight (lb/ft<sup>3</sup>) of the coarse-aggregate fraction.  $U_c$  is calculated from bulk specific gravity, as determined in Step 2.2<sup>c</sup>, multiplied by 62.4 lb/ft.

### Optimum Content of Fine Aggregate

5.1. Determine the optimum content of fine-aggregate fraction using the following relationship:

$$Y = \frac{[\% \text{ VMA} - V] - [(\% \text{ AC}) (X)/U_a]}{[(\% \text{ VMA} - V)/100] + [(X)/U_f]}$$

Where:

$Y$  = Percent passing the No. 8 (2.38mm) sieve (by weight)

$X$  = Actual vibrated unit weight of coarse aggregate (retained on the No. 8 (2.38 mm) sieve)

$U_f$  = Theoretical bulk dry solid unit weight of fine aggregate (passing the No. 8 (2.38 mm) sieve)

$U_a$  = Unit weight of asphalt cement

% AC = Percent asphalt by total weight of aggregate

$$(2.0 K_c + 4.0) \frac{2.65}{(SG)_{ca}}$$

$V$  = Design percent air voids (15.0 percent)

% VMA = Percent voids mineral aggregate of the coarse aggregate (retained on the No. 8 (2.38 mm) sieve), which is  $100 - (100) (X)/U_c$

$U_c$  = Theoretical bulk dry solid unit weight of coarse aggregate (retained on the No. 8 (2.38 mm) sieve)

Note:  $X$ ,  $U_a$ ,  $U_c$  and  $U_f$  are in pounds per cubic foot.

In the above relationship, asphalt absorption by aggregate has been assumed to be negligible. Because asphalt absorption requirements are considered in the test for  $K_c$  (see Step 3.1), the estimated air voids of 15 percent in the mixture<sup>c</sup> will actually be greater by an

amount equivalent to the volume of asphalt absorbed, in percent. This condition provides, if anything, an additional safety factor.

If the value thus obtained for fine-aggregate content is greater than 15 percent, a value of 15.0 percent shall be used.

5.2. Compare the optimum fine-aggregate content (Y) determined in Step 5.1. to the amount passing the No. 8 (2.38 mm) sieve of the contractor's proposed job-mix formula. If these values differ by more than plus or minus 1 percentage point, reconstruct a revised or adjusted job-mix formula using the value determined for optimum fine-aggregate content. Recompute the proportions of coarse and fine aggregates (as received) to meet the revised job-mix formula for submission to the contractor.

Note: If the proposed and revised job-mix gradations are significantly different, it may be necessary to rerun portions of this procedure.

#### Sulfur-Extended Asphalt (SEA) Binder

6.1. Establish sulfur content in the sulfur-extended asphalt binder.

6.2. It is recommended that equal volume of SEA binder should be used to replace the asphalt cement binder. Since binder content is expressed in unit of percent by dry weight of total aggregates, and that the specific gravities of sulfur and asphalt are different, it is suggested that the following formula be used to determine the SEA binder content:

$$\text{Percent SEA binder} = (A) \frac{100R}{[100R - S(R-G)]}$$

where

A = Percent asphalt by dry weight of aggregate before substitution.

R = Ratio of specific gravities of sulfur to asphalt-(2:1),

S = Percent sulfur by weight in the SEA binder, and

G = Specific gravity of the asphalt

6.3. Place the desired proportions of hot sulfur and hot asphalt in a weigh bucket, stir slowly for about 60 sec. (stir before each use).

### Optimum Mixing Temperature

7.1. Prepare a 2.2 lb. (1,000 g) sample of aggregate [1.32 lb. (600 g) sample for lightweight aggregate] in the proportions determined in Section 5. Mix this sample at the asphalt content determined in Step 3.2 (or at the SEA binder content determined in Step 6.2) at a temperature corresponding to an asphalt viscosity of 800 centistokes determined in Step 2.3. When the mixture is completely coated, transfer it to a pyrex glass plate (diam. 8 to 9 in.) (diam. 203.2 to 228.6 mm) and spread the mixture with a minimum of manipulation. Return it to the oven at the mixing temperature. Observe the bottom of the plate after 15 and 60 min. A slight puddle at points of contact between aggregate and glass plate is suitable and desirable. Otherwise, repeat the test at a lower mixing temperature, or higher if necessary.

Note: If asphalt drainage occurs at a mixing temperature that is too low to provide for adequate drying of the aggregate, an asphalt of a higher grade should be used.

A schematic diagram of the SEA-OGFC Mix-Design Procedure is presented in Figure H-1.

### Laboratory Evaluation Procedure

1. Prepare four 4" (101.6 mm) diameter x 2 1/2" (63.5 mm) thickness test specimens according to the compaction procedure outlined in Appendix B.
2. Place half of the samples in a vacuum-saturated condition for 7 days.
3. Conduct the following tests on the test specimens (both before and after moisture conditioning):
  - a. Bulk specific gravity of compacted specimen
  - b. Resilient modulus at 32°F (0°C), 68°F (20°C) and 77°F (25°C). (Ref. H-2)
  - c. Hveem stability (ASTM D1560)
  - d. Marshall stability (ASTM D1559)
  - e. Maximum specific gravity of mixture (ASTM D2041-71)
  - f. Air voids



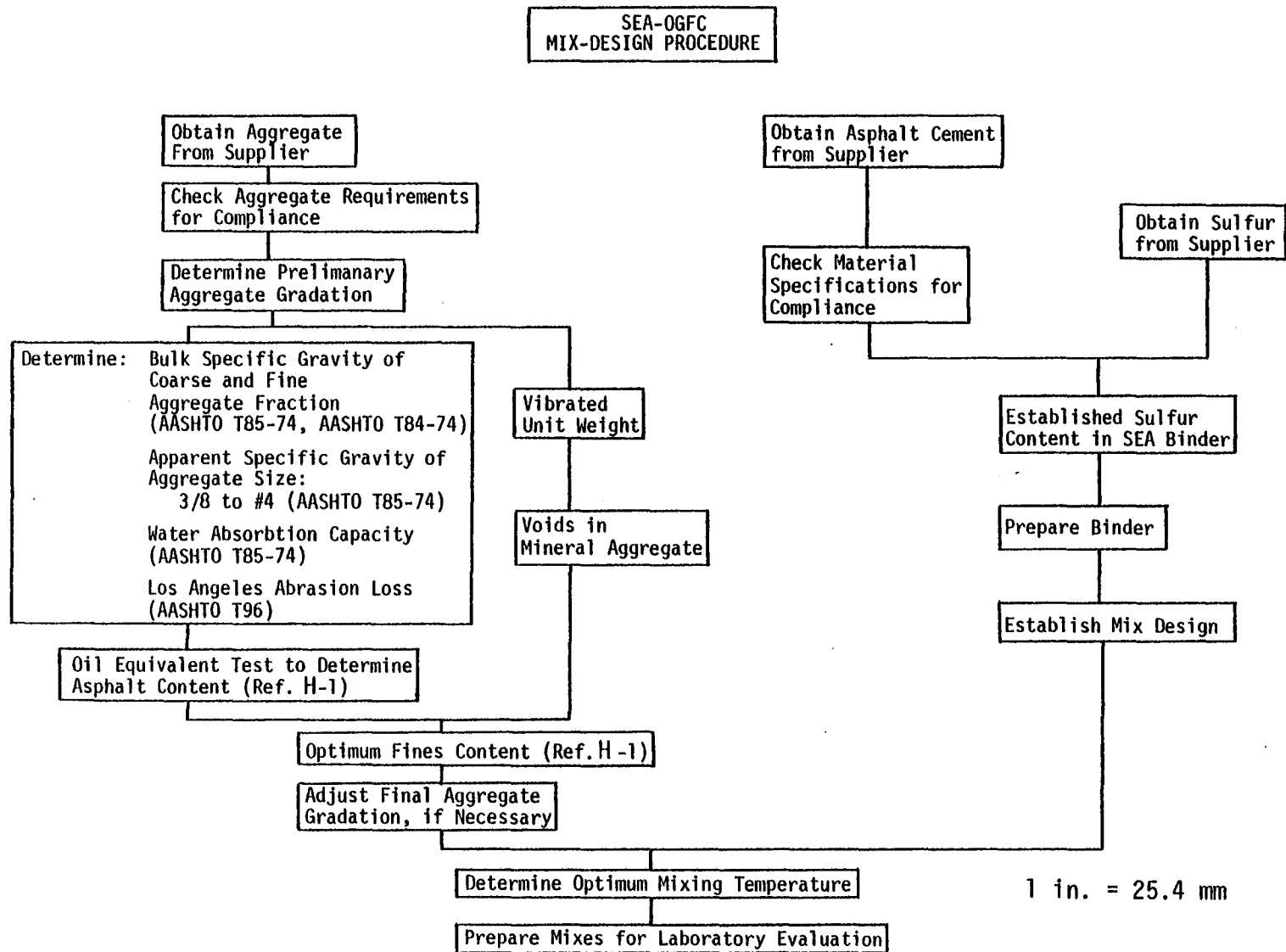


Figure H-1. Mix-Design Procedure for SEA-OGFC Mixture

4. Prepare six 4" (101.6 mm) x 4" (101.6 mm) thickness test specimens according to the compaction procedure outlined in Appendix D.

5. Conduct the following tests on the prepared test specimens:

- a. Bulk specific gravity of compacted specimen
- b. Compressive strength and retained strength (AASHTO T167)
- c. Maximum specific gravity of mixture (ASTM D2041-71)
- d. Air voids

6. Prepare three 6" (152.4 mm) diameter x 1" (25.4 mm) thickness test specimens according to the compaction procedures outlined in Appendix E.

7. Conduct the following tests on the prepared test specimens:

- a. Permeability test (Appendix F)
- b. Freeze-thaw durability (Appendix G)

8. After the evaluation tests (See Figure H-2) have been completed on the mixes, compare the test results for compliance with the tentatively established criteria:

Marshall Stability, lb.	300 min.
Flow, .01 in.	8-20
Hveem Stability, percent	30 min.
Resilient Modulus at 68°F (20°C), psi	300,000 min.
Index of Retained Strength, percent	60 min.
Permeability, cm/sec	0.30 min.
Freeze-Thaw Durability	No visual damage after 100 cycles

1 lb = 4.5N                      1 in = 25.4 mm                      1 psi = 6.9 kPa  
1 in/hr = 0.0007 cm/sec

# LABORATORY EVALUATION PROCEDURE

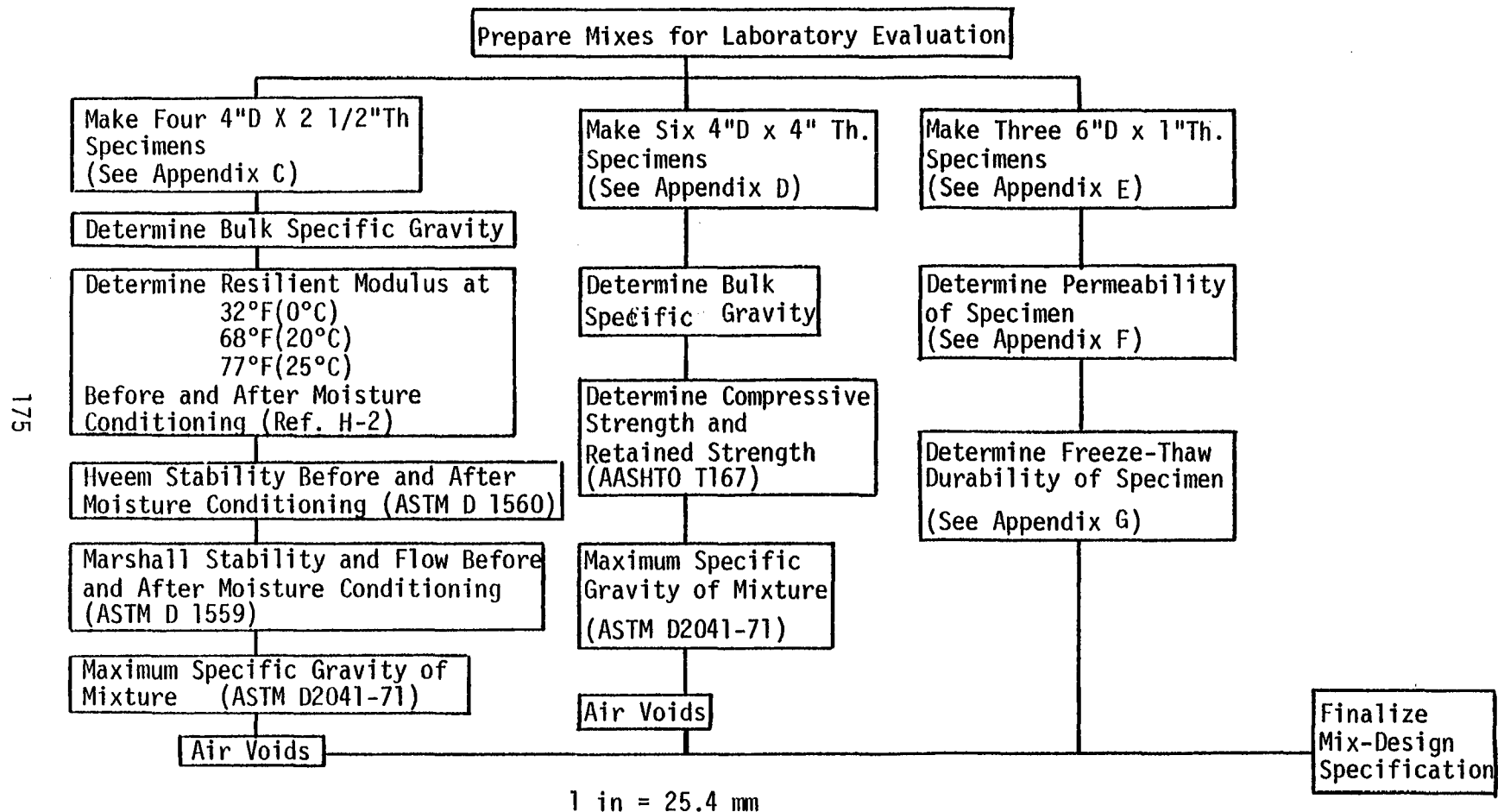


Figure H-2. Laboratory Evaluation Procedure for SEA-OGFC Mixture

## REFERENCE

- H-1. "Open-Graded Friction Courses for Highways", National Cooperative Highway Research Program Synthesis of Highway Practice No. 49, Transportation Research Board, National Research Council, Appendix A.
- H-2. Schmidt, R. J., "A Practical Method for Determining the Resilient Modulus of Asphalt Treated Mixes", Highway Research Record No. 404, 1972.

1

# DATA SHEET FOR SEA-OGFC MIX-DESIGN

## 1. AGGREGATES

### A. Supplier

Source \_\_\_\_\_

Polish value \_\_\_\_\_

### B. Proposed Proportions (by Volume)

### C. Proposed Job-Mix Gradation

<u>Sieve size</u>	<u>Specification limits</u>	<u>Percent Passing</u>			<u>Job-mix blend</u>
		_____	_____	_____	
1/2 in. (12.7 mm)	95-100	_____	_____	_____	_____
3/8 in. (9.52 mm)		_____	_____	_____	_____
No. 4 (4.76 mm)	30-50	_____	_____	_____	_____
No. 8 (2.38 mm)	5-15	_____	_____	_____	_____
No. 16 (1.19 mm)	2-5	_____	_____	_____	_____
No. 200 (0.074 mm)		_____	_____	_____	_____

### D. Specific Gravity - Unit Weight

	<u>Apparent SG</u>	<u>Bulk SG (dry basis)</u>	<u>Bulk solid unit weight (lb/ft<sup>3</sup>)</u>
Coarse Aggregate (retained on No. 8 (2.38 mm) sieve)	_____	_____	_____ (U <sub>c</sub> )
Fine Aggregate (passing No. 8 (2.38 mm) sieve)	_____	_____	_____ (U <sub>f</sub> )
3/8 in. (9.52 mm) to No. 4 (4.76 mm) sieve fraction	_____	_____	_____

E. Resistance to Abrasion

Los Angeles Abrasion Loss, percent = \_\_\_\_\_

F. Void Capacity of Coarse Aggregate

Unit weight (vibrated, lb/ft<sup>3</sup>) = \_\_\_\_\_ (X)

Voids in mineral aggregate (%) = \_\_\_\_\_ (VMA)

G. K<sub>C</sub> Determination

Oil retention (g oil per 100 g aggregate) = \_\_\_\_\_

Oil retention (corrected, 2.65 SG) = \_\_\_\_\_

K<sub>C</sub> (from chart) = \_\_\_\_\_

H. Asphalt Content (AC, %)

Percent asphalt (aggregate basis) = \_\_\_\_\_ (AC)

$(2.0 K_C + 4.0) \times \frac{2.65}{\text{apparent SG of coarse aggregate (3/8 in. to No. sieve)}}$

2. BINDER

A. Asphalt

Source and Type \_\_\_\_\_

Specific gravity at 77°F (25°C) = \_\_\_\_\_

Unit weight (lb/ft<sup>3</sup>) = \_\_\_\_\_ (U<sub>a</sub>)

Viscosity at 140°F (60°C), poises = \_\_\_\_\_

Viscosity at 270°F (135°C), poises = \_\_\_\_\_

Flash point, °F = \_\_\_\_\_

Ductility at 77°F (25°C), cm = \_\_\_\_\_

Ring and Ball, °F = \_\_\_\_\_

B. Sulfur

Source \_\_\_\_\_

Melting point, °F = \_\_\_\_\_

C. Sulfur-Extended Asphalt Binder

Sulfur/Asphalt (S/A) weight ratio = \_\_\_\_\_

<u>Temperature F° (°C)</u>	<u>Viscosity (centistokes)</u>
270 (132.2)	_____
260 (126.7)	_____
250 (121.1)	_____
245 (118.3)	_____
240 (115.6)	_____
230 (110.0)	_____
220 (104.4)	_____

SEA binder weight percent (aggregate basis) required to  
replace the asphalt content =

$$(AC) \frac{100R}{[100R - S(R-G)]} = \underline{\hspace{2cm}}$$

3. OPTIMUM FINE-AGGREGATE CONTENT (Y)

Using Formula \_\_\_\_\_ Chart \_\_\_\_\_

Where:  $X = \underline{\hspace{2cm}} \text{ lb/ft}^3$        $VMA = \underline{\hspace{2cm}} \%$   
 $U_f = \underline{\hspace{2cm}} \text{ lb/ft}^3$        $AC = \underline{\hspace{2cm}} \%$   
 $U_c = \underline{\hspace{2cm}} \text{ lb/ft}^3$        $V = \underline{\hspace{2cm}} \%$   
 $U_a = \underline{\hspace{2cm}} \text{ lb/ft}^3$

Find:  $Y =$  \_\_\_\_\_ % (specs. limits:  $5 < Y < 15$ )

Remarks:

4. FINAL JOB-MIX GRADATION

Sieve size	Job-mix blend (percent passing)
1/2 in. (12.7 mm)	_____
3/8 in. (9.52 mm)	_____
No. 4 (4.76 mm)	_____
No. 8 (2.38 mm)	_____
No. 16 (1.19 mm)	_____
No. 200 (0.074 mm)	_____

5. OPTIMUM MIXING TEMPERATURE (SEA-OGFC)

<u>Temperature (°F)</u>	<u>Viscosity (centistokes)</u>	<u>Drainage</u>	<u>Use</u>
_____	_____	_____	_____
_____	_____	_____	_____
_____	_____	_____	_____



REPORT ON LABORATORY EVALUATION TESTS

1. 4" (101.6 mm) DIAMETER X 2 1/2" (63.5 mm) THICKNESS SPECIMENS

A. Bulk specific gravity of specimens, average = \_\_\_\_\_

B. Resilient modulus at 32°F (0°C), psi = \_\_\_\_\_

68°F (20°C), psi = \_\_\_\_\_

77°F (25°C), psi = \_\_\_\_\_

C. Hveem stability, percent = \_\_\_\_\_

D. Marshall stability, lb. = \_\_\_\_\_

Flow, .01 in. = \_\_\_\_\_

E. Maximum specific gravity of mixture = \_\_\_\_\_

F. Air voids, percent of mixture = \_\_\_\_\_

2. 4" (152.4 mm) DIAMETER X 1" (25.4 mm) THICKNESS SPECIMENS

A. Bulk specific gravity of specimens, average = \_\_\_\_\_

B. Compressive strength, psi: (a) \_\_\_\_\_ (b) \_\_\_\_\_ (c) \_\_\_\_\_

Retained strength, psi: (a) \_\_\_\_\_ (b) \_\_\_\_\_ (c) \_\_\_\_\_

Average compressive strength, psi = \_\_\_\_\_

Average retained strength, psi = \_\_\_\_\_

Index of retained strength, percent = \_\_\_\_\_

3. 6" (152.4 mm) DIAMETER X 1" (25.4 mm) THICKNESS SPECIMENS

A. Permeability, (cm/sec): (a) \_\_\_\_\_ (b) \_\_\_\_\_ (c) \_\_\_\_\_

B. Predicted flooding rainfall intensity, (in/hr) = (a) \_\_\_\_\_

(b) \_\_\_\_\_ (c) \_\_\_\_\_

C. Freeze-thaw durability (visual observation after 100 cycles)

(a) \_\_\_\_\_ (b) \_\_\_\_\_ (c) \_\_\_\_\_

D. Brush test, gm. (a) \_\_\_\_\_ (b) \_\_\_\_\_ (c) \_\_\_\_\_

Table I-1. Preliminary Gradation for Limestone, East Texas Sandstone and Streetman Aggregate.

Sieve Size	Aggregate	Percent Passing by Volume			
		Limestone	East Texas Sandstone	Streetman	FHWA Specification
1/2" (12.7 mm)		100	100	100	100
3/8" (9.52 mm)		97	95	98	95-100
#4 (4.76 mm)		41	32	30.3	30-15
#8 (2.38 mm)		10	7	7	5-15
#16 (1.19 mm)		7	4.9	4.9	
#30		6	4.2	4.2	
#50		5	3.5	3.5	
#100		4	2.8	2.8	
#200 (0.074 mm)		3	2.1	2.1	2-5

Table I-2. Characterization Data for Selected Program Aggregate Systems

Test	Test Method	Aggregate		
		Limestone	East TX Sandstone	Streetman (Lt. Wt.)
(1) Bulk Sp. Gr. (Coarse)	AASHTO T85	2.66	2.74	1.25
(2) Bulk Sp. Gr. (Fine)	AASHTO T84	2.54	2.54	2.54
(3) Asphalt Content, %	FHWA Design [1]	6	6	11
(4) L.A. Abrasion Loss, %	AASHTO T96	23	22	18
(5) Absorption, % (Coarse)	AASHTO T85	0.7	0.9	14.8
(6) Absorption, % (Fine)	AASHTO T84	0.2	2.2	2.2
(7) Void Capacity (VMA) %	FHWA Design [1]	31.5	31.3	41.8
(8) Vibrated Unit Wt. PCF	FHWA Design [1]	114	117	46.6
(9) Opt. Fine Content, %	FHWA Design [1]	6.3	5.6	15 (Vol) 26 (Wt.)

Table I-3. Asphalt Cement Properties.

Test	Test Methods	AC-10	AC-20	AC-40
		Test Results	Test Results	Test Results
Viscosity @ 140°F, poises	ASTM D-2171	1576	1934	4815
Pen @ 77°F, dmm	ASTM D-5	118	63	63
Flash Point, °F	ASTM D-92	615	580	608
Ductility @ 77°F, cm	ASTM D-113	150+	141+	60
Ring and Ball, °F	ASTM D-36	102	120	132

$$^{\circ}\text{C} = \frac{5}{9} (^{\circ}\text{F} - 32)$$

$$1 \text{ in.} = 2.54 \text{ cm}$$

Table I-4. Resilient Modulus vs. Temperature for East Texas Sandstone Mixes-  $M_R \times 10^{-3}$  psi

Asphalt Grade	o/w S*	34°F		68°F		77°F	
		Dry	VacSat	Dry	VacSat	Dry	VacSat
AC-10	0	668	237	69	29	28	27
	20 (D <sub>1</sub> )	945	914	114	133	68	78
	30 (D <sub>2</sub> )	1039	881	198	120	113	73
	40 (D <sub>1</sub> )	1421	1276	337	248	162	129
AC-20	0	1107	627	163	77	140	70
	20 (D <sub>2</sub> )	2085	1690	353	253	218	170
	30 (D <sub>2</sub> )	2317	1515	520	374	338	192
	40 (D <sub>2</sub> )	2355	1738	534	396	356	239
AC-40	0	563	616	99	51	84	48
	20 (D <sub>1</sub> )	670	589	196	141	143	124
	30 (D <sub>2</sub> )	1432	1104	373	206	267	175
	40 (D <sub>1</sub> )	1750	1513	387	288	242	303

Note: D<sub>1</sub> - Specimens Prepared by Direct Substitution.

D<sub>2</sub> - Specimens Prepared by Modified Direct Substitution (Bureau of Mines).

\* o/w S denotes weight percent sulfur in the binder.

1 psi = 6.9 kPa

Table I-5. Resilient Modulus vs. Temperature for Limestone Mixes-  $M_R \times 10^{-3}$  psi

ID's	Asphalt Grade	o/w S*	34°F		68°F		77°F	
			Dry	VacSat	Dry	VacSat	Dry	VacSat
1-4	AC-10	0	686	548	80	51	50	49
13-16		20 (D <sub>2</sub> )	862	838	204	141	104	81
25-28		30 (D <sub>2</sub> )	860	841	219	168	102	92
37-40		40 (D <sub>2</sub> )	907	888	237	206	144	119
5-8	AC-20	0	1243	398	275	128	145	112
17-20		20 (D <sub>2</sub> )	1359	1204	400	309	268	150
29-32		30 (D <sub>2</sub> )	1803	1457	494	286	290	203
41-44		40 (D <sub>2</sub> )	1794	1572	761	612	376	305
9-12	AC-40	0	664	445	107	74	92	67
21-24		20 (D <sub>2</sub> )	1186	877	346	248	235	154
33-36		30 (D <sub>2</sub> )	1211	983	332	261	220	156
45-48		40 (D <sub>2</sub> )	2125	1637	499	323	355	234

Note. D<sub>2</sub> Specimens Prepared by Modified Direct Substitution - (Bureau of Mines).

\* o/w S denotes weight percent sulfur in the binder.

1 psi = 6.9 kPa

Table I-6. Resilient Modulus vs. Temperature for Streetman Aggregate Mixes -  $M_R \times 10^{-3}$  psi

Asphalt Grade	o/w S*	34°F		68°F		77°F	
		Dry	VacSat	Dry	VacSat	Dry	VacSat
AC-10	0	579	494	108	115	93	57
	20 (D <sub>2</sub> )	636	590	152	130	107	104
	30 (D <sub>2</sub> )	777	685	234	156	144	92
	40 (D <sub>2</sub> )	777	736	296	176	207	137
AC-20	0	901	807	284	297	232	134
	20 (D <sub>2</sub> )	956	916	229	243	146	168
	30 (D <sub>2</sub> )	1124	935	279	281	168	185
	40 (D <sub>2</sub> )	1444	1225	348	352	242	220
AC-40	0	540	424	153	149	133	96
	20 (D <sub>2</sub> )	747	X	136	234	118	190
	30 (D <sub>2</sub> )	1074	683	328	232	251	259
	40 (D <sub>2</sub> )	843	X	451	315	323	297

Note: D<sub>2</sub> - Specimens Prepared by Modified Direct Substitution - (Bureau of Mines).

\* o/w S denotes weight percent of sulfur in the binder.

1 psi = 6.9 kPa



Table I-7. Structural Characterization Test Results for East Texas Sandstone Mixes.

Asphalt Grade	o/w S*	Hveem Stability,%		Marshall Stability,lb.		Marshall Flow, 0.01 in		Bulk Compacted Sp. Gr.	Rice Theoretical Sp. Gr.	Air Voids	VMA
		Dry	VacSat	Dry	VacSat	Dry	VacSat				
AC-10	0	21	23	136	180	24	21	1.929	2.498	22.8	33.3
	20 (D <sub>1</sub> )	24	24	250	270	20	20	1.933	2.548	24.2	33.6
	30 (D <sub>2</sub> )	42	43	340	320	18	19	1.967	2.552	22.9	32.7
	40 (D <sub>1</sub> )	27	24	400	320	24	20	1.972	2.536	22.3	33.2
AC-20	0	22	26	300	340	28	25	1.937	2.534	23.6	33.0
	20 (D <sub>2</sub> )	35	39	470	430	14	15	1.963	2.540	22.7	32.6
	30 (D <sub>2</sub> )	32	33	480	440	15	17	1.959	2.532	22.6	32.9
	40 (D <sub>2</sub> )	35	32	550	520	19	17	1.967	2.565	23.3	32.9
AC-40	0	23	27	320	380	30	24	1.918	2.533	24.3	33.7
	20 (D <sub>1</sub> )	26	25	450	420	27	23	1.948	2.520	22.7	33.1
	30 (D <sub>2</sub> )	33	32	610	690	20	17	1.991	2.566	22.4	32.4
	40 (D <sub>1</sub> )	29	26	680	660	30	21	2.003	2.600	23.0	31.7

Note: D<sub>2</sub> - Specimens Prepared by Modified Direct Substitution (Bureau of Mines)<sup>o</sup>

\* o/w S denotes weight percent of sulfur in the binder.

1 lb. = 4.5 N

Table I-8. Structural Characterization Test Results for Limestone Mixes.

Asphalt Grade	o/w S*	Hveem Stability,%		Marshall Stability,lb.		Marshall Flow, 0.01 in		Bulk Compacted Sp. Gr.	Theoretical Sp. Gr.	Air Voids	VMA
		Dry	VacSat	Dry	VacSat	Dry	VacSat				
AC-10	0	X	27	X	130	X	16	1.974	2.514	21.5	29.8
	20 (D <sub>2</sub> )	26	31	220	300	20	19	1.986	2.517	21.1	29.8
	30 (D <sub>2</sub> )	33	32	200	260	24	19	1.969	2.523	22.2	30.7
	40 (D <sub>2</sub> )	29	27	250	300	17	19	1.963	2.529	22.4	31.2
AC-20	0	23	29	200	250	22	18	1.967	2.558	23.1	30.0
	20 (D <sub>2</sub> )	32	33	240	270	15	19	1.958	2.535	22.8	30.8
	30 (D <sub>2</sub> )	30	30	290	330	23	19	1.957	2.506	22.7	31.1
	40 (D <sub>2</sub> )	33	29	390	410	16	18	1.947	2.512	22.5	31.7
AC-40	0	23	27	270	260	25	24	1.973	2.481	20.5	29.8
	20 (D <sub>2</sub> )	32	33	410	440	19	19	1.949	2.531	23.0	31.1
	30 (D <sub>2</sub> )	30	30	330	360	18	17	1.962	2.527	22.3	30.9
	40 (D <sub>2</sub> )	30	32	450	500	17	19	1.977	2.544	22.3	30.7

Note: D<sub>2</sub> - Specimens Prepared by Modified Direct Substitution (Bureau of Mines).

\* o/w S denotes weight percent of sulfur in the binder.

Table I-9. Structural Characterization Tests Results for Streetman Aggregate Mixes.

Asphalt Grade	o/w S*	Hveem Stability,%		Marshall Stability,lb.		Marshall Flow, 0.01 in		Compacted Sp. Gr.	Theoretical Sp. Gr.	Air Voids	VMA
		Dry	VacSat	Dry	VacSat	Dry	VacSat				
AC-10	0	23	27	520	580	27	25	1.306	1.580	17.4	18.3
	20 (D <sub>2</sub> )	26	24	600	690	34	26	1.232	1.690	27.1	23.8
	30 (D <sub>2</sub> )	31	30	680	670	26	29	1.251	1.625	23.0	23.1
	40 (D <sub>2</sub> )	28	23	710	680	28	32	1.227	1.622	24.4	25.2
AC-20	0	21	27	620	580	28	28	1.291	1.592	18.9	19.2
	20 (D <sub>2</sub> )	33	29	680	670	25	23	1.240	1.652	25.0	23.3
	30 (D <sub>2</sub> )	31	32	670	770	27	28	1.238	1.669	25.8	23.9
	40 (D <sub>2</sub> )	30	29	840	910	30	31	1.237	1.614	23.4	24.5
AC-40	0	24	28	670	700	23	28	1.306	1.643	20.5	18.3
	20 (D <sub>2</sub> )	26	31	780	820	32	34	1.212	1.615	25.0	25.0
	30 (D <sub>2</sub> )	33	32	900	950	28	32	1.246	1.617	25.4	23.4
	40 (D <sub>2</sub> )	25	34	950	740	40	32	1.246	1.655	24.7	24.0

Note: D<sub>2</sub> - Specimens Prepared by Modified Direct Substitution (Bureau of Mines).

\*o/w S denotes weight percent of sulfur in the binder

Table I-10. Immersion-Compression Test Results for East Texas Sandstone Mixes

Asphalt Grade	o/w S	Compressive Strength (psi)		Index of Retained Strength %	Air Voids %
		Dry	Wet		
AC-10	0	114.3	99.2	86.8	17.7
	20	95.0	70.0	73.7	20.2
	30	107.7	81.7	75.9	20.3
	40	119.0	74.4	62.5	20.7
AC-20	0	185.7	149.6	80.6	19.3
	20	158.9	114.9	72.3	20.1
	30	169.6	111.7	65.7	19.5
	40	215.9	138.0	63.9	20.7
AC-40	0	158.6	107.2	67.6	20.1
	20	148.5	117.5	79.1	19.2
	30	158.6	124.7	78.6	20.7
	40	199.2	143.0	71.2	21.3

1 psi = 6.9 kPa

Table I-11. Immersion-Compression Test Results for Limestone Mixes.

Asphalt Grade	o/w S	Compressive Strength (psi)		Index of Retained Strength %	Air Voids %
		Dry	Wet		
AC-10	0	101.0	84.6	83.7	20.4
	20	84.9	66.3	78.1	20.1
	30	120.2	76.1	63.4	20.8
	40	120.7	61.8	60.2	20.7
AC-20	0	88.6	74.0	83.5	--
	20	132.9	109.6	82.4	21.5
	30	126.5	100.0	79.0	20.3
	40	144.3	104.8	72.6	20.6
AC-40	0	12.30	93.3	76	--
	20	161.3	102.9	63.8	20.7
	30	190.6	103.1	54.0	20.1
	40	252.3	119.7	47.4	20.4

1 psi = 6.9 kPa

Table I-12. Permeability Test Results for East Texas Sandstone SEA-OGFC Mixtures

	o/w S	Permeability Coefficient $K_v$ (cm/sec)
AC-10	20	0.504
	30	0.505
AC-20	20	0.519
	30	0.400
	40	0.502
AC-40	20	0.515
	30	0.401
	40	0.336

1 in/hr = 0.0007 cm/sec

Table I-13. Permeability Test Results for Limestone Mixtures

	o/w S	Permeability Coefficient $K_v$ (cm/sec)
AC-10	0	0.071
	20	0.200
	30	0.259
	40	0.287
AC-20	0	0.056
	20	0.238
	30	0.210
	40	0.187
AC-40	0	0.107
	20	0.262
	30	0.230
	40	0.248

1 in/hr = 0.0007 cm/sec

Table I-14. Permeability Test Results for Streetman SEA-OGFC Mixtures

	o/w S	Permeability Coefficient $K_v$ (cm/sec)
AC-10	0	0.225
	20	0.266
	30	0.321
AC-20	20	0.296
	30	0.283
	40	0.320
AC-40	30	0.277

1 in/hr = 0.0007 cm/sec



Table I-15. Freeze Thaw Test Results for East Texas Sandstone SEA-OGFC Mixtures

Asphalt Grade	o/w S	Visual Observation at					Brush Test	
		0 Cycles	25 Cycles	50 Cycles	75 Cycles	100 Cycles	Weight Loss (gm)	
AC-10	30	(1)	0	0	0	0	0	0.3
		(2)	0	0	0	0	0	0.6
		(3)	0	0	0	0	0	0.8
AC-20	20	(1)	0	0	0	0	0	0.4
		(2)	0	0	0	0	0	0.4
		(3)	0	0	0	0	0	0.6
AC-20	30	(1)	0	0	0	0	0	0.3
		(2)	0	0	0	0	0	0.2
		(3)	0	0	0	0	0	0.6
AC-20	40	(1)	0	0	0	0	0	0.3
		(2)	0	0	0	0	0	0.4
		(3)	0	0	0	0	0	0.4
AC-40	30	(1)	0	0	0	0	2 - 1	2.6
		(2)	2 - 1	2 - 1	2 - 1	3 - 1	3 - 1	3.8
		(3)	0	0	0	0	2 - 1	2.0

1 lb. = 454 gms

Table I-16. Freeze-Thaw Test Results for Limestone Mixes.

Asphalt Grade	o/w S		Visual Observation at					Brush Test Weight Loss (gm)
			0 Cycles	25 Cycles	50 Cycles	75 Cycles	100 Cycles	
AC-10	0	(1)	0	0	0	2-1	2-1, 3-1	0.4
		(2)	0	0	2-1	2-1, 3-1	2-1, 3-1	0.1
		(3)	0	0	0	2-1	2-2	0.2
AC-10	20	(1)	0	0	0	2-1	3-1	0.2
		(2)	0	0	0	0	0	0.1
		(3)	0	0	0	0	2-1	0.3
AC-10	30	(1)	0	0	0	2-1	3-1	0.2
		(2)	0	0	0	0	0	0.2
		(3)	0	0	0	0	0	0.1
AC-10	40	(1)	0	0	0	2-1	2-1	0.3
		(2)	2-1	2-1	2-1	2-1	2-1, 3-1	0.9
		(3)	2-1	2-1	2-1	2-1	2-1	0.4
AC-20	0	(1)	0	0	0	2-1	2-1, 3-1	0.2
		(2)	0	0	0	2-1	2-1, 3-1	0.4
		(3)	0	0	0	2-1	2-2, 3-1	0.3
AC-20	20	(1)	0	0	0	0	2-1	0
		(2)	0	0	0	0	0	0
		(3)	0	0	0	0	0	0

1 lb. = 454 gm

Table I-16.Continued

Asphalt Grade	o/w S		Visual Observation at					Brush Test Weight Loss (gm)
			0 Cycles	25 Cycles	50 Cycles	75 Cycles	100 Cycles	
AC-20	30	(1)	0	0	0	0	2-1	0
		(2)	0	0	0	0	2-1	0
		(3)	0	0	0	0	2-1	0.1
AC-20	40	(1)	0	0	0	0	0	0.2
		(2)	0	0	0	0	0	0.2
		(3)	0	0	0	0	0	0.2
AC-40	0	(1)	0	0	0	2-1, 3-1	2-2, 3-1	0.2
		(2)	0	0	0	2-1	2-1, 3-1	0.4
		(3)	0	0	0	2-1	2-1, 3-1	0.4
AC-40	20	(1)	0	0	0	0	0	0
		(2)	0	0	0	0	2-1	0.3
		(3)	0	0	0	0	0	0.3
AC-40	30	(1)	0	0	0	0	0	0.4
		(2)	0	0	0	0	0	0.1
		(3)	0	0	0	0	0	0.1
AC-40	40	(1)	0	0	0	0	0	0.4
		(2)	0	0	0	0	0	0.2
		(3)	0	0	0	0	0	0.1

1 lb. = 454 gms

Table I-17. Freeze-Thaw Test Results for Streetman SEA-OGFC Mixtures

Asphalt Grade	o/w S		<u>Visual Observation at</u>					Brush Test Weight Loss (gm)
			0 Cycles	25 Cycles	50 Cycles	75 Cycles	100 Cycles	
AC-10	30	(1)	0	0	0	0	0	1.3
		(2)	0	0	0	0	0	1.2
		(3)	0	0	0	0	0	2.0
AC-20	20	(1)	0	0	0	3 - 1	3 - 2	1.3
		(2)	0	0	0	2 - 1	3 - 1	1.0
		(3)	0	0	0	3 - 1	3 - 2	1.2
AC-20	30	(1)	0	0	0	0	0	0.7
		(2)	0	0	0	0	0	0.5
		(3)	0	0	0	0	0	0.4
AC-20	40	(1)	0	0	0	0	2 - 1	0.7
		(2)	0	0	0	0	0	0.5
		(3)	0	0	0	3 - 1	3 - 1	1.2
AC-40	30	(1)	0	0	0	0	0	0.8
		(2)	0	0	0	2 - 1	2 - 1	2.4
		(3)	0	0	0	0	3 - 1	2.0

1 lb. = 454 gms

SUMMARY OF THE MAJOR STEPS REQUIRED BY THE GRADUATE  
COLLEGE IN FULFILLING REQUIREMENTS FOR THE  
MASTER'S DEGREE

Dean - Dean of Graduate College

GA - Graduate Advisor

D.A. & R. - Dean of Admissions and Records

ADCom - Advisory Committee

DH - Department Head

Procedure	Initiate Through	Time
	Approved by	
1. Apply for admission	D.A. & R. DH & Dean	At least 4 weeks prior to registration (3-4 months prior to registration for foreign students; use foreign student application)
2. Become familiar with general regulations and appropriate master's degree section of <u>Catalog</u>		Before registration
3. Meet with graduate advisor assigned by department head to plan course of study for first semester	DH & GA GA	Before first semester registration
4. Establish advisory committee; prepare degree program	GA & DH ADCom, DH & Dean	Before second semester registration
5. If thesis is required, submit thesis proposal	ADCom DH & Dean	14 weeks prior to graduation
6. Apply for degree	Dean	See deadlines in Graduate College Calendar
7. Check to be sure degree program and advisory committee are up to date and all course work is complete		Well before final examination. Follow regular procedures for changes (see <u>Catalog</u> )
8. Submit 3 approved copies of the thesis and announcement of the final examination	ADCom & DH Dean	At least 2 weeks prior to final oral examination (see <u>Catalog</u> for specific details)
9. File letter-of-intent to continue for doctorate, if you so plan	DH Dean & D.A. & R.	Immediately upon completion of all requirements for master's degree
10. Arrange for cap and gown		Texas A&M Bookstore in the University Center

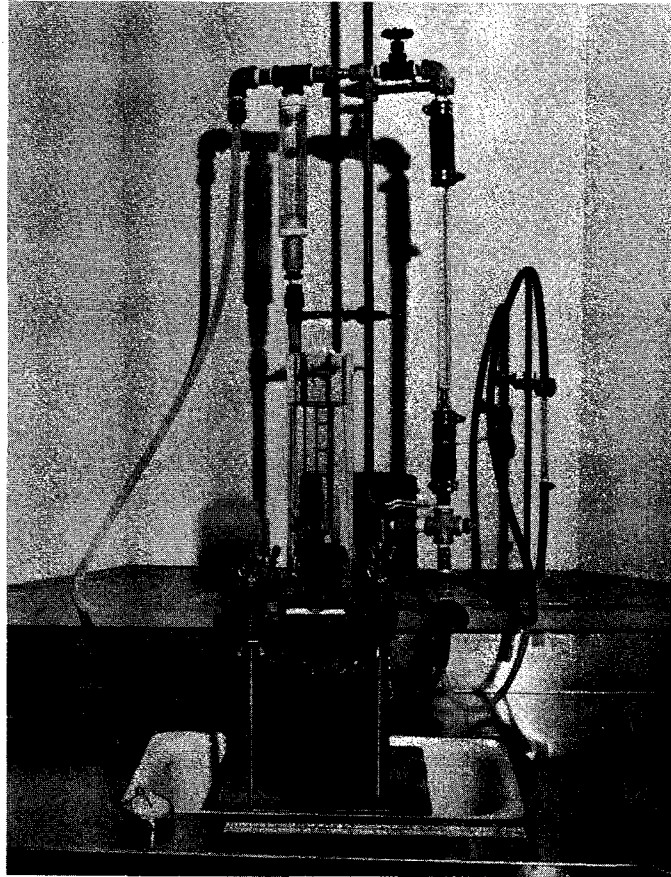


Figure E-1. Permeability Test Apparatus

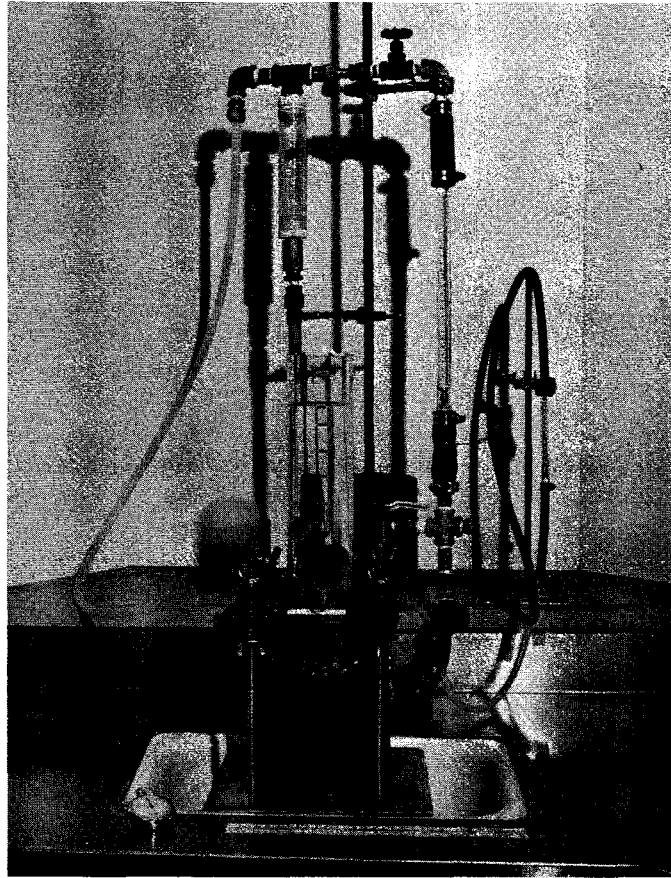


Figure E-1. Permeability Test Apparatus

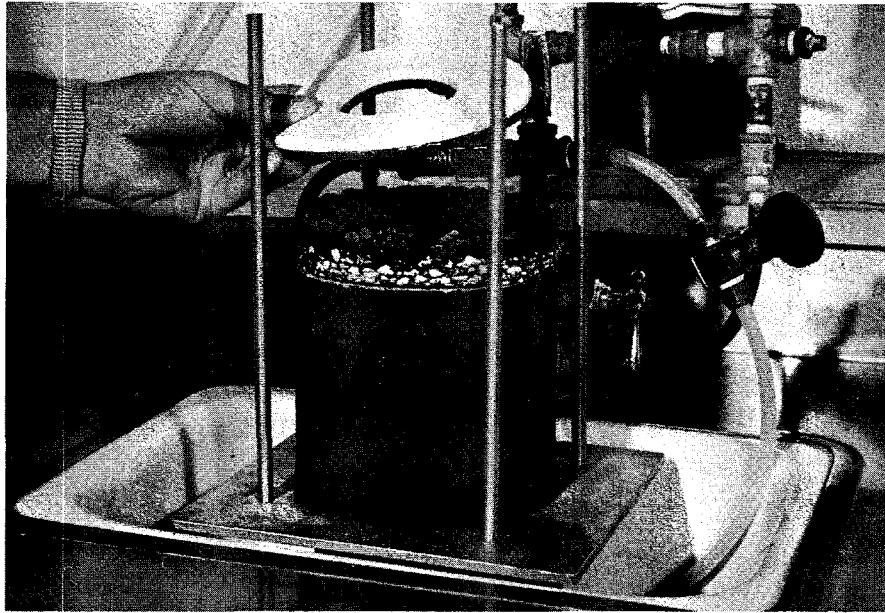


Figure E-4. Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

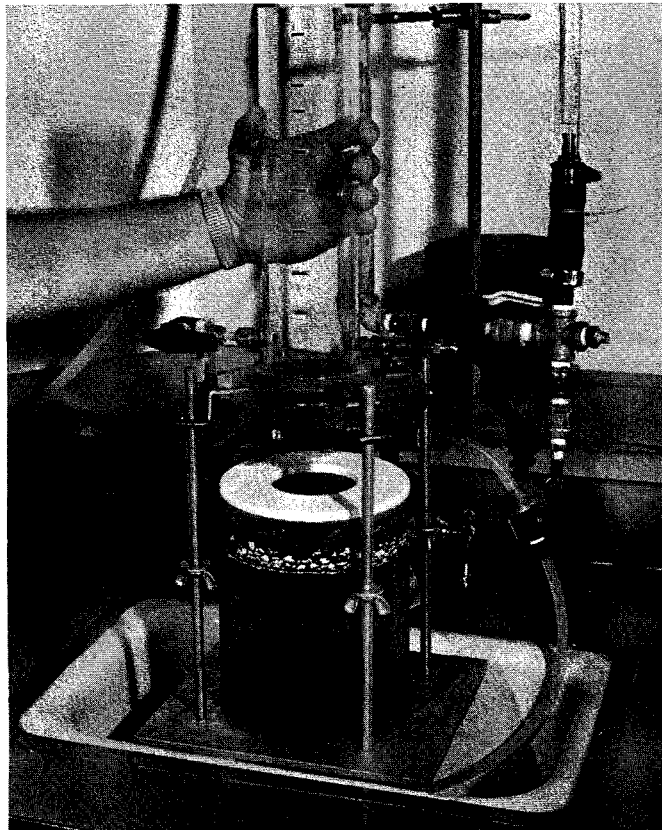


Figure E-5. Gasket in Place and Permeameter Assembly Being Lowered Into Position



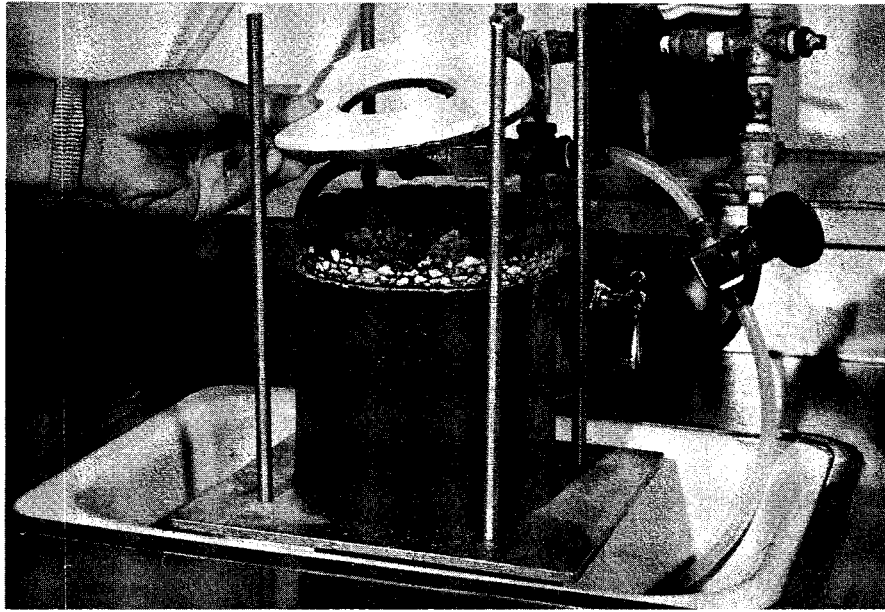


Figure E-4. Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied



Figure E-5. Gasket in Place and Permeameter Assembly Being Lowered Into Position

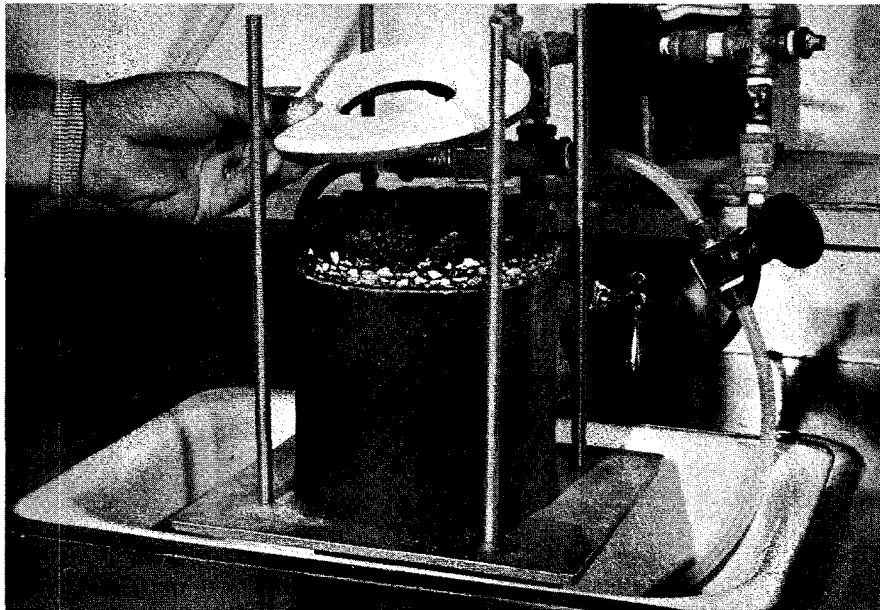


Figure E-4. Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

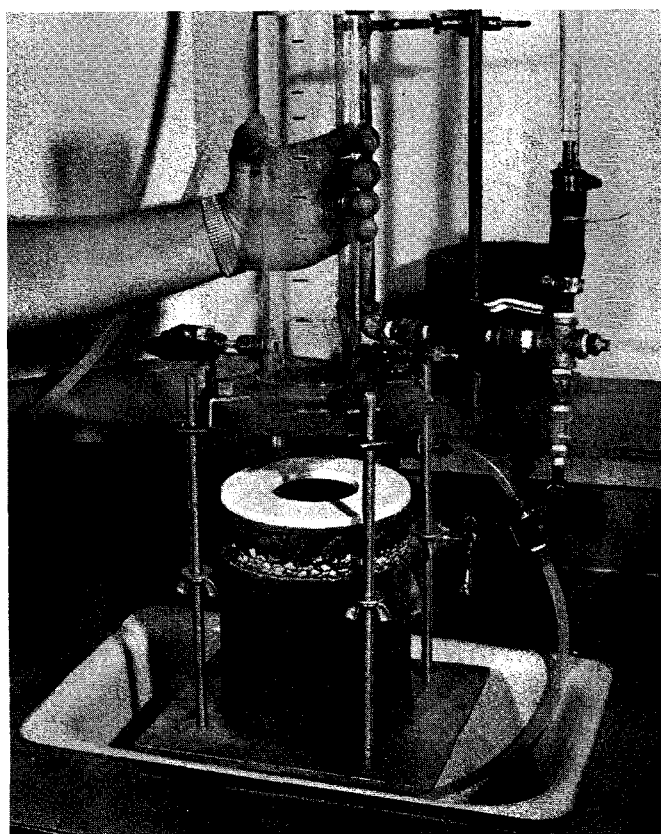


Figure E-5. Gasket in Place and Permeameter Assembly Being Lowered Into Position

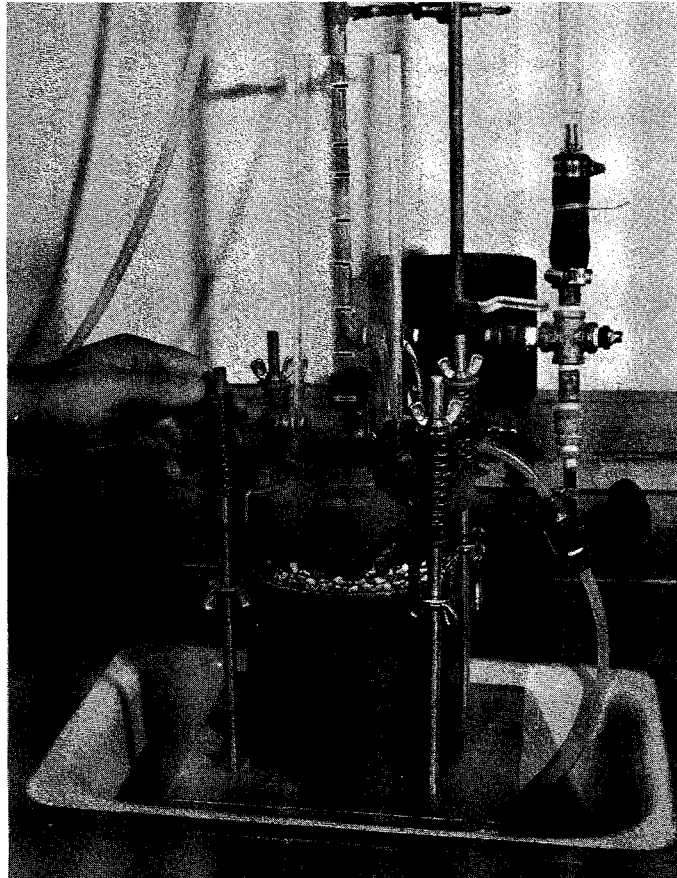


Figure E-6. Compression Springs Being Positioned on Support Rods

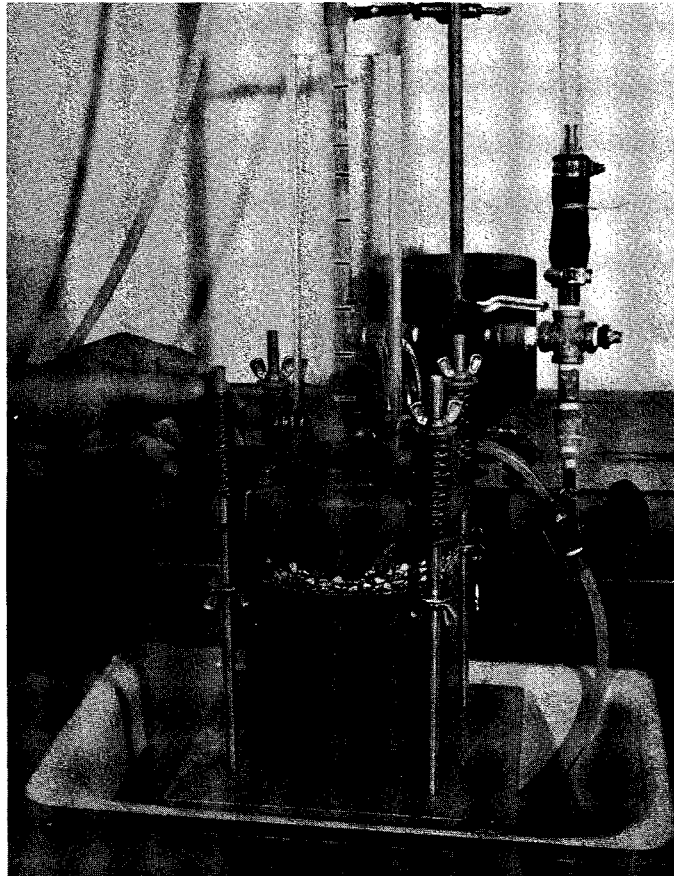


Figure E-6. Compression Springs Being Positioned on Support Rods

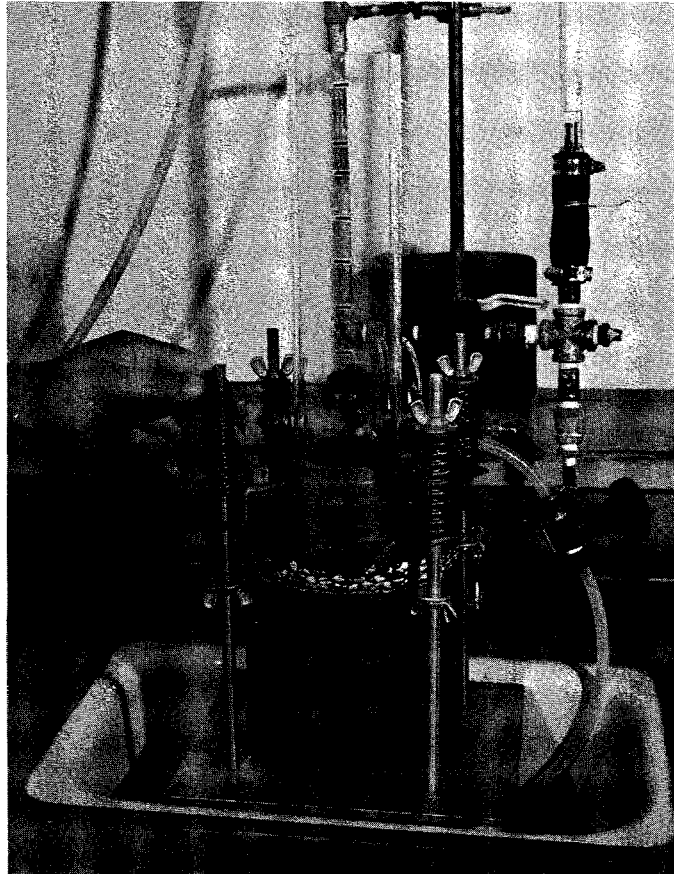


Figure E-6. Compression Springs Being Positioned on Support Rods

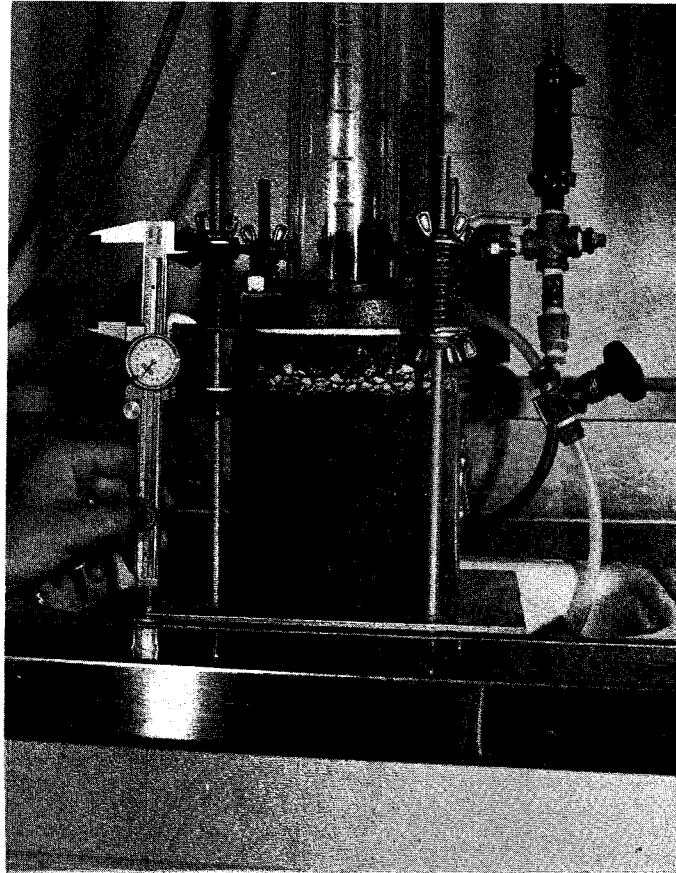


Figure E-7. Compression Springs in Place and Load Being Applied

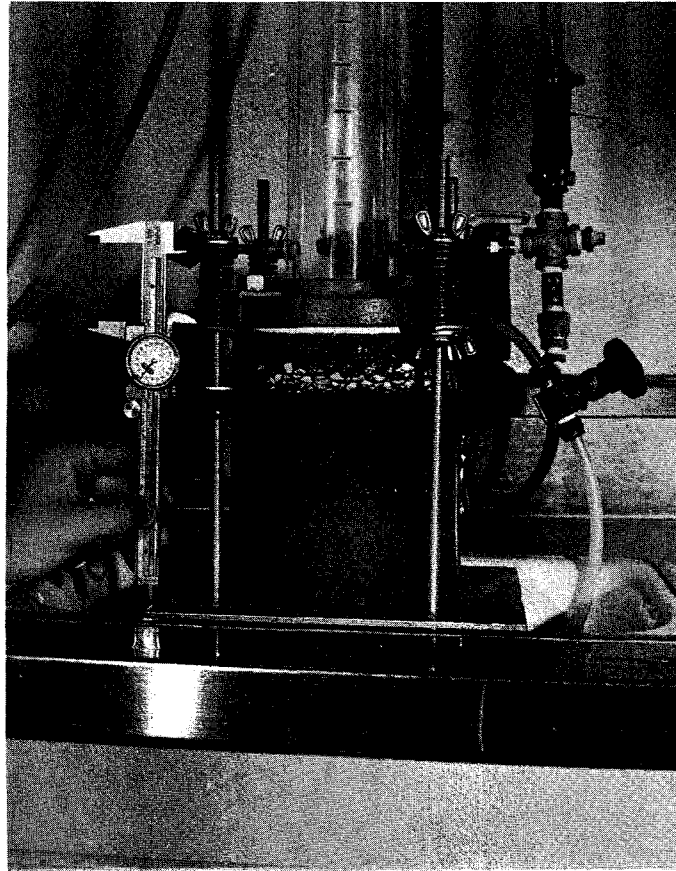


Figure E-7. Compression Springs in Place and Load Being Applied

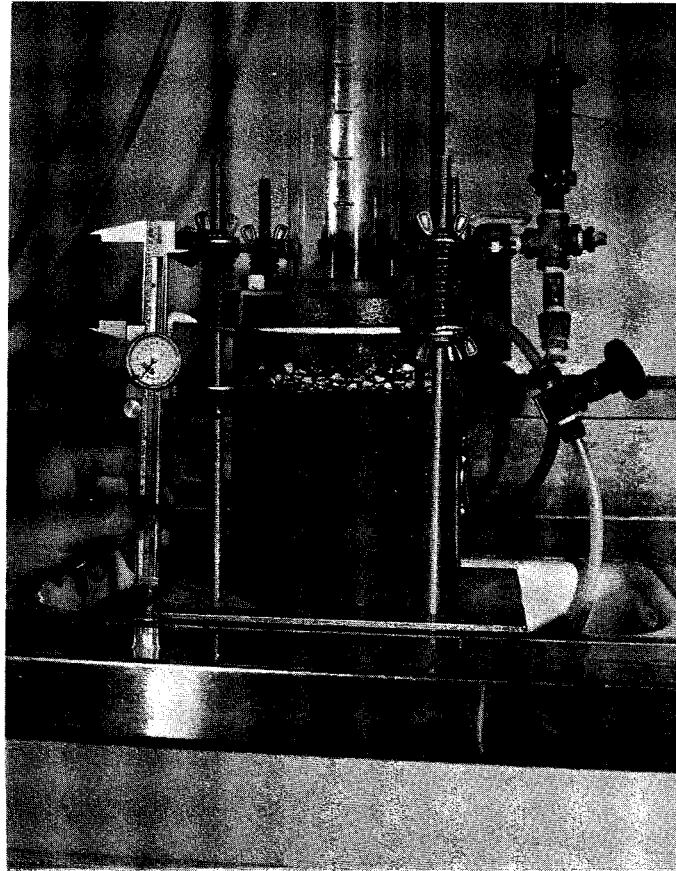


Figure E-7. Compression Springs in Place and Load Being Applied



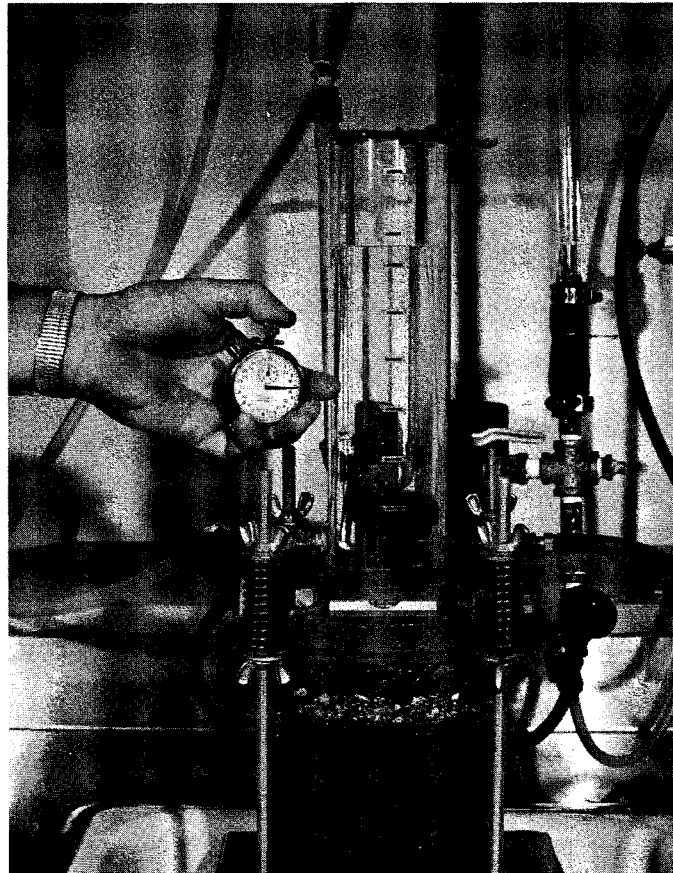


Figure E-8. Variable Head Permeability Test in Progress

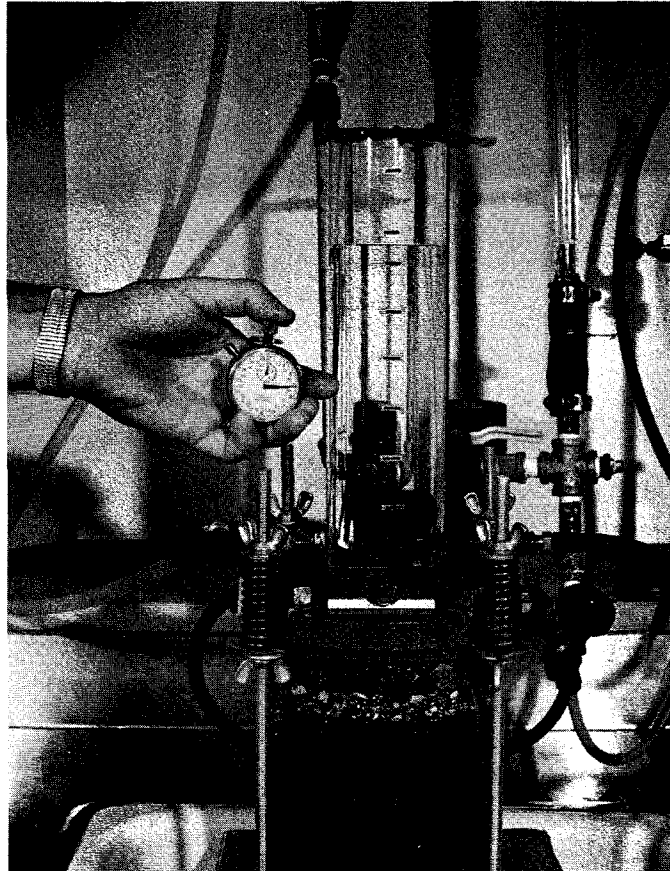


Figure E-8. Variable Head Permeability Test in Progress

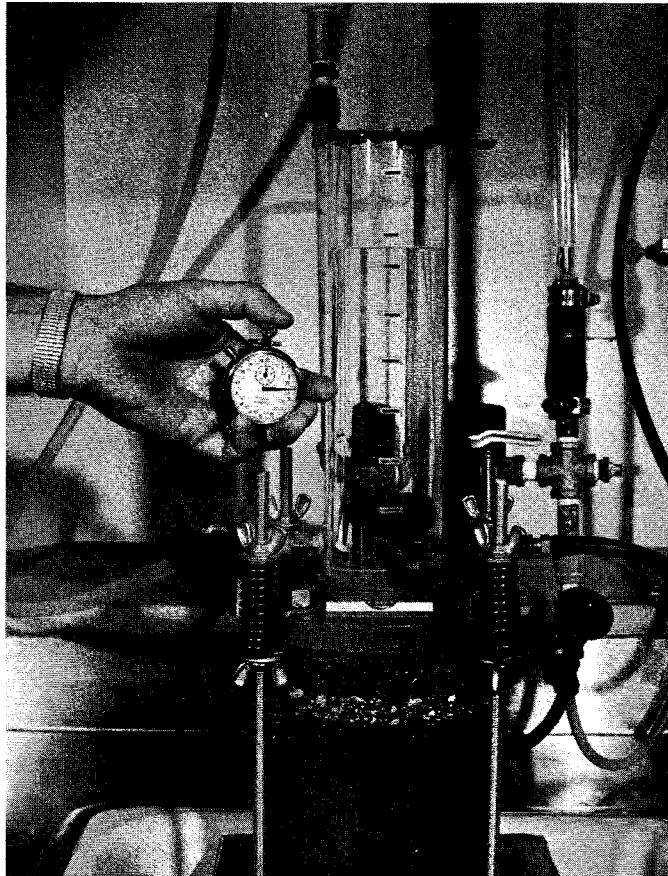


Figure E-8. Variable Head Permeability Test in Progress

GRADUATE COLLEGE CALENDAR, SPRING SEMESTER 1980

Failure to Meet Certain Requirements by the Time  
Specified May Result in the Postponement of  
Receipt of the Degree

January 9 - 11	Delayed registration.
January 14	Classes begin.
January 18	Last day for enrolling in the University or for adding new courses.
January 29	Last day for dropping courses with no record.
February 1	Last day for filing thesis proposals for master's degrees to be conferred in May 1980.
<u>February 1</u>	<u>Last day for filing for advanced degrees to be conferred on May 2, 1980.*</u>
March 7	Last day for scheduling with the Office of the Graduate College the final examinations for May, 1980, degrees which <u>require</u> theses, dissertations, or records of study. <u>Final examinations must be completed on or before March 31, 1980.</u>
March 17	Last day for dropping courses with no penalty (Q-drop).
March 31	Last day for filing 3 copies of theses, dissertations, or records of study (with all committee members' signatures and committee corrections) in final form with the Thesis/Dissertation Clerk in the Library for May graduation.
April 4	Last day for scheduling final examinations for May 1980 degrees which <u>do not require</u> theses, dissertations, or records of study (non-thesis degrees).
April 7	Last day for the Graduate Council Representative to file letter of approval of dissertation or record of study.
April 18	Last day for final examinations for May 1980 degrees which <u>do not require</u> theses, dissertations, or records of study.
April 25	Final corrections of theses, dissertations, or records of study required by the Graduate College must be certified by the Thesis/Dissertation Clerk by 5:00 p.m.
May 2	Commencement

\* \* \* \* \*

\*Applications for degree must be filed in the Office of the Graduate College. First, pay an \$11.00 diploma fee at the Fiscal Office (Coke Building) and take the receipt to the Office of the Graduate College (Room 125, Teague Building).

NOTE: Graduate students writing theses, dissertations, or records of study should, during the formative stages of these documents, consult the Thesis/Dissertation Clerk (in the Library) concerning style and format.

Item	North Carolina	FHWA Region 9 (Colorado, Wyoming, Utah and New Mexico)	FHWA Region 7 (California, Arizona, Nevada, Hawaii)	Franklin Institute	Louisiana																																																																				
Mix Designation	Bituminous Seal Coat	Open Graded Plant Mix Seal	Plant Mix Seal Coats	Open Graded Asphalt Concrete	Plant Mix Seal																																																																				
Aggregate Type	95% of material retained on No. 4 have at least one fractured face. Percent wear 45% or less (AASHTO T96)	Hard, durable, resistance to abrasion and stripping, sharp angular and polish resistant. Minimum 75% crushed.	Broken stone or crushed gravel with 90% by weight having at least one fractured face.	Same as California specifications.	Crushed gravel, slag or expanded clay. Maximum abrasion loss 45% (by LDH Designation TR11) for expanded clay.																																																																				
Aggregate Gradation	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>25-45</td></tr><tr><td>#10</td><td>0-10</td></tr><tr><td>#200</td><td>0-2</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	25-45	#10	0-10	#200	0-2	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>95-100</td></tr><tr><td>#4</td><td>30-50</td></tr><tr><td>#8</td><td>10-25</td></tr><tr><td>#16</td><td>0-18</td></tr><tr><td>#200</td><td>0-5</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	95-100	#4	30-50	#8	10-25	#16	0-18	#200	0-5	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>30-50</td></tr><tr><td>#8</td><td>15-32</td></tr><tr><td>#16</td><td>0-15</td></tr><tr><td>#200</td><td>0-3</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	30-50	#8	15-32	#16	0-15	#200	0-3	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>35-50</td></tr><tr><td>#8</td><td>15-32</td></tr><tr><td>#16</td><td>0-15</td></tr><tr><td>#200</td><td>0-3</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	35-50	#8	15-32	#16	0-15	#200	0-3	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>95-100</td></tr><tr><td>#4</td><td>30-55</td></tr><tr><td>#10</td><td>0-20</td></tr><tr><td>#40</td><td>0-12</td></tr><tr><td>#200</td><td>0-6</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	95-100	#4	30-55	#10	0-20	#40	0-12	#200	0-6
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	25-45																																																																								
#10	0-10																																																																								
#200	0-2																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	95-100																																																																								
#4	30-50																																																																								
#8	10-25																																																																								
#16	0-18																																																																								
#200	0-5																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	30-50																																																																								
#8	15-32																																																																								
#16	0-15																																																																								
#200	0-3																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	35-50																																																																								
#8	15-32																																																																								
#16	0-15																																																																								
#200	0-3																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	95-100																																																																								
#4	30-55																																																																								
#10	0-20																																																																								
#40	0-12																																																																								
#200	0-6																																																																								
Approximate * Cost per Ton	\$8.00 to \$15.00	\$7.00 to \$10.00	\$8.00 to \$12.00	None specified	None specified																																																																				
Asphalt Cement	60-70 penetration	60-70 penetration or 85-100 penetration	85-100 penetration	85-100 penetration	AC-40 (with 0.5% anti-stripping additive)																																																																				
Asphalt Content (percent)	6 to 10 actual value fixed by engineer	6 to 7	5 to 7	4.0 to 5.5	Crushed 4.0-10.0 Slag 6.0-12.0 Expanded clay 10.0-17.0																																																																				
Aggregate and Asphalt Temp. (°F) at Mixing	250 Maximum 300 maximum	260 to 300 260 to 300	290 290	--- ---	Mixing temperature of mix = 260° maximum.																																																																				
Stability, Flow and Voids	None specified	Retained stability 50% minimum by AASHTO T165	None specified	Exceed criterion for medium traffic uses (10-100 DTN)	None specified																																																																				
Remarks	Asphalt cement to contain 0.3% of: No Strip concentrate 380, Kling XX, Pave-bond 206, Kling HS-BETA-1000-3 or approved equal. No. 10 size aggregate limited to 10% to prevent bleeding.	Asphalt should be 200-300 centistokes at mixing temperature and ductility of 50 cm at 39°F. Stripping to be 95% retained coating (AASHTO T182).	Specifications are for California, Arizona and Nevada have similar specifications. No plant mix seal coat placed in Hawaii. Require film stripping test - 95% retained coating.	Mix with 5.5% asphalt most durable. Resistance to stripping by water (ASTM D1664) to be more than 95%. If less, add anti-stripping agent.	Placement temperature 180°F minimum. Hauling in excess of 20 miles may cause separation. A maximum of 260°F most critical for good mix.																																																																				

Item	North Carolina	FHWA Region 9 (Colorado, Wyoming, Utah and New Mexico)	FHWA Region 7 (California, Arizona, Nevada, Hawaii)	Franklin Institute	Louisiana
Mix Designation	Bituminous Seal Coat	Open Graded Plant Mix Seal	Plant Mix Seal Coats	Open Graded Asphalt Concrete	Plant Mix Seal
Aggregate Type	95% of material re- tained on No. 4 have at least one fractured face. Percent wear 45% or less (AASHTO T96)	Hard, durable, resistance to abrasion and stripping, sharp angular and polish resistant. Minimum 75% crushed.	Broken stone or crushed gravel with 90% by weight having at least one fractured face.	Same as California specifications.	Crushed gravel, slag or expanded clay. Maximum abrasion loss 45% (by LDH Designation TR11) for expanded clay.
Aggregate Gradation	<u>Sieve</u> Size	<u>Sieve</u> Size	<u>Sieve</u> Size	<u>Sieve</u> Size	<u>Sieve</u> Size
	Percent Passing	Percent Passing	Percent Passing	Percent Passing	Percent Passing
	1/2 100	1/2 100	1/2 100	1/2 100	1/2 100
	3/8 90-100	3/8 95-100	3/8 90-100	3/8 90-100	3/8 95-100
	#4 25-45	#4 30-50	#4 30-50	#4 35-50	#4 30-55
	#10 0-10	#8 10-25	#8 15-32	#8 15-32	#10 0-20
	#200 0-2	#16 0-18	#16 0-15	#16 0-15	#40 0-12
		#200 0-5	#200 0-3	#200 0-3	#200 0-6
Approximate * Cost per Ton	\$8.00 to \$15.00	\$7.00 to \$10.00	\$8.00 to \$12.00	None specified	None specified
Asphalt Cement	60-70 penetration	60-70 penetration or 85-100 penetration	85-100 penetration	85-100 penetration	AC-40 (with 0.5% anti- stripping additive)
Asphalt Content (percent)	6 to 10 actual value fixed by engineer	6 to 7	5 to 7	4.0 to 5.5	Crushed 4.0-10.0 Slag 6.0-12.0 Expanded clay 10.0-17.0
Aggregate and Asphalt Temp. (°F) at Mixing	250 Maximum 300 maximum	260 to 300 260 to 300	290 290	--- ---	Mixing temperature of mix = 260° maximum.
Stability, Flow and Voids	None specified	Retained stability 50% minimum by AASHTO T165	None specified	Exceed criterion for medium traffic uses (10- 100 DTN)	None specified
Remarks	Asphalt cement to con- tain 0.3% of: No Strip concentrate 380, Kling XX, Pave-bond 206, Kling HS-BETA-1000-3 or approved equal. No. 10 size aggregate limited to 10% to prevent bleeding.	Asphalt should be 200- 300 centistokes at mixing temperature and ductility of 50 cm at 39°F. Stripping to be 95% retained coat- ing (AASHTO T182).	Specifications are for California. Arizona and Nevada have simi- lar specifications. No plant mix seal coat placed in Hawaii. Require film stripping test - 95% retained coating.	Mix with 5.5% asphalt most durable. Resistance to stripping by water (ASTM D1664) to be more than 95%. If less, add anti- stripping agent.	Placement temperature 180°F minimum. Hauling in excess of 20 miles may cause separation. A maximum of 260°F most critical for good mix.

Item	North Carolina	FHWA Region 9 (Colorado, Wyoming, Utah and New Mexico)	FHWA Region 7 (California, Arizona, Nevada, Hawaii)	Franklin Institute	Louisiana																																																																				
Mix Designation	Bituminous Seal Coat	Open Graded Plant Mix Seal	Plant Mix Seal Coats	Open Graded Asphalt Concrete	Plant Mix Seal																																																																				
Aggregate Type	95% of material retained on No. 4 have at least one fractured face. Percent wear 45% or less (AASHTO T96)	Hard, durable, resistance to abrasion and stripping, sharp angular and polish resistant. Minimum 75% crushed.	Broken stone or crushed gravel with 90% by weight having at least one fractured face.	Same as California specifications.	Crushed gravel, slag or expanded clay. Maximum abrasion loss 45% (by LDH Designation TR11) for expanded clay.																																																																				
Aggregate Gradation	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>25-45</td></tr><tr><td>#10</td><td>0-10</td></tr><tr><td>#200</td><td>0-2</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	25-45	#10	0-10	#200	0-2	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>95-100</td></tr><tr><td>#4</td><td>30-50</td></tr><tr><td>#8</td><td>10-25</td></tr><tr><td>#16</td><td>0-18</td></tr><tr><td>#200</td><td>0-5</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	95-100	#4	30-50	#8	10-25	#16	0-18	#200	0-5	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>30-50</td></tr><tr><td>#8</td><td>15-32</td></tr><tr><td>#16</td><td>0-15</td></tr><tr><td>#200</td><td>0-3</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	30-50	#8	15-32	#16	0-15	#200	0-3	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>90-100</td></tr><tr><td>#4</td><td>35-50</td></tr><tr><td>#8</td><td>15-32</td></tr><tr><td>#16</td><td>0-15</td></tr><tr><td>#200</td><td>0-3</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	90-100	#4	35-50	#8	15-32	#16	0-15	#200	0-3	<table><tr><td>Sieve Size</td><td>Percent Passing</td></tr><tr><td>1/2</td><td>100</td></tr><tr><td>3/8</td><td>95-100</td></tr><tr><td>#4</td><td>30-55</td></tr><tr><td>#10</td><td>0-20</td></tr><tr><td>#40</td><td>0-12</td></tr><tr><td>#200</td><td>0-6</td></tr></table>	Sieve Size	Percent Passing	1/2	100	3/8	95-100	#4	30-55	#10	0-20	#40	0-12	#200	0-6
	Sieve Size	Percent Passing																																																																							
	1/2	100																																																																							
	3/8	90-100																																																																							
	#4	25-45																																																																							
	#10	0-10																																																																							
#200	0-2																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	95-100																																																																								
#4	30-50																																																																								
#8	10-25																																																																								
#16	0-18																																																																								
#200	0-5																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	30-50																																																																								
#8	15-32																																																																								
#16	0-15																																																																								
#200	0-3																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	90-100																																																																								
#4	35-50																																																																								
#8	15-32																																																																								
#16	0-15																																																																								
#200	0-3																																																																								
Sieve Size	Percent Passing																																																																								
1/2	100																																																																								
3/8	95-100																																																																								
#4	30-55																																																																								
#10	0-20																																																																								
#40	0-12																																																																								
#200	0-6																																																																								
Approximate * Cost per Ton	\$8.00 to \$15.00	\$7.00 to \$10.00	\$8.00 to \$12.00	None specified	None specified																																																																				
Asphalt Cement	60-70 penetration	60-70 penetration or 85-100 penetration	85-100 penetration	85-100 penetration	AC-40 (with 0.5% anti-stripping additive)																																																																				
Asphalt Content (percent)	6 to 10 actual value fixed by engineer	6 to 7	5 to 7	4.0 to 5.5	Crushed 4.0-10.0 Slag 6.0-12.0 Expanded clay 10.0-17.0																																																																				
Aggregate and Asphalt Temp. (°F) at Mixing	250 Maximum 300 maximum	260 to 300 260 to 300	290 290	--- ---	Mixing temperature of mix = 260° maximum.																																																																				
Stability, Flow and Voids	None specified	Retained stability 50% minimum by AASHTO T165	None specified	Exceed criterion for medium traffic uses (10-100 DTN)	None specified																																																																				
Remarks	Asphalt cement to contain 0.3% of: No Strip concentrate 380, Kling XX, Pave-bond 206, Kling HS-BETA-1000-3 or approved equal. No. 10 size aggregate limited to 10% to prevent bleeding.	Asphalt should be 200-300 centistokes at mixing temperature and ductility of 50 cm at 39°F. Stripping to be 95% retained coating (AASHTO T182).	Specifications are for California, Arizona and Nevada have similar specifications. No plant mix seal coat placed in Hawaii. Require film stripping test - 95% retained coating.	Mix with 5.5% asphalt most durable. Resistance to stripping by water (ASTM D1664) to be more than 95%. If less, add anti-stripping agent.	Placement temperature 180°F minimum. Hauling in excess of 20 miles may cause separation. A maximum of 260°F most critical for good mix.																																																																				

# TEXAS A&M UNIVERSITY

COLLEGE STATION, TEXAS 77843



Liberal Arts — Cooperative Education  
Telephone 713/845-7814

September 17, 1981

## MEMORANDUM

TO: Students of all majors  
College of Liberal Arts

FROM: Henry D. Pope and Susannah R. Lindsay  
Cooperative Education Office

SUBJECT: Official Notice for CO-OP placements in the College of Liberal Arts

ATTENTION!  
CO-OP EDUCATION  
College of Liberal Arts

### HAS CAREER OPPORTUNITIES AVAILABLE WITH THE FOLLOWING EMPLOYERS

BROWN&ROOT: HOUSTON, TEXAS	
JOB DESCRIPTION:	PERSONNEL WORK
CORPS OF ENGINEERS: GALVESTON, TEXAS	
JOB DESCRIPTION:	ECONOMIC PLANNING
IBM CORPORATION: GENERAL SYSTEMS DIVISON (TEXAS)	
JOB DESCRIPTION:	SALES AND MARKETING
FOLEY'S: HOUSTON, TEXAS	
JOB DESCRIPTION:	MANAGEMENT TRAINEE
NATIONAL AERONAUTICS&SPACE ADMINISTRATION: HOUSTON, TEXAS	
JOB DESCRIPTION:	MANAGEMENT TRAINEE
LEGAL CO-OP: HOUSTON, DALLAS, AUSTIN, AND SAN ANTONIO, TEXAS	
JOB DESCRIPTION:	LEGAL ASSISTANT
BRAZOS COUNTY AND BURLESON COUNTY PROBATION OFFICES	
JOB DESCRIPTION:	ASSISTANT PROBATION OFFICER

FOR INFORMATION COME TO: 107 HARRINGTON TOWER  
PHONE: 845-7814



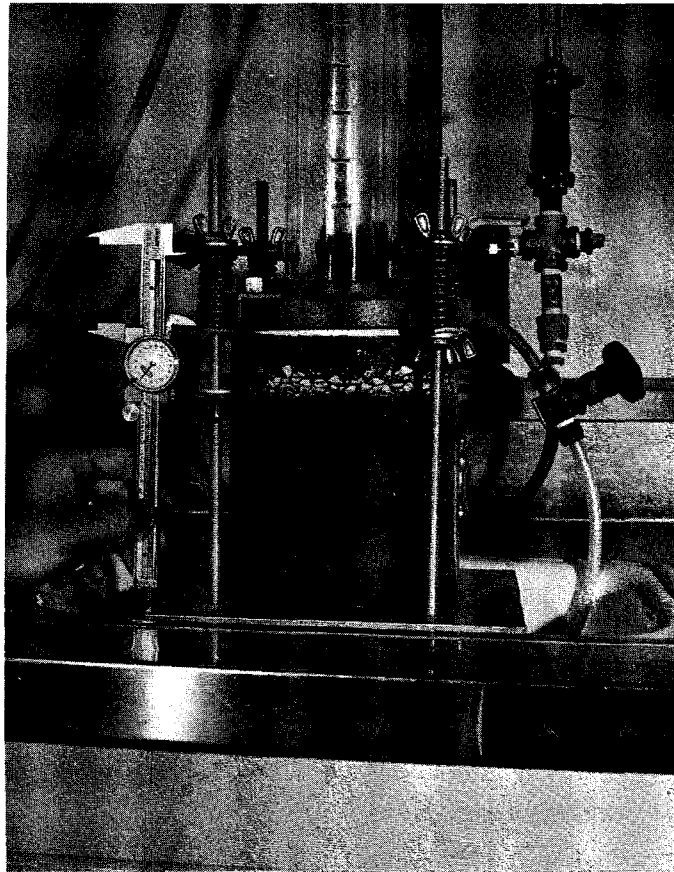


Figure                      Compression Springs in Place and Load Being Applied

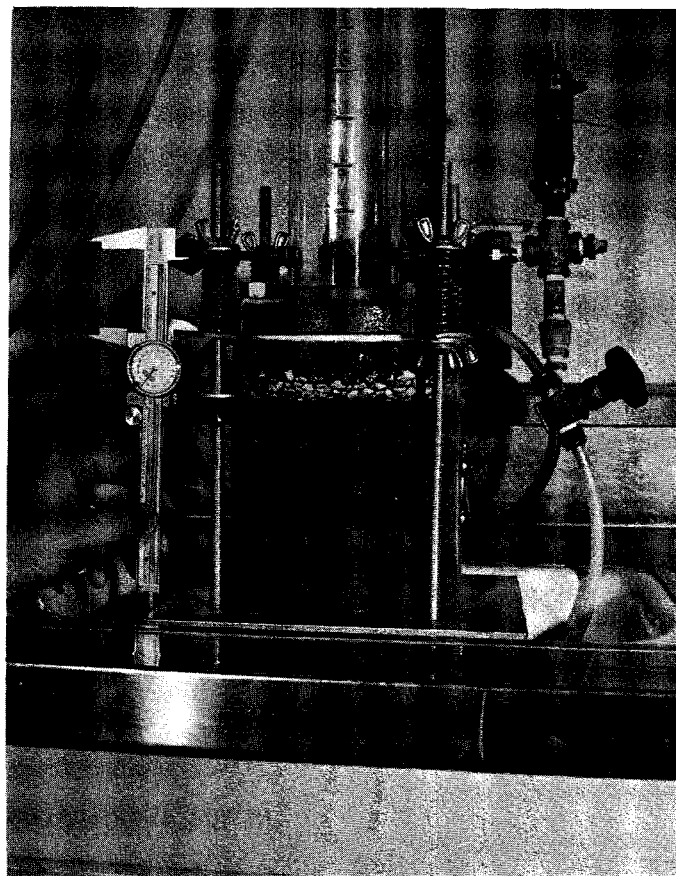


Figure            Compression Springs in Place and Load Being Applied

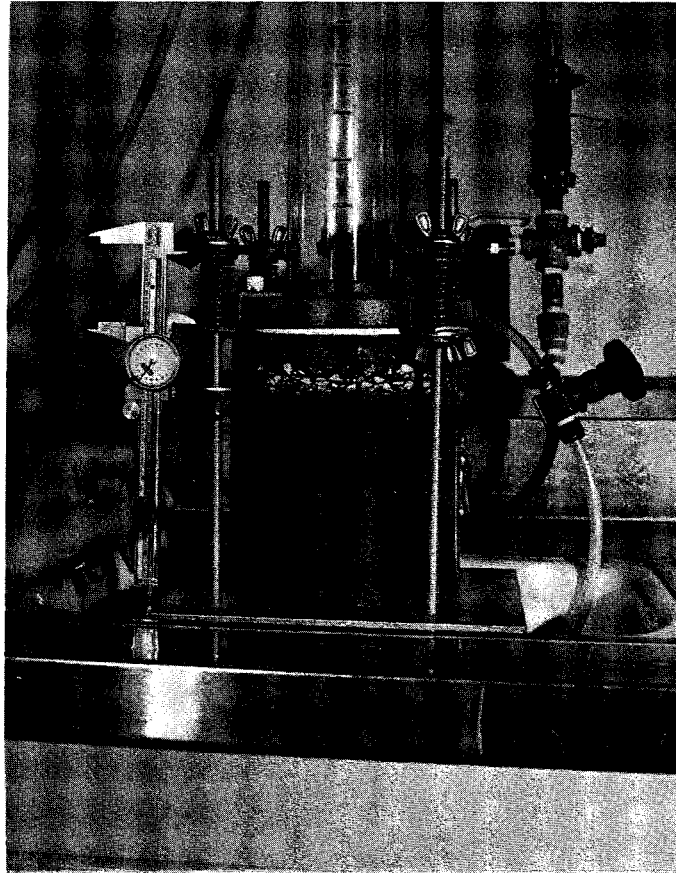


Figure            Compression Springs in Place and Load Being Applied

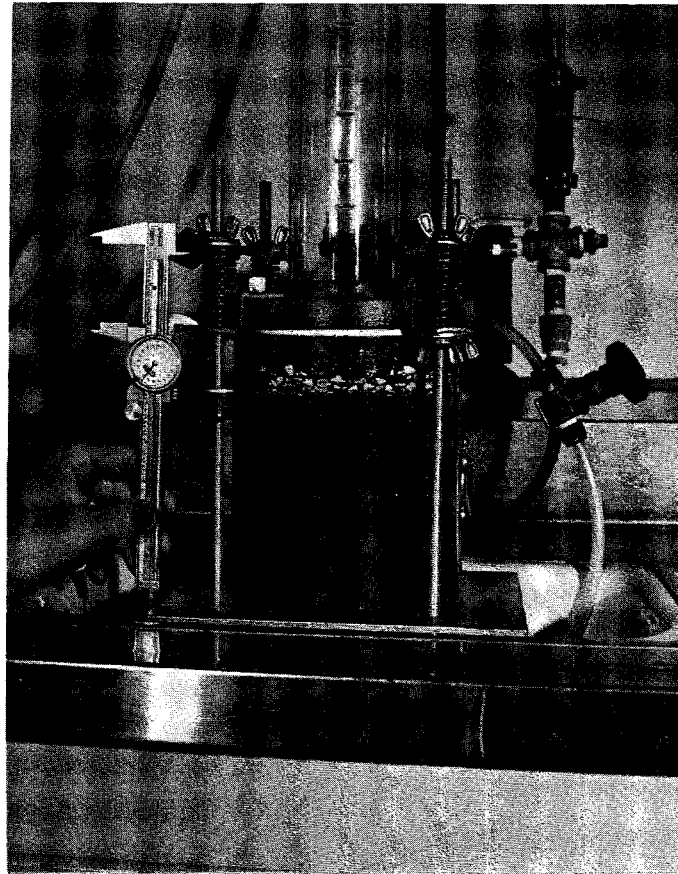


Figure                      Compression Springs in Place and Load Being Applied

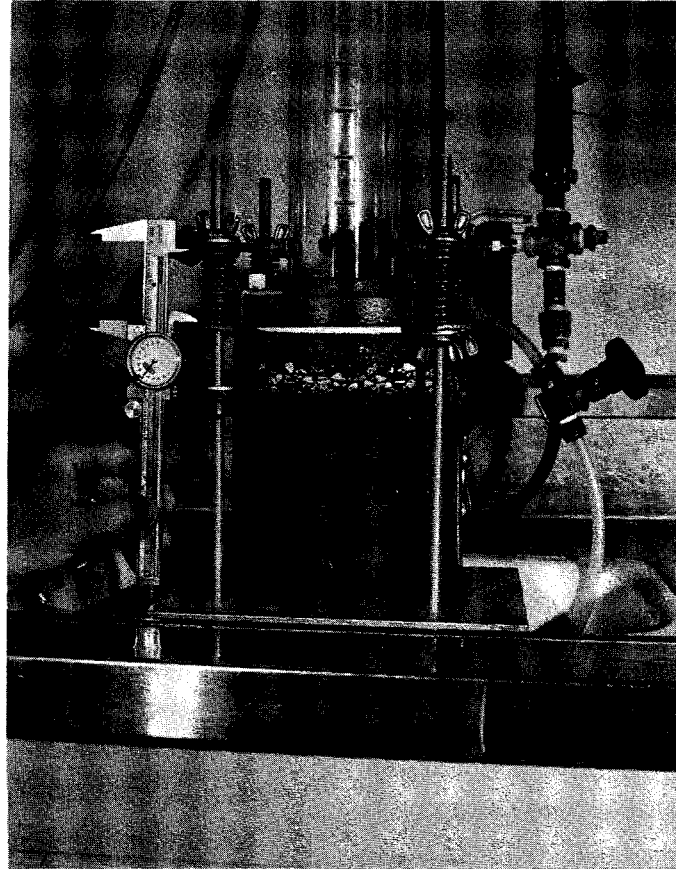


Figure            Compression Springs in Place and Load Being Applied

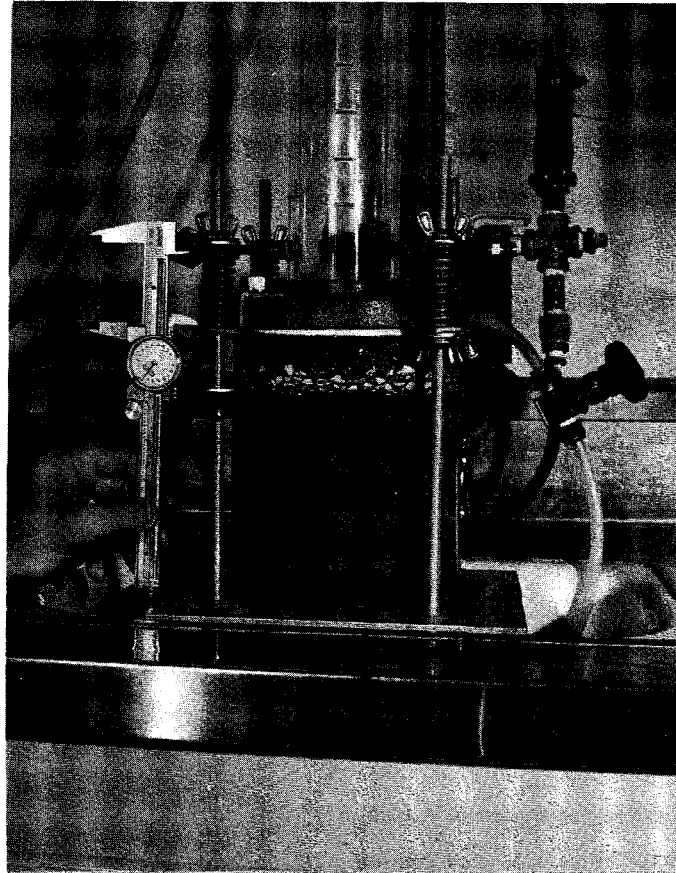


Figure                      Compression Springs in Place and Load Being Applied

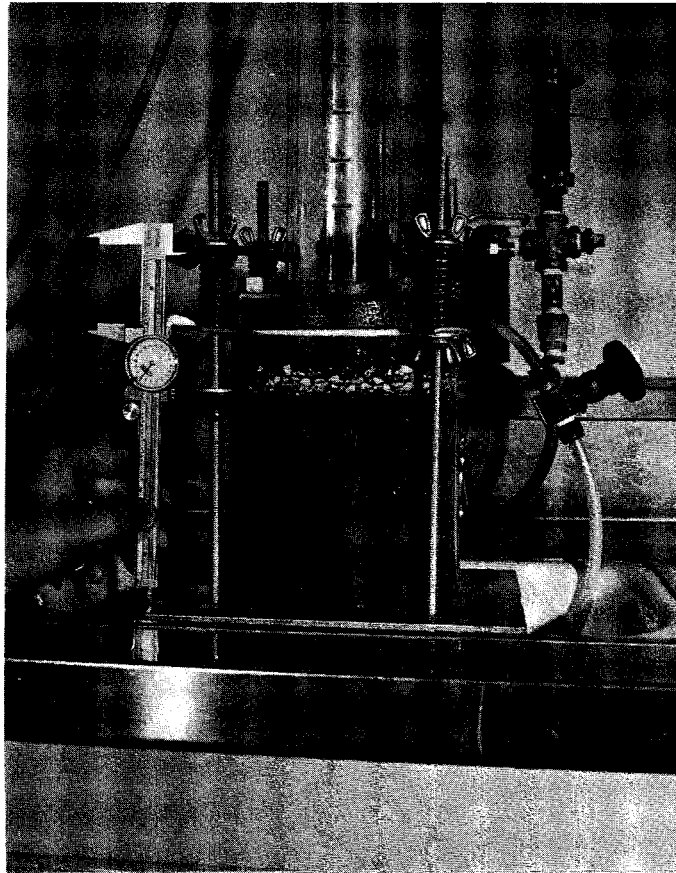


Figure            Compression Springs in Place and Load Being Applied

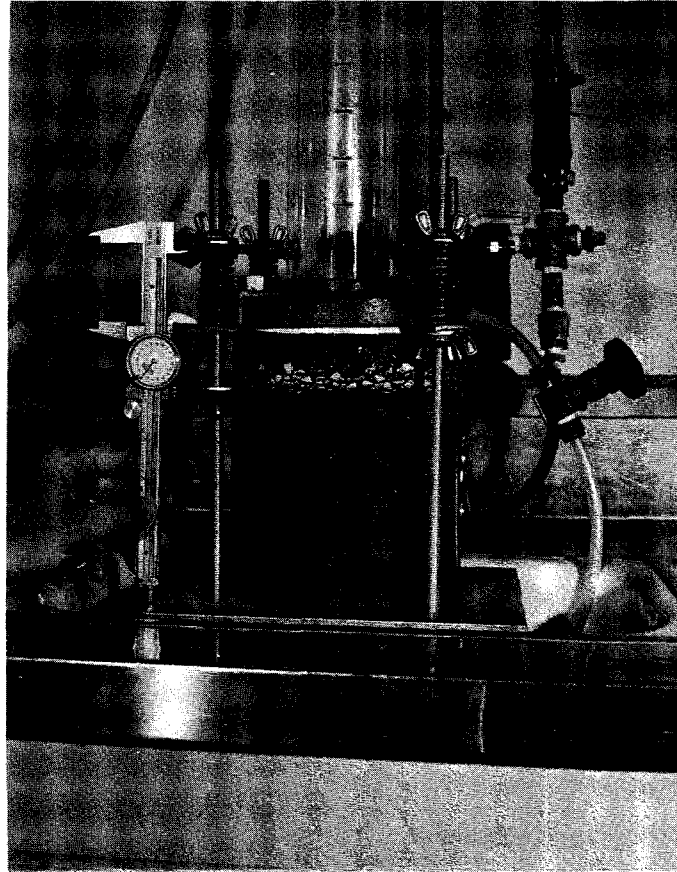


Figure            Compression Springs in Place and Load Being Applied



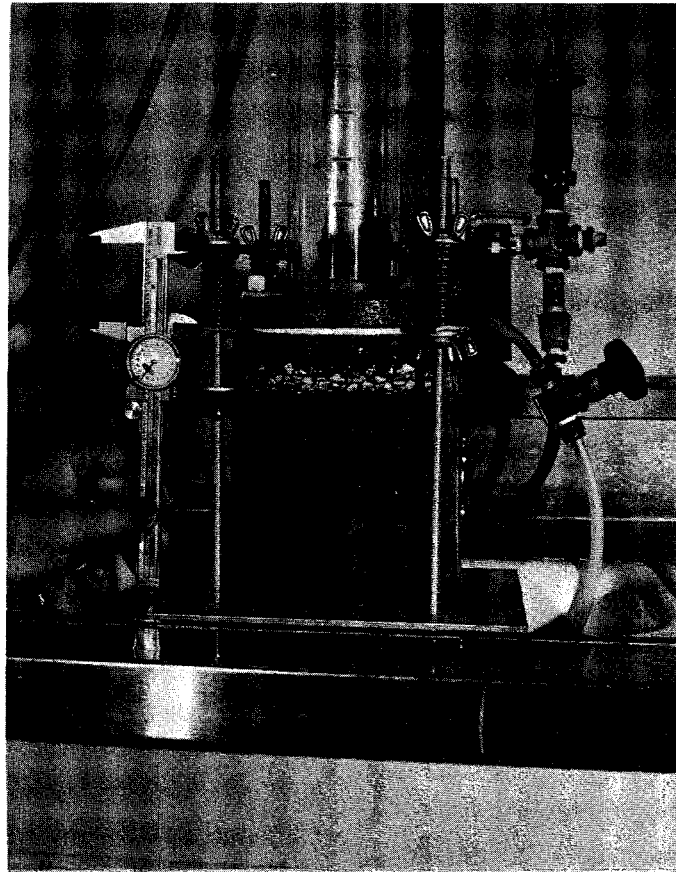


Figure            Compression Springs in Place and Load Being Applied

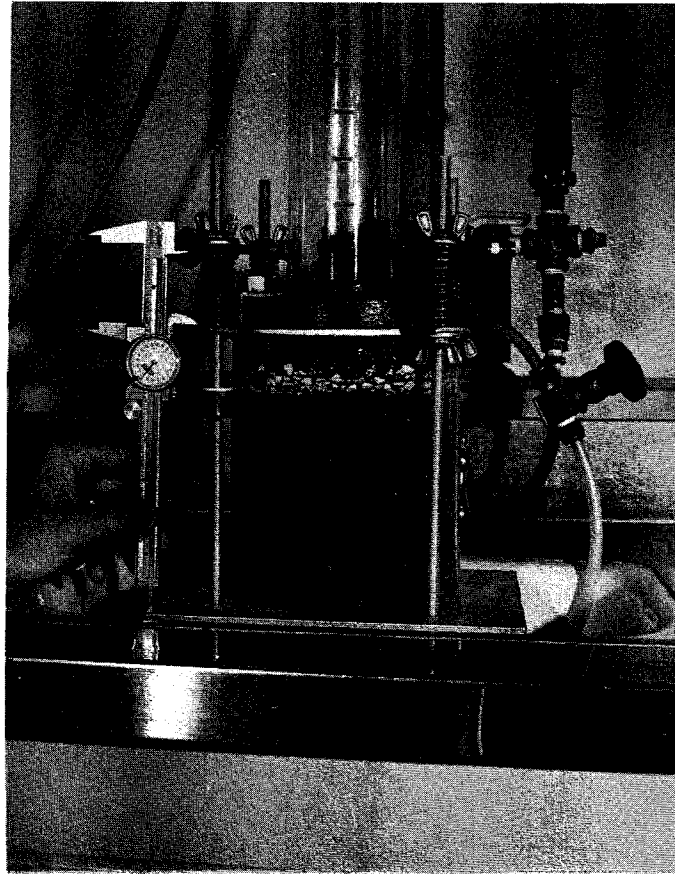


Figure                      Compression Springs in Place and Load Being Applied

# TEXAS A&M UNIVERSITY

COLLEGE STATION, TEXAS 77843



Liberal Arts — Cooperative Education  
Telephone 713/845-7814

September 17, 1981

## MEMORANDUM

TO: Students of all majors  
College of Liberal Arts

FROM: Henry D. Pope and Susannah R. Lindsay  
Cooperative Education Office

SUBJECT: Official Notice for CO-OP placements in the College of Liberal Arts

ATTENTION!  
CO-OP EDUCATION  
College of Liberal Arts

HAS CAREER OPPORTUNITIES AVAILABLE WITH THE FOLLOWING EMPLOYERS

BROWN&ROOT: HOUSTON, TEXAS	JOB DESCRIPTION:	PERSONNEL WORK
CORPS OF ENGINEERS: GALVESTON, TEXAS	JOB DESCRIPTION:	ECONOMIC PLANNING
IBM CORPORATION: GENERAL SYSTEMS DIVISON (TEXAS)	JOB DESCRIPTION:	SALES AND MARKETING
FOLEY'S: HOUSTON, TEXAS	JOB DESCRIPTION:	MANAGEMENT TRAINEE
NATIONAL AERONAUTICS&SPACE ADMINISTRATION: HOUSTON, TEXAS	JOB DESCRIPTION:	MANAGEMENT TRAINEE
LEGAL CO-OP: HOUSTON, DALLAS, AUSTIN, AND SAN ANTONIO, TEXAS	JOB DESCRIPTION:	LEGAL ASSISTANT
BRAZOS COUNTY AND BURLESON COUNTY PROBATION OFFICES	JOB DESCRIPTION:	ASSISTANT PROBATION OFFICER

FOR INFORMATION COME TO: 107 HARRINGTON TOWER  
PHONE: 845-7814

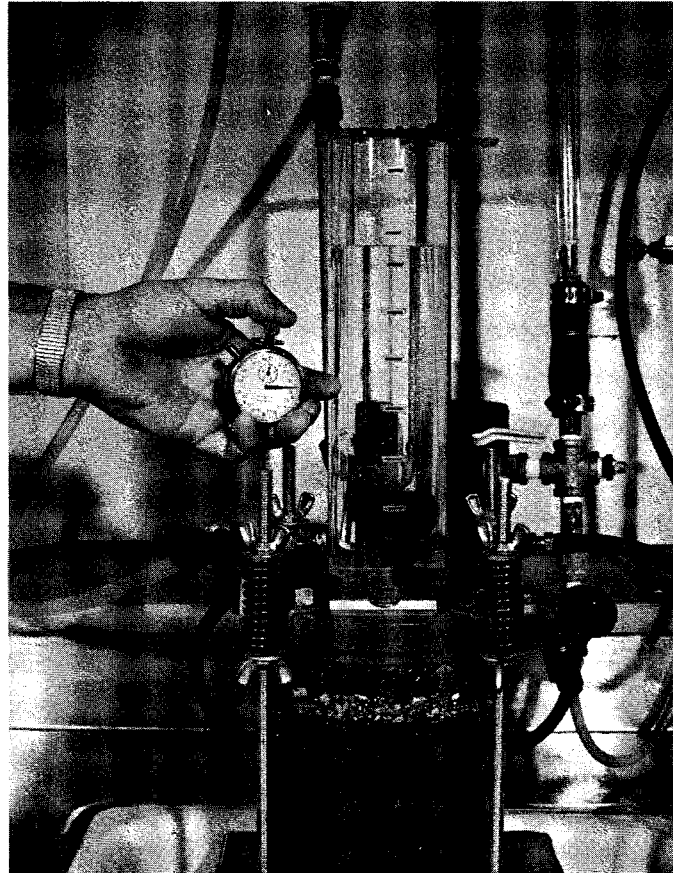
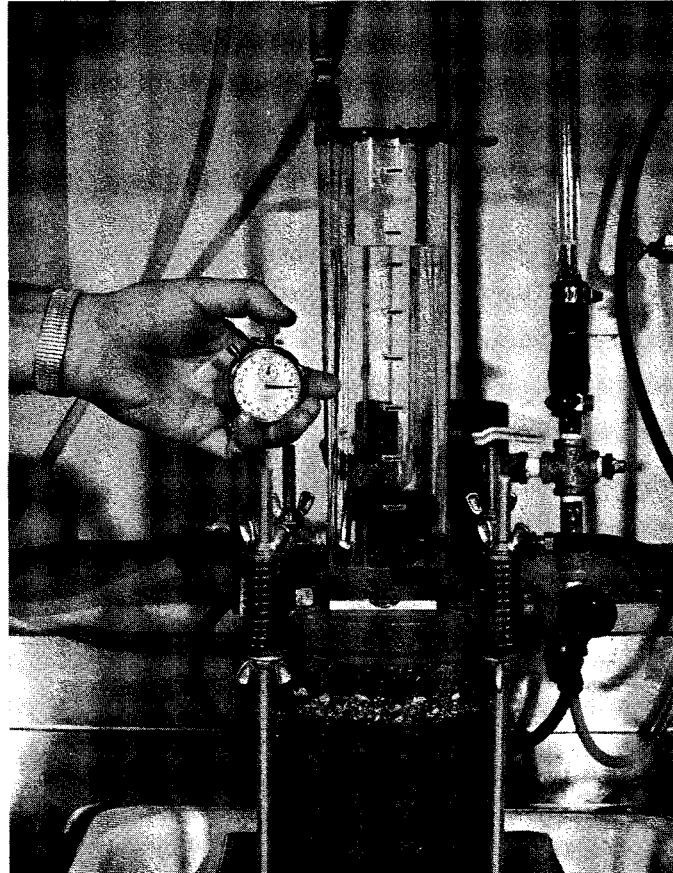


Figure      Variable Head Permeability Test in Progress



Figure

Variable Head Permeability Test in Progress

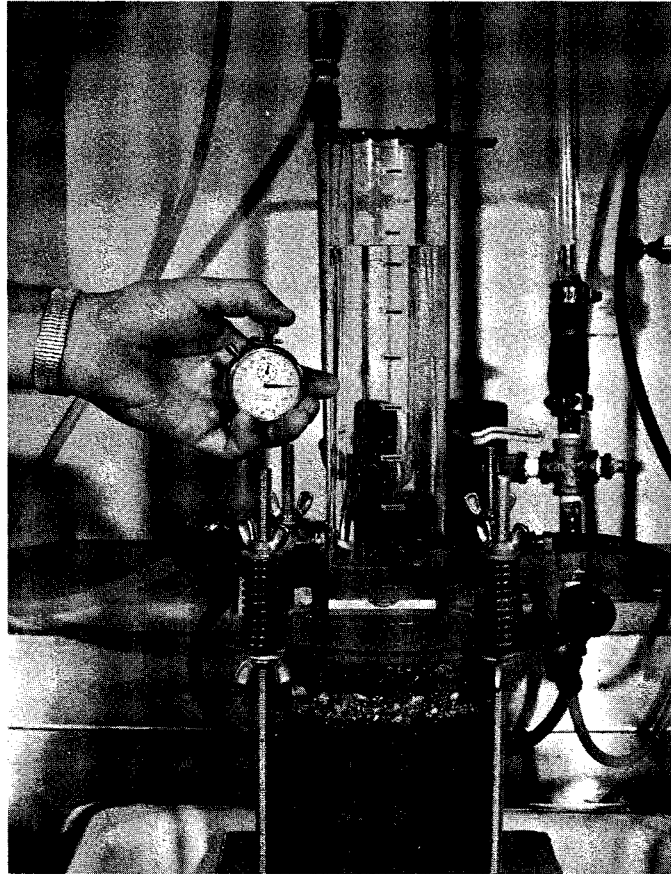


Figure      Variable Head Permeability Test in Progress

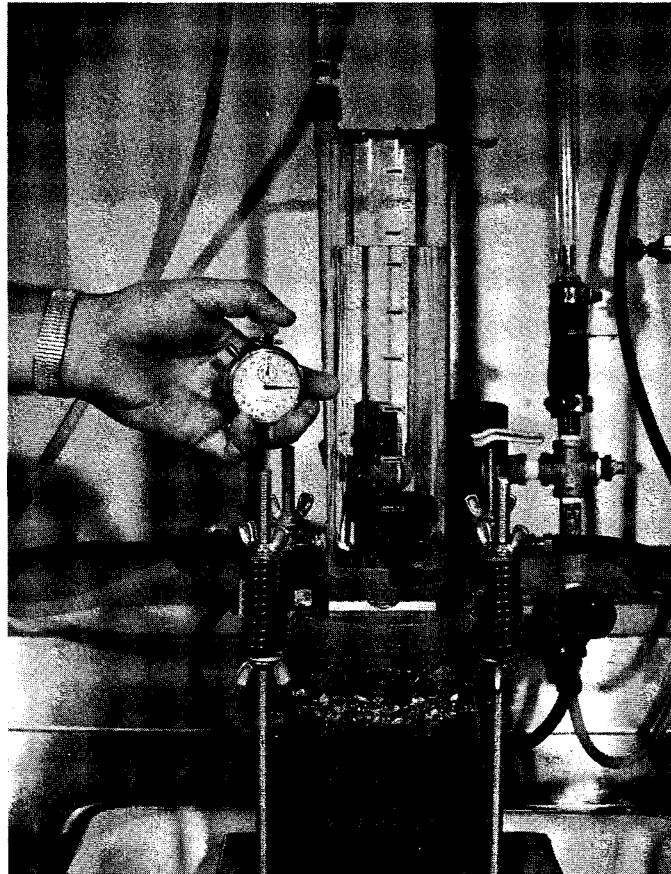


Figure      Variable Head Permeability Test in Progress

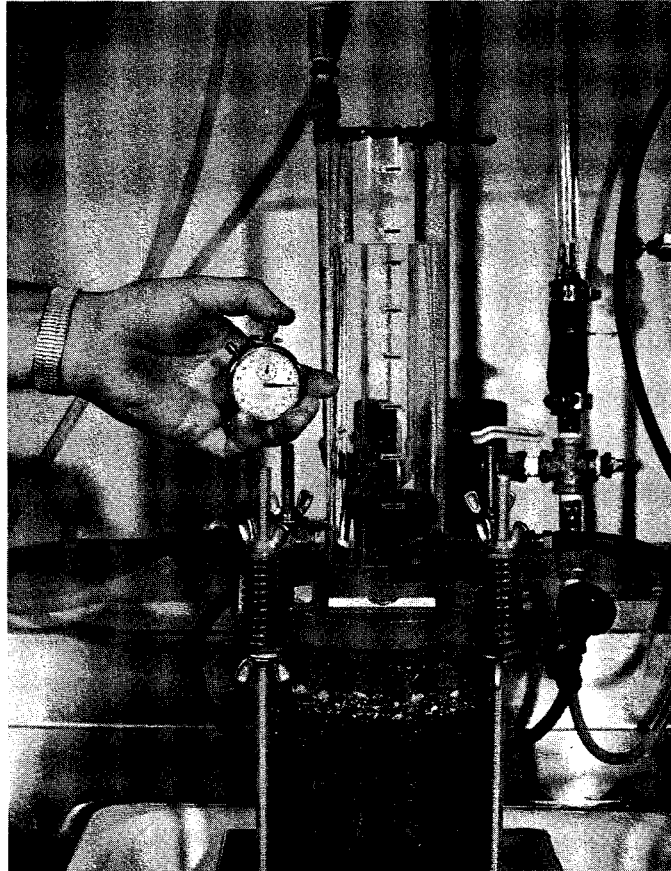


Figure      Variable Head Permeability Test in Progress



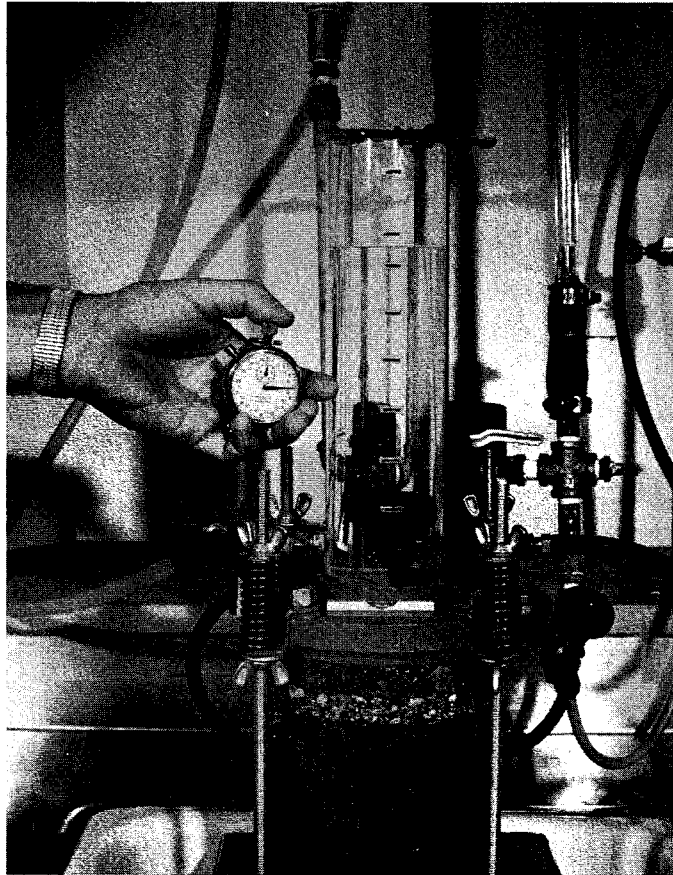


Figure      Variable Head Permeability Test in Progress

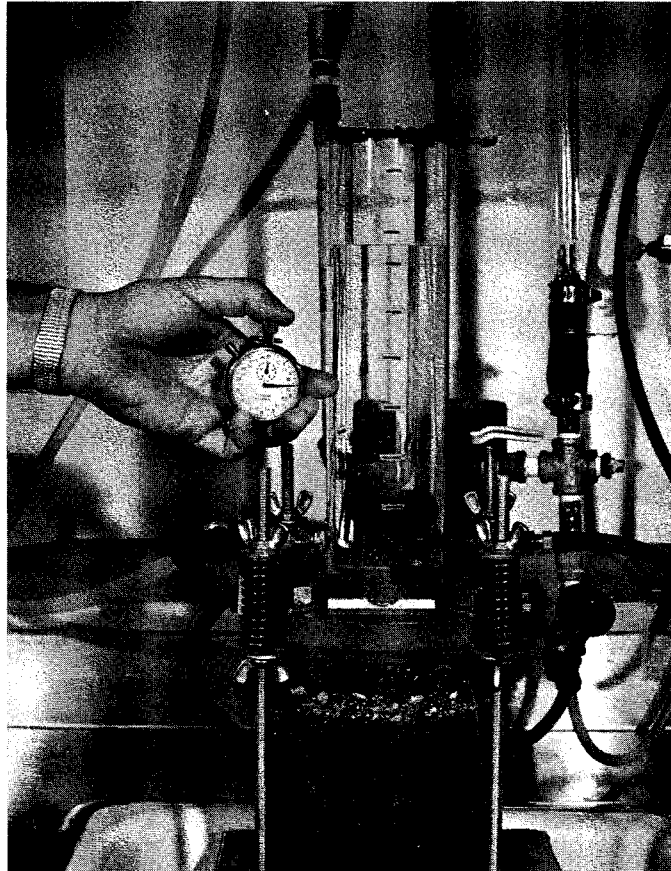


Figure            Variable Head Permeability Test in Progress

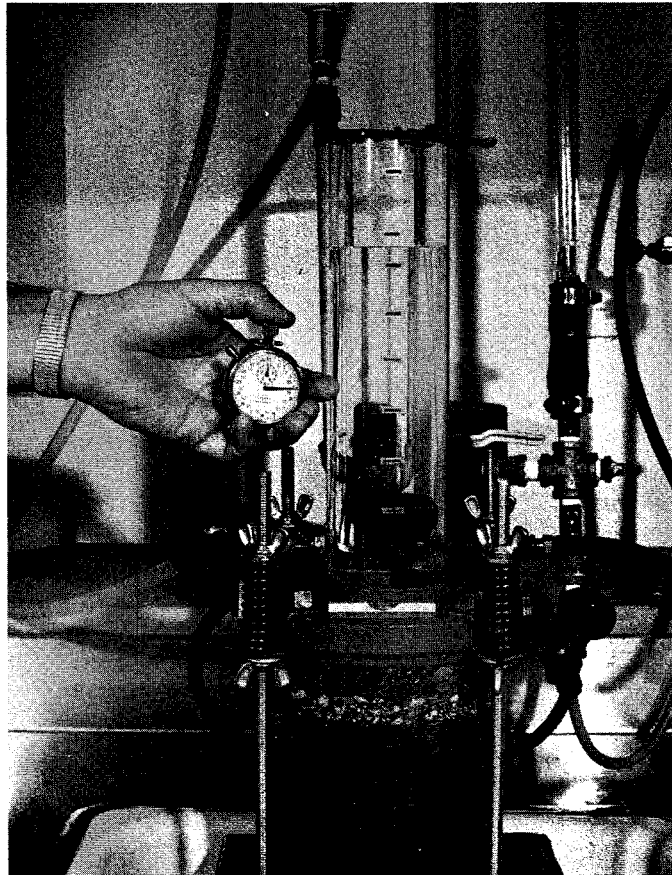


Figure      Variable Head Permeability Test in Progress

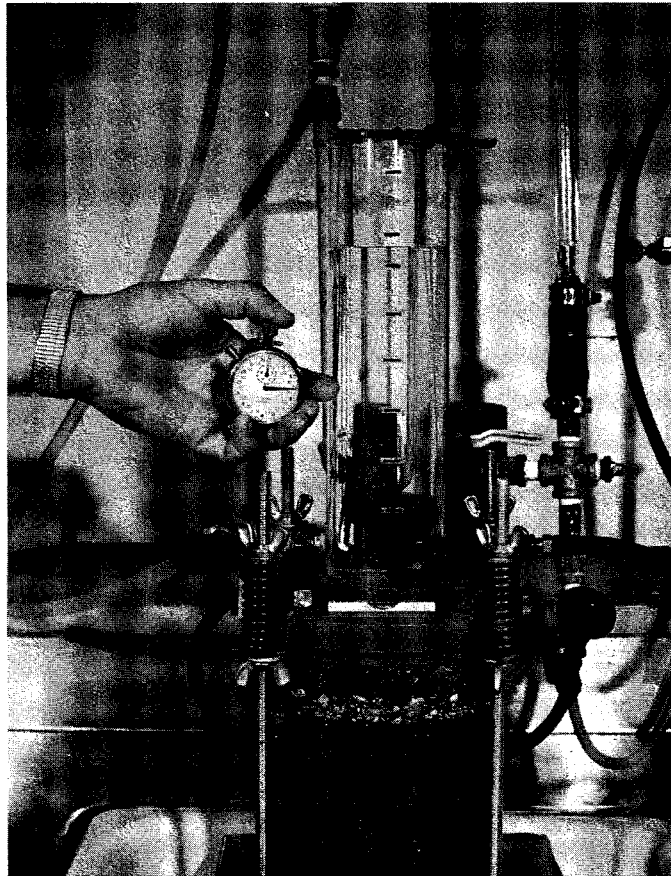
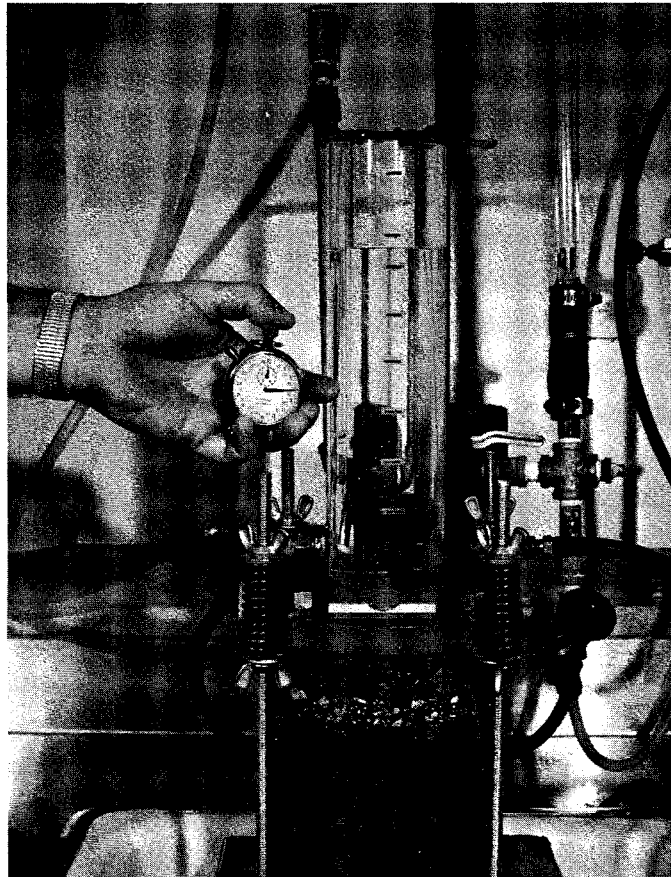


Figure      Variable Head Permeability Test in Progress



Figure

Variable Head Permeability Test in Progress

# TEXAS A&M UNIVERSITY

COLLEGE STATION, TEXAS 77843

Liberal Arts — Cooperative Education  
Telephone 713/845-7814



September 17, 1981

## MEMORANDUM

TO: Students of all majors  
College of Liberal Arts

FROM: Henry D. Pope and Susannah R. Lindsay  
Cooperative Education Office

SUBJECT: Official Notice for CO-OP placements in the College of Liberal Arts

ATTENTION!

CO-OP EDUCATION

College of Liberal Arts

HAS CAREER OPPORTUNITIES AVAILABLE WITH THE FOLLOWING EMPLOYERS

BROWN&ROOT: HOUSTON, TEXAS

JOB DESCRIPTION:

PERSONNEL WORK

CORPS OF ENGINEERS: GALVESTON, TEXAS

JOB DESCRIPTION:

ECONOMIC PLANNING

IBM CORPORATION: GENERAL SYSTEMS DIVISION (TEXAS)

JOB DESCRIPTION:

SALES AND MARKETING

FOLEY'S: HOUSTON, TEXAS

JOB DESCRIPTION:

MANAGEMENT TRAINEE

NATIONAL AERONAUTICS&SPACE ADMINISTRATION: HOUSTON, TEXAS

JOB DESCRIPTION:

MANAGEMENT TRAINEE

LEGAL CO-OP: HOUSTON, DALLAS, AUSTIN, AND SAN ANTONIO, TEXAS

JOB DESCRIPTION:

LEGAL ASSISTANT

BRAZOS COUNTY AND BURLESON COUNTY PROBATION OFFICES

JOB DESCRIPTION:

ASSISTANT PROBATION OFFICER

FOR INFORMATION COME TO: 107 HARRINGTON TOWER  
PHONE: 845-7814

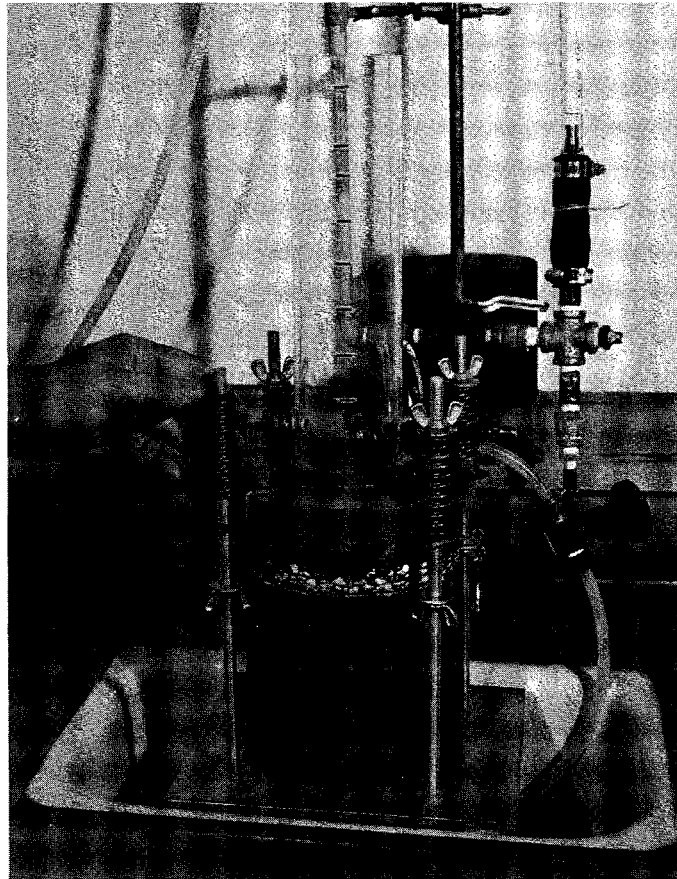


Figure      Compression Springs Being Positioned on Support Rods

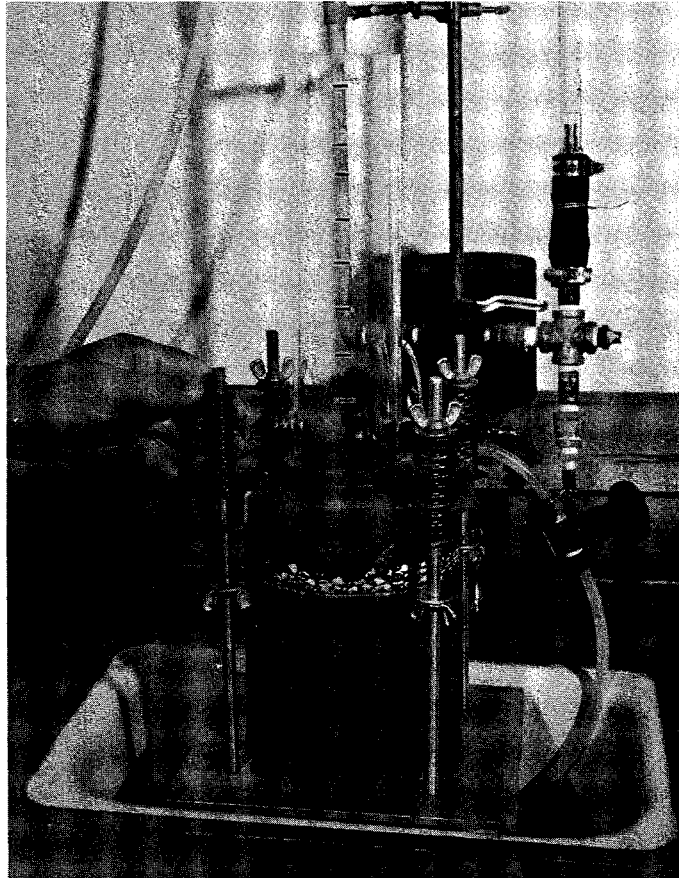


Figure      Compression Springs Being Positioned on Support Rods



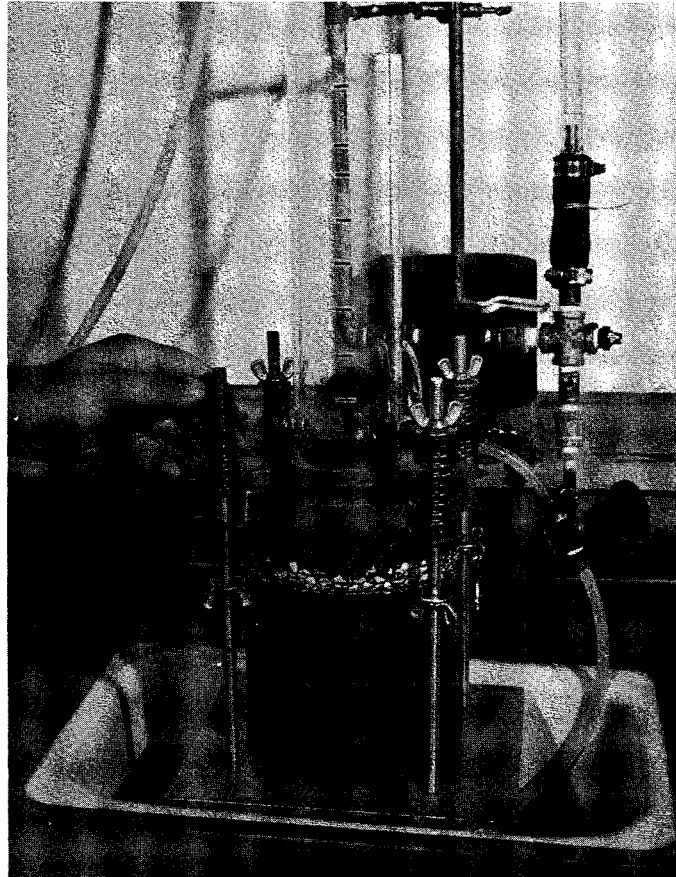


Figure      Compression Springs Being Positioned on Support Rods

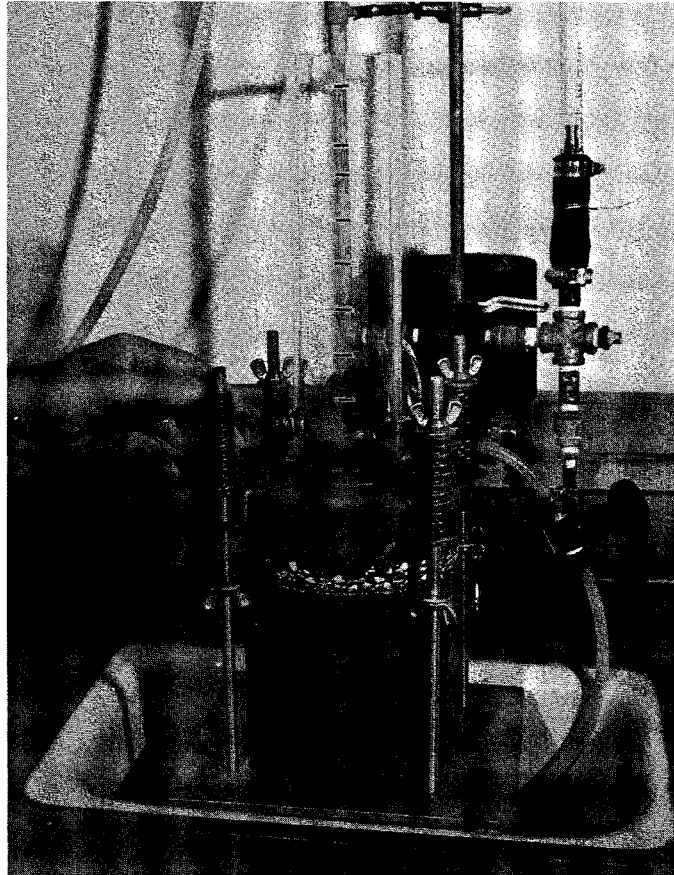


Figure      Compression Springs Being Positioned on Support Rods

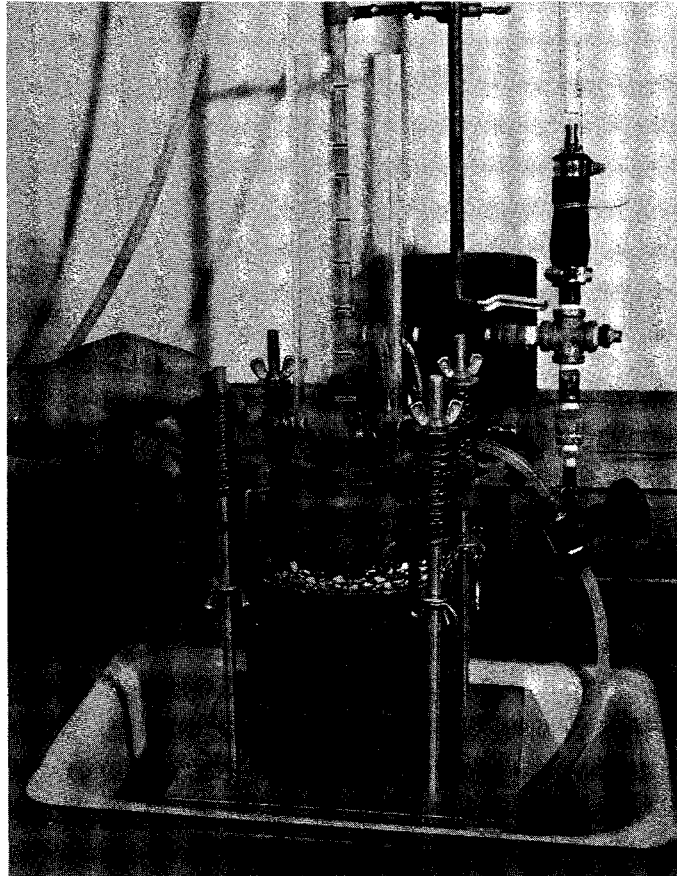


Figure      Compression Springs Being Positioned on Support Rods

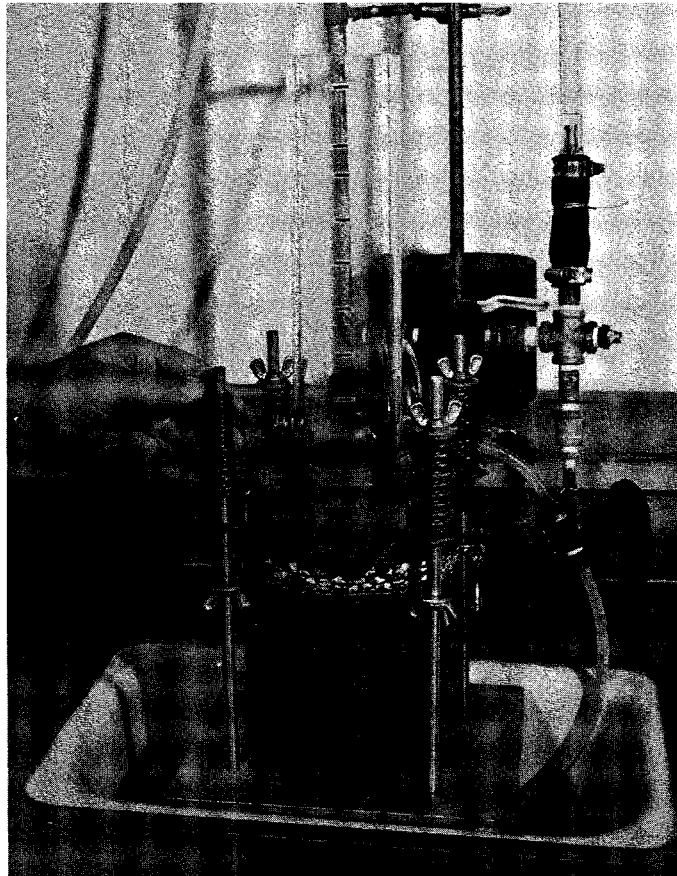


Figure      Compression Springs Being Positioned on Support Rods

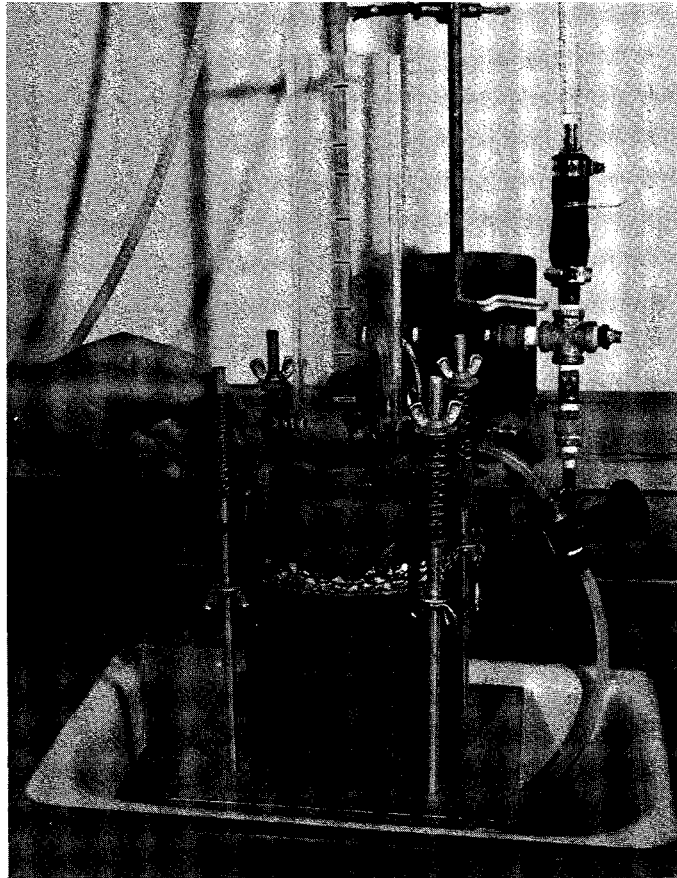


Figure      Compression Springs Being Positioned on Support Rods

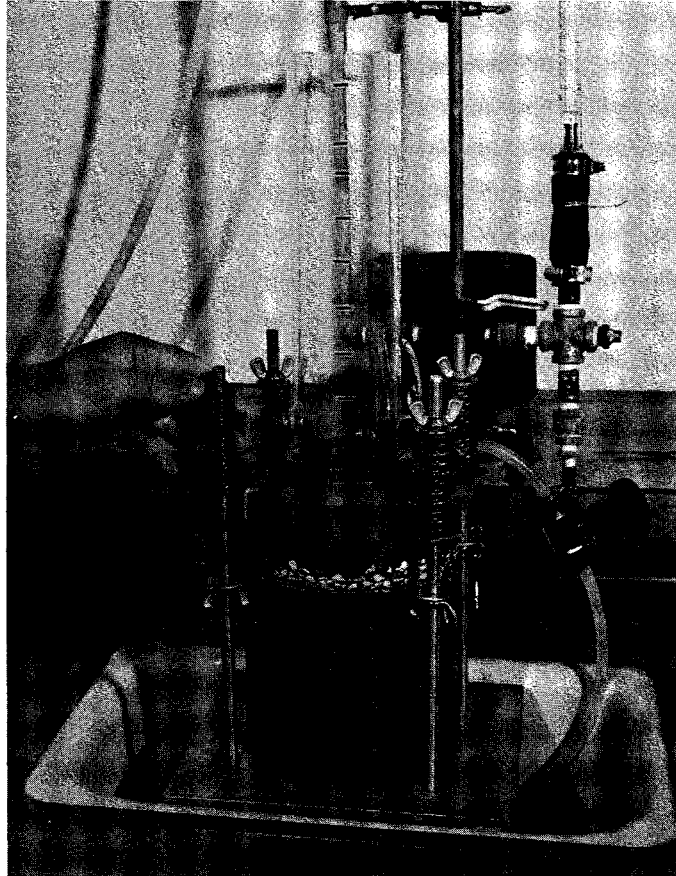


Figure      Compression Springs Being Positioned on Support Rods

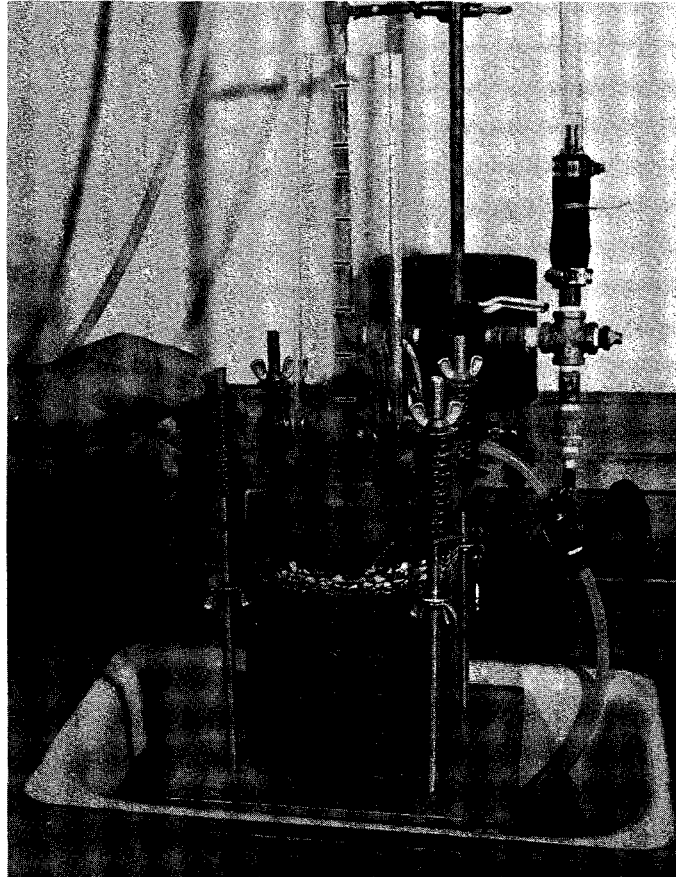


Figure      Compression Springs Being Positioned on Support Rods

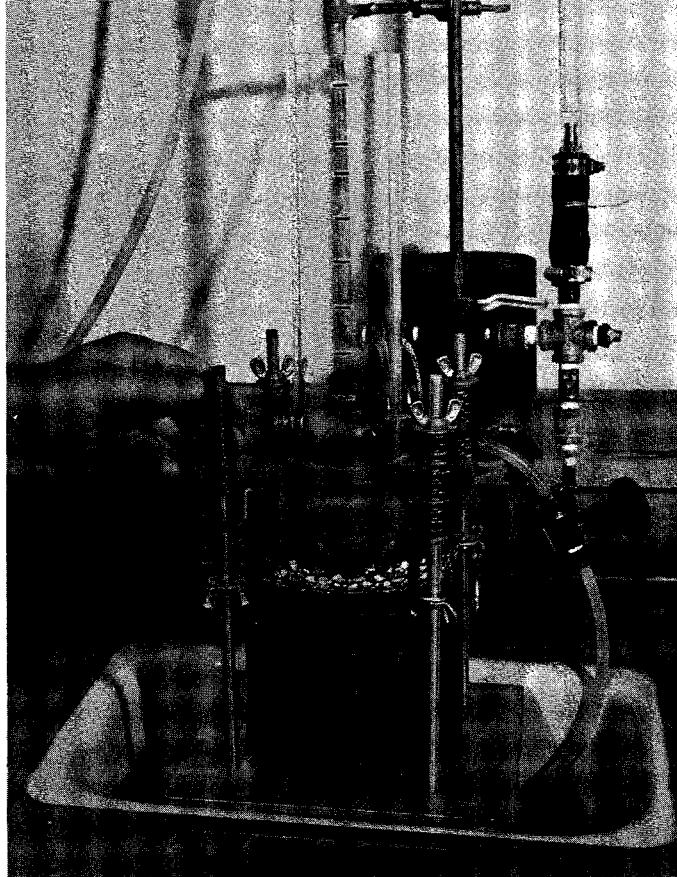


Figure      Compression Springs Being Positioned on Support Rods



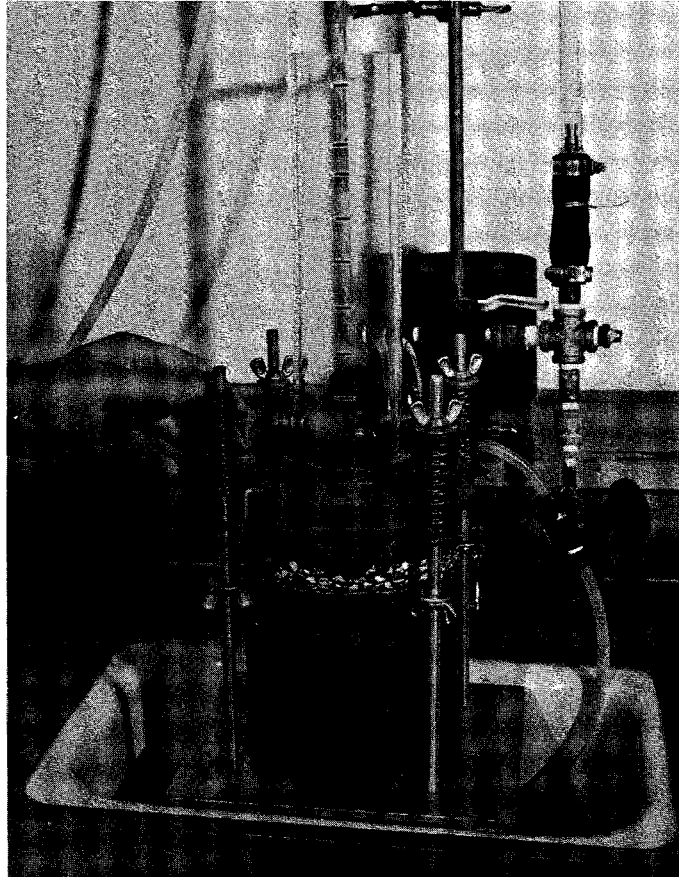


Figure      Compression Springs Being Positioned on Support Rods

# TEXAS A&M UNIVERSITY

COLLEGE STATION, TEXAS 77843



Liberal Arts — Cooperative Education  
Telephone 713/845-7814

September 17, 1981

## MEMORANDUM

TO: Students of all majors  
College of Liberal Arts

FROM: Henry D. Pope and Susannah R. Lindsay  
Cooperative Education Office

SUBJECT: Official Notice for CO-OP placements in the College of Liberal Arts

ATTENTION!

CO-OP EDUCATION

College of Liberal Arts

HAS CAREER OPPORTUNITIES AVAILABLE WITH THE FOLLOWING EMPLOYERS

BROWN&ROOT: HOUSTON, TEXAS

JOB DESCRIPTION:

PERSONNEL WORK

CORPS OF ENGINEERS: GALVESTON, TEXAS

JOB DESCRIPTION:

ECONOMIC PLANNING

IBM CORPORATION: GENERAL SYSTEMS DIVISION (TEXAS)

JOB DESCRIPTION:

SALES AND MARKETING

FOLEY'S: HOUSTON, TEXAS

JOB DESCRIPTION:

MANAGEMENT TRAINEE

NATIONAL AERONAUTICS&SPACE ADMINISTRATION: HOUSTON, TEXAS

JOB DESCRIPTION:

MANAGEMENT TRAINEE

LEGAL CO-OP: HOUSTON, DALLAS, AUSTIN, AND SAN ANTONIO, TEXAS

JOB DESCRIPTION:

LEGAL ASSISTANT

BRAZOS COUNTY AND BURLESON COUNTY PROBATION OFFICES

JOB DESCRIPTION:

ASSISTANT PROBATION OFFICER

FOR INFORMATION COME TO: 107 HARRINGTON TOWER  
PHONE: 845-7814

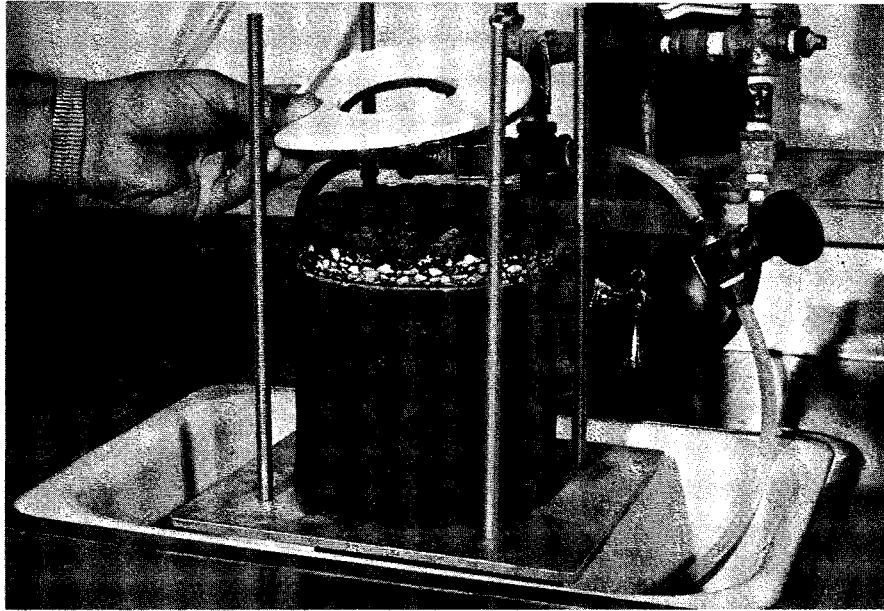


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

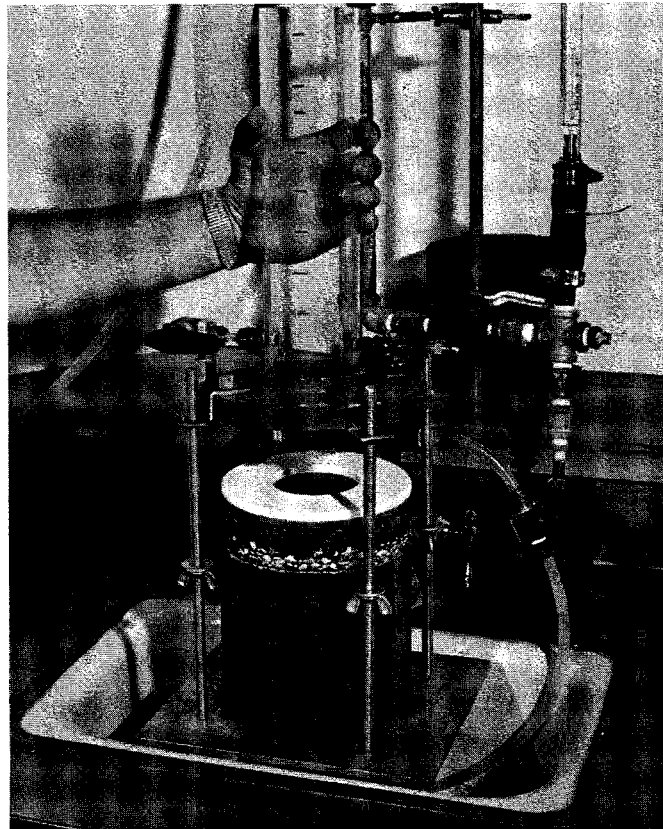


Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

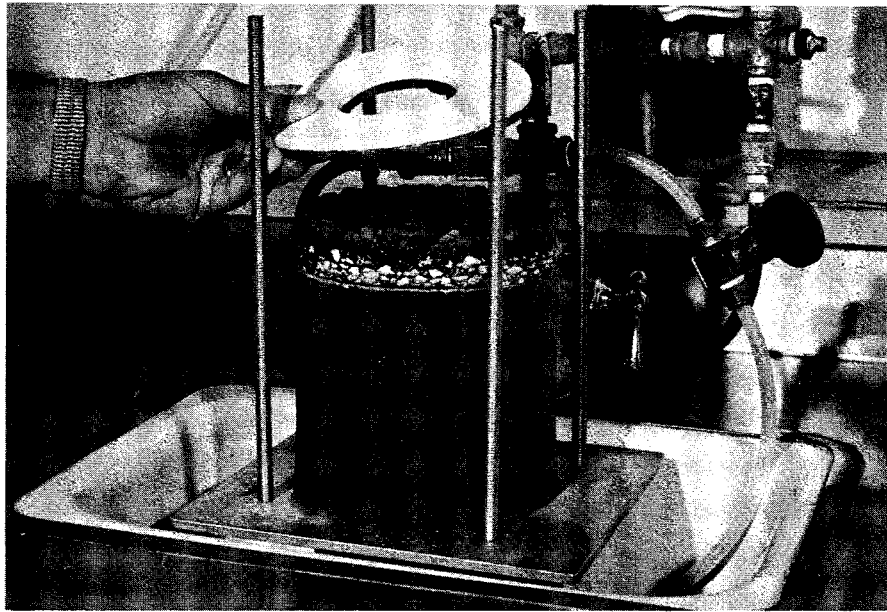


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied



Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

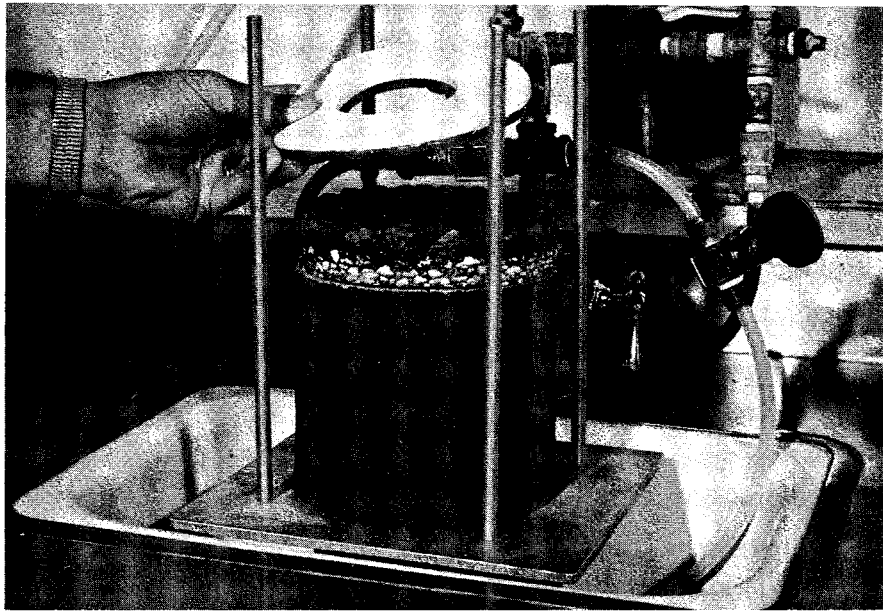


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

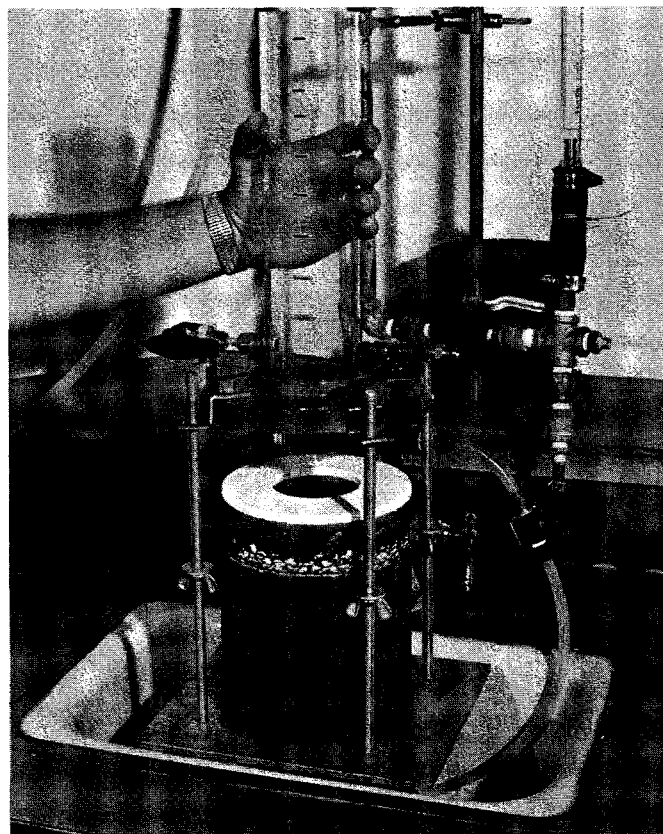


Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

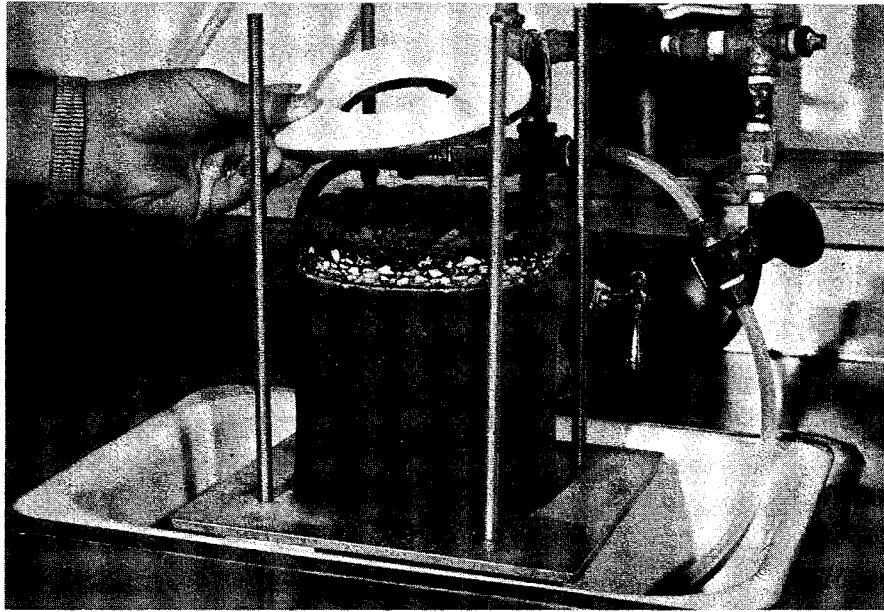


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

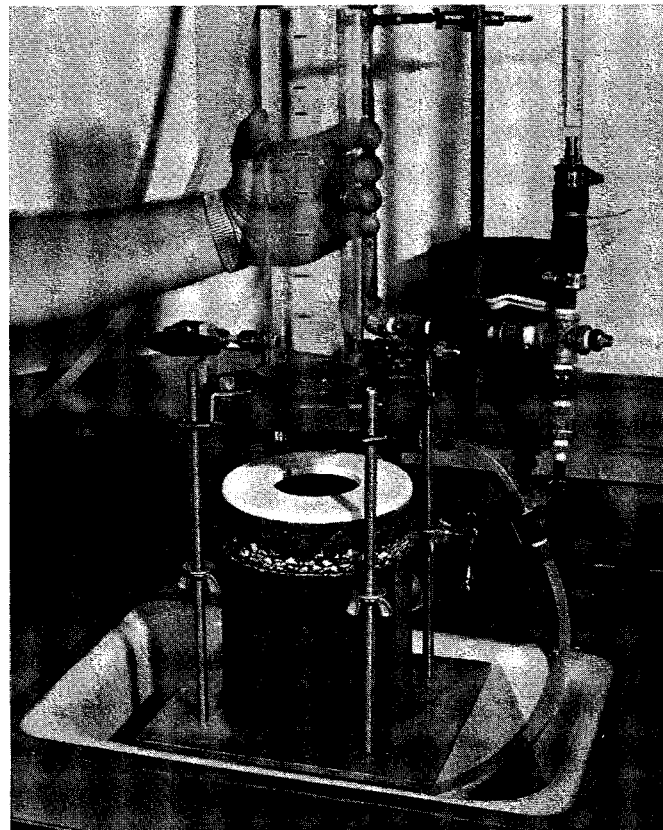


Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

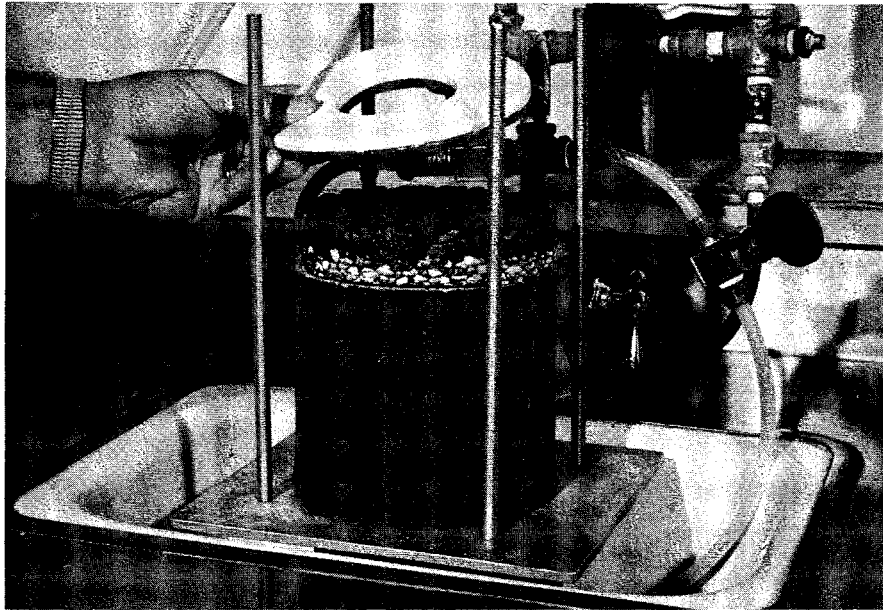


Figure 1 Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied



Figure 2 Gasket in Place and Permeameter Assembly Being Lowered Into Position



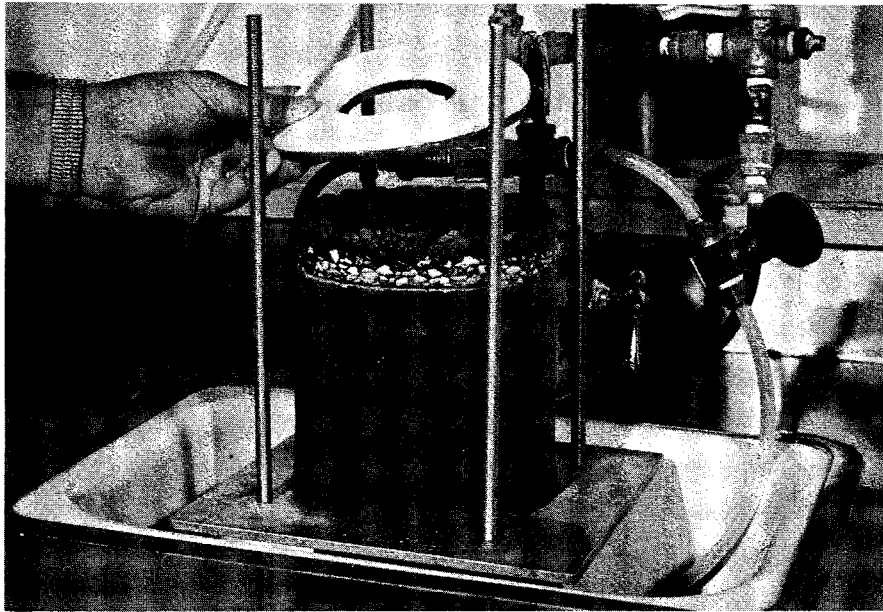


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied



Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position



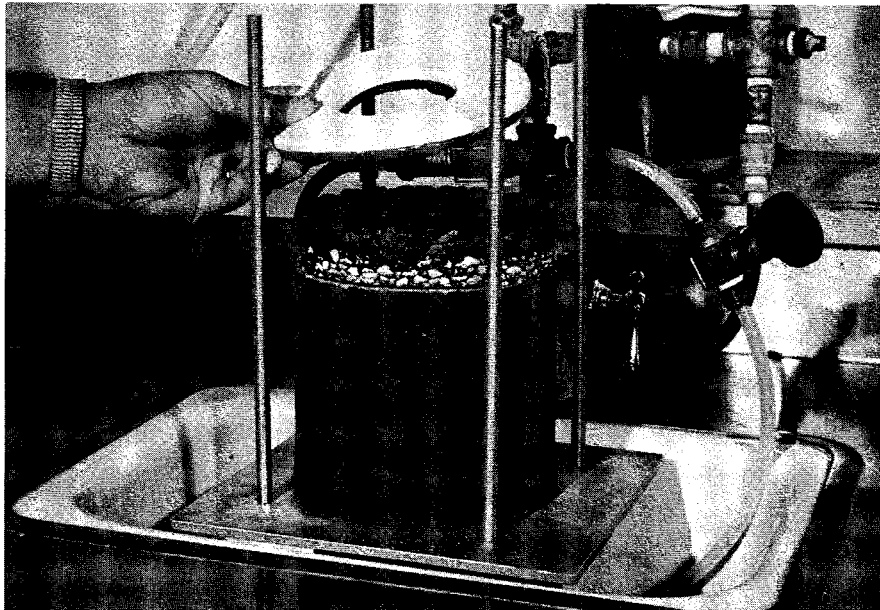


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

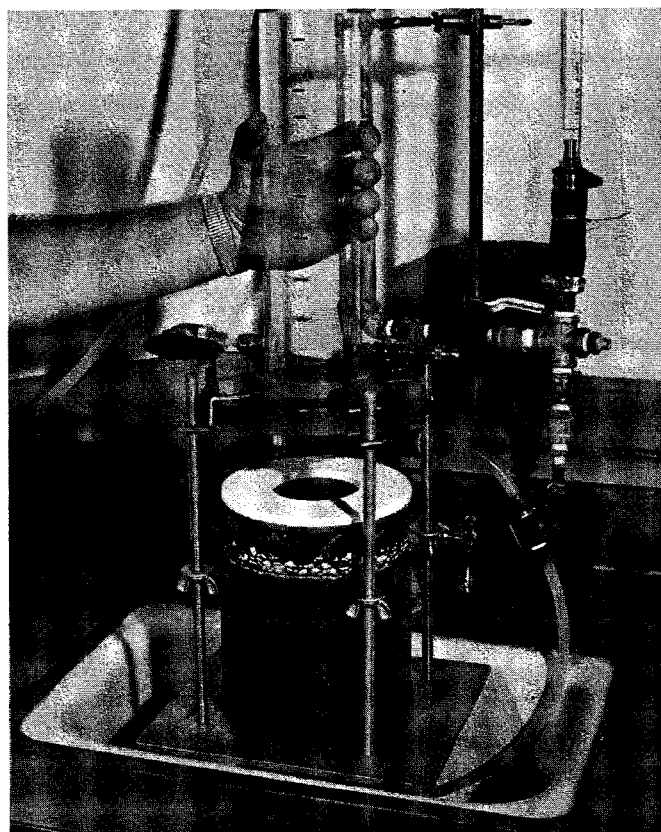


Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

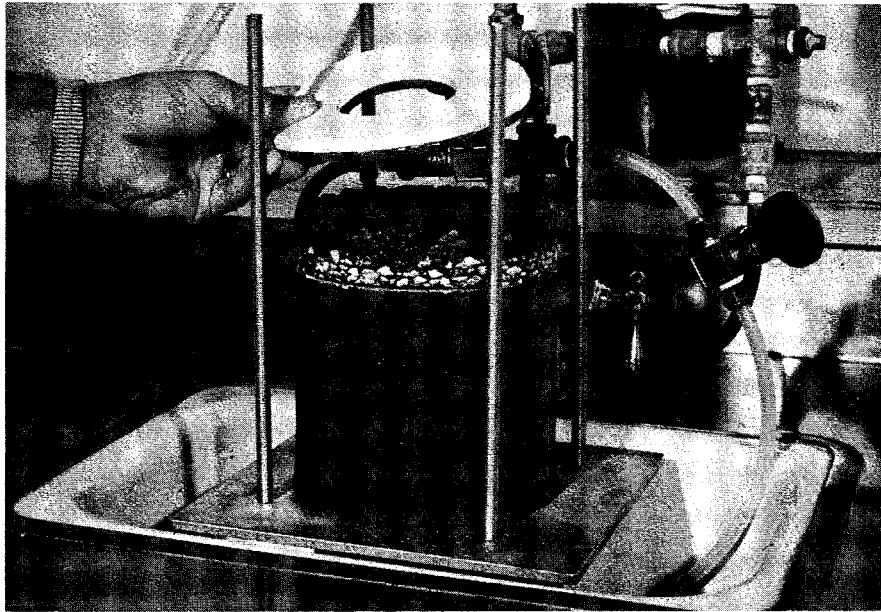


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

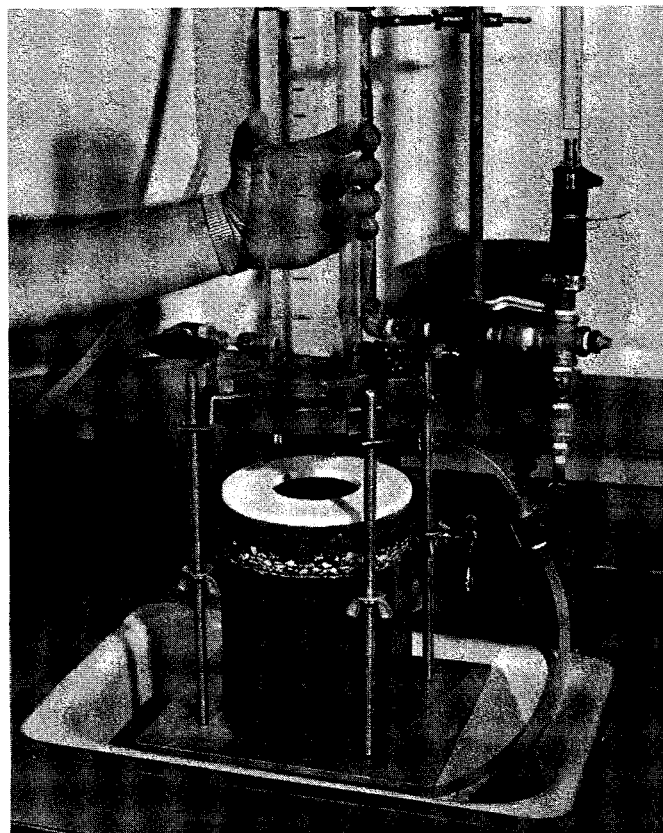


Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

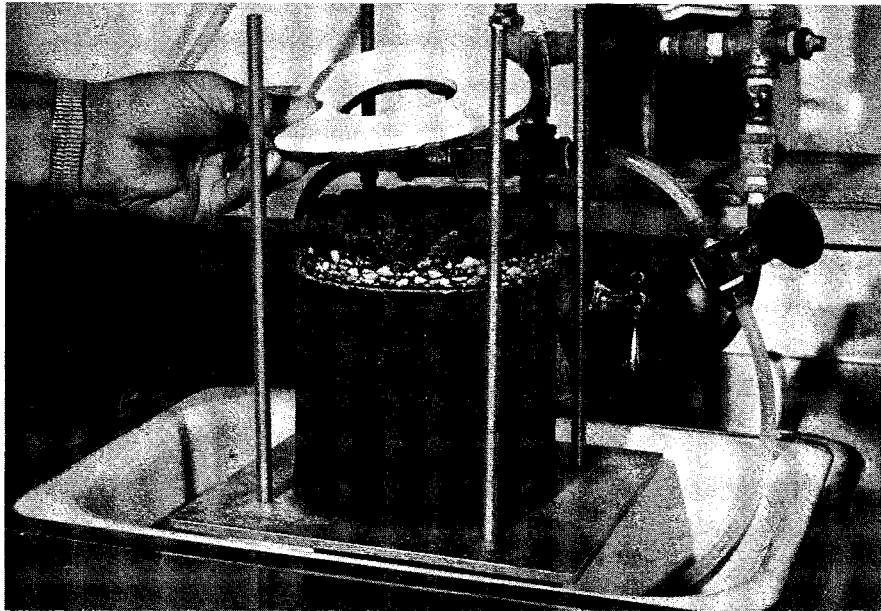


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied



Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

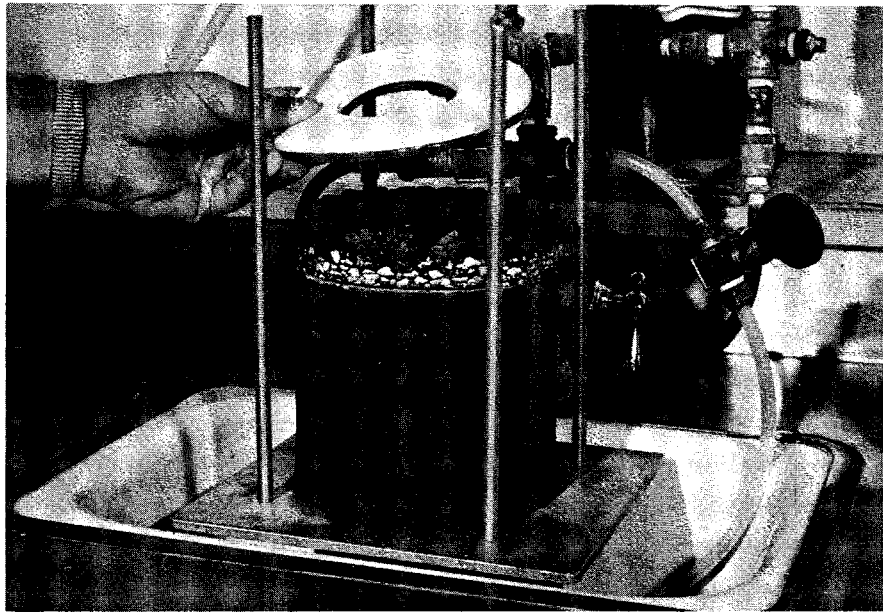


Figure Specimen Positioned on Baseplate and "Silicone-Sponge Rubber" Gasket Being Applied

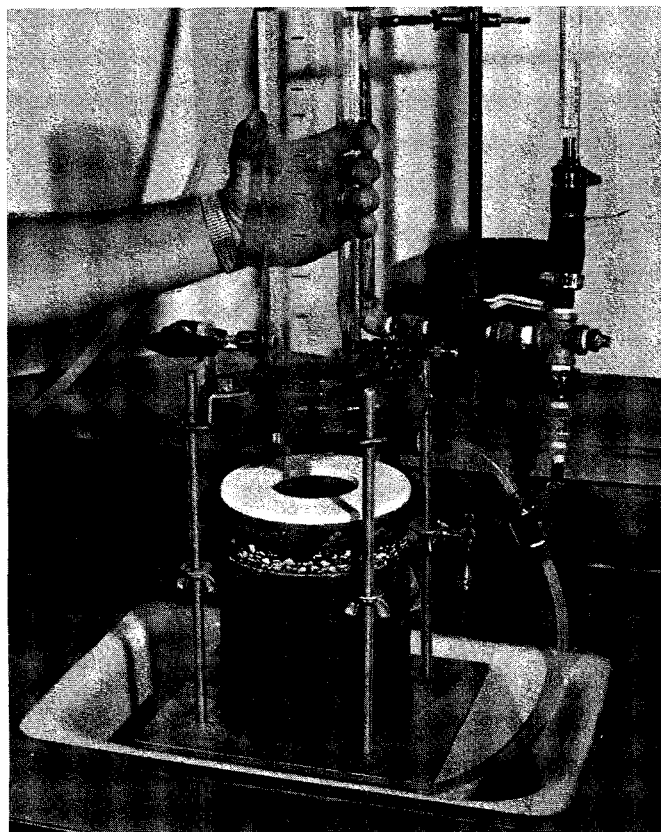


Figure Gasket in Place and Permeameter Assembly Being Lowered Into Position

# TEXAS A&M UNIVERSITY

COLLEGE STATION, TEXAS 77843

Liberal Arts — Cooperative Education  
Telephone 713/845-7814



September 17, 1981

## MEMORANDUM

TO: Students of all majors  
College of Liberal Arts

FROM: Henry D. Pope and Susannah R. Lindsay  
Cooperative Education Office

SUBJECT: Official Notice for CO-OP placements in the College of Liberal Arts

ATTENTION!

CO-OP EDUCATION

College of Liberal Arts

HAS CAREER OPPORTUNITIES AVAILABLE WITH THE FOLLOWING EMPLOYERS

BROWN&ROOT: HOUSTON, TEXAS

JOB DESCRIPTION:

PERSONNEL WORK

CORPS OF ENGINEERS: GALVESTON, TEXAS

JOB DESCRIPTION:

ECONOMIC PLANNING

IBM CORPORATION: GENERAL SYSTEMS DIVISION (TEXAS)

JOB DESCRIPTION:

SALES AND MARKETING

FOLEY'S: HOUSTON, TEXAS

JOB DESCRIPTION:

MANAGEMENT TRAINEE

NATIONAL AERONAUTICS&SPACE ADMINISTRATION: HOUSTON, TEXAS

JOB DESCRIPTION:

MANAGEMENT TRAINEE

LEGAL CO-OP: HOUSTON, DALLAS, AUSTIN, AND SAN ANTONIO, TEXAS

JOB DESCRIPTION:

LEGAL ASSISTANT

BRAZOS COUNTY AND BURLESON COUNTY PROBATION OFFICES

JOB DESCRIPTION:

ASSISTANT PROBATION OFFICER

FOR INFORMATION COME TO: 107 HARRINGTON TOWER  
PHONE: 845-7814

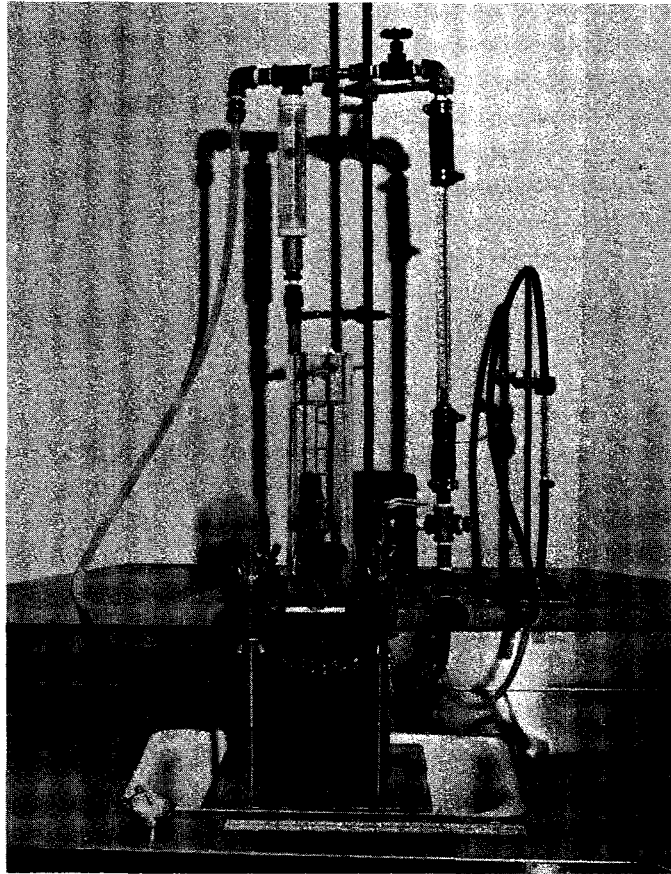


Figure      Permeability Test Apparatus

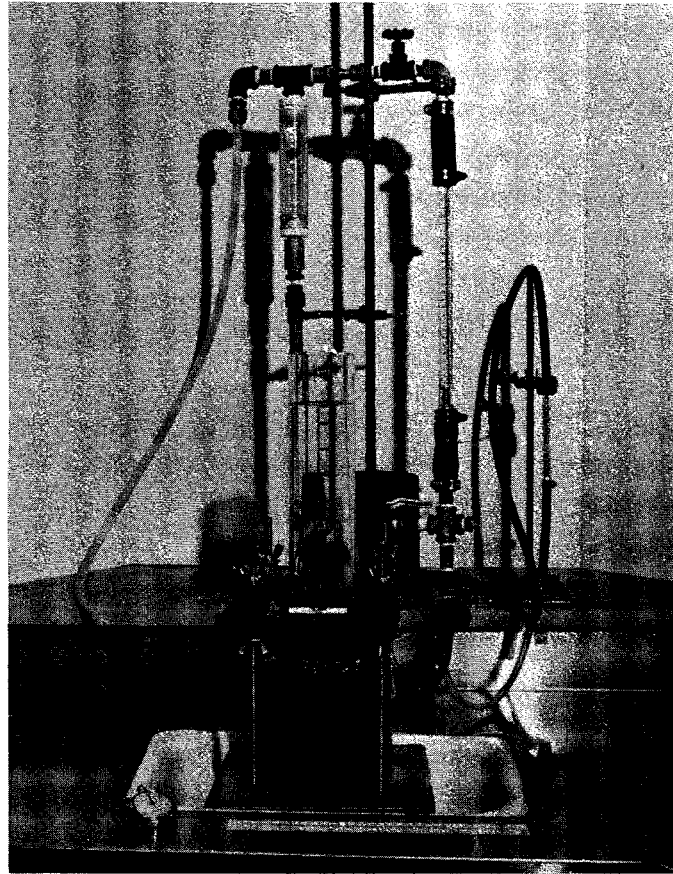


Figure      Permeability Test Apparatus

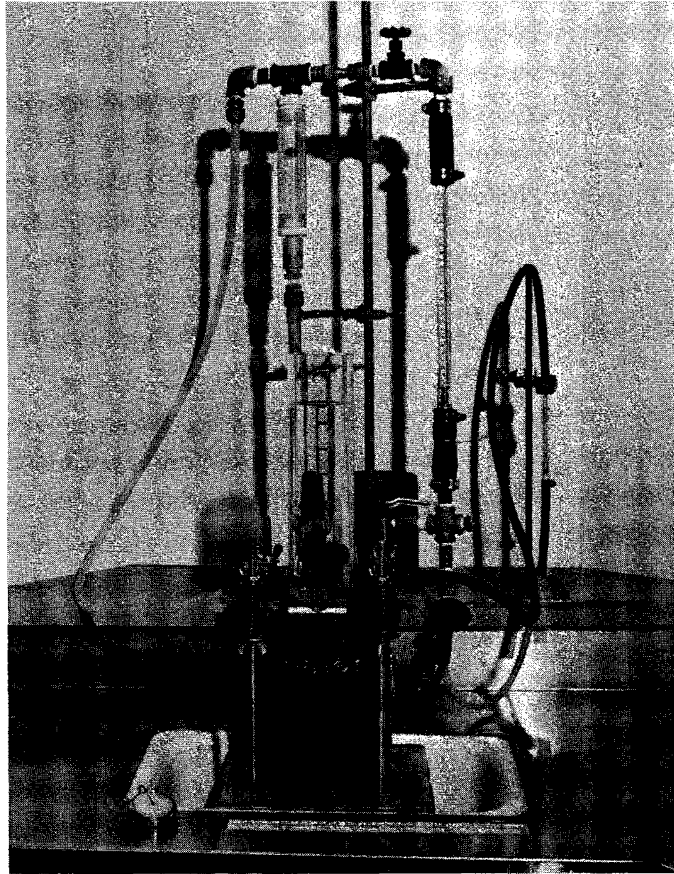


Figure      Permeability Test Apparatus





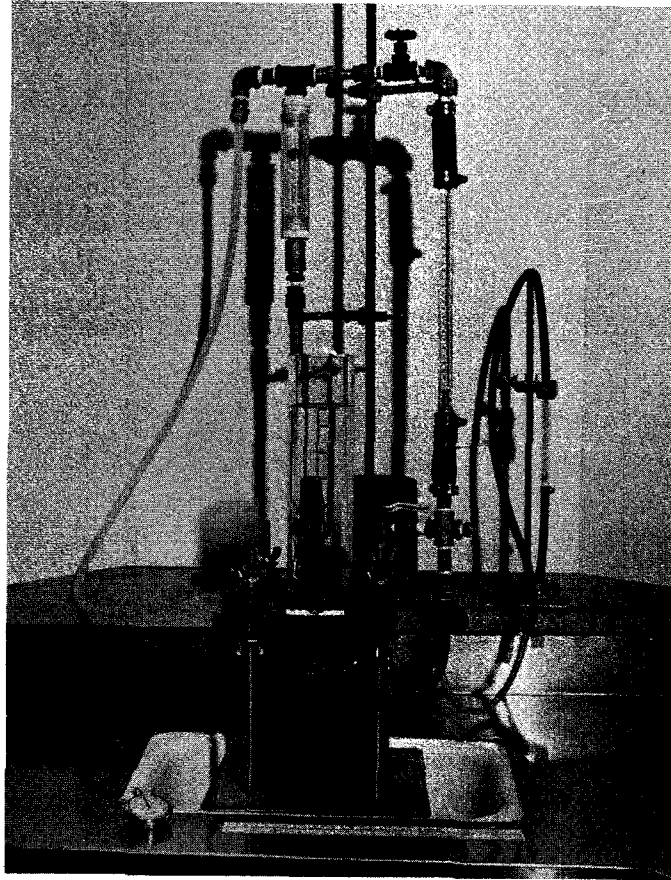


Figure      Permeability Test Apparatus

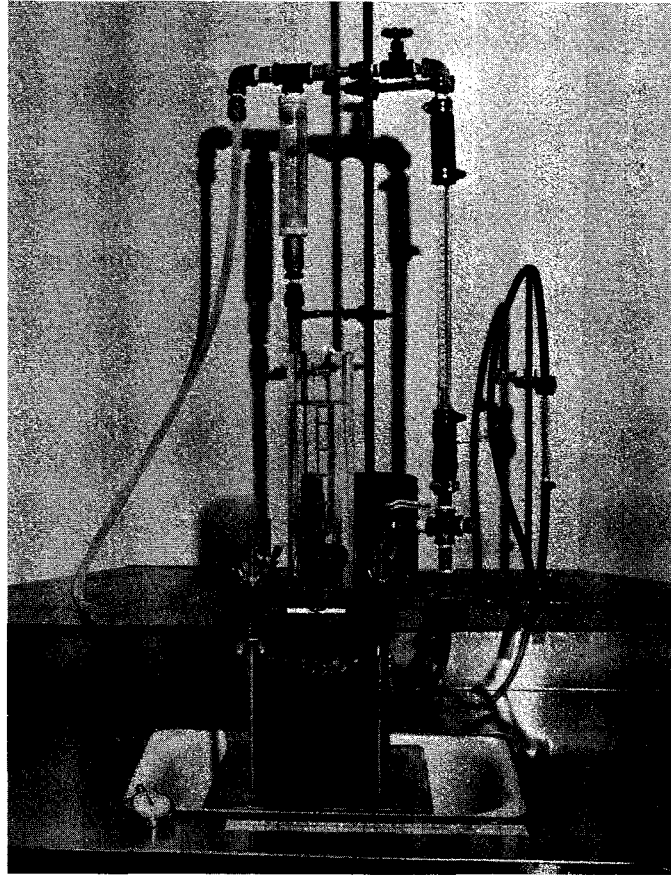


Figure      Permeability Test Apparatus

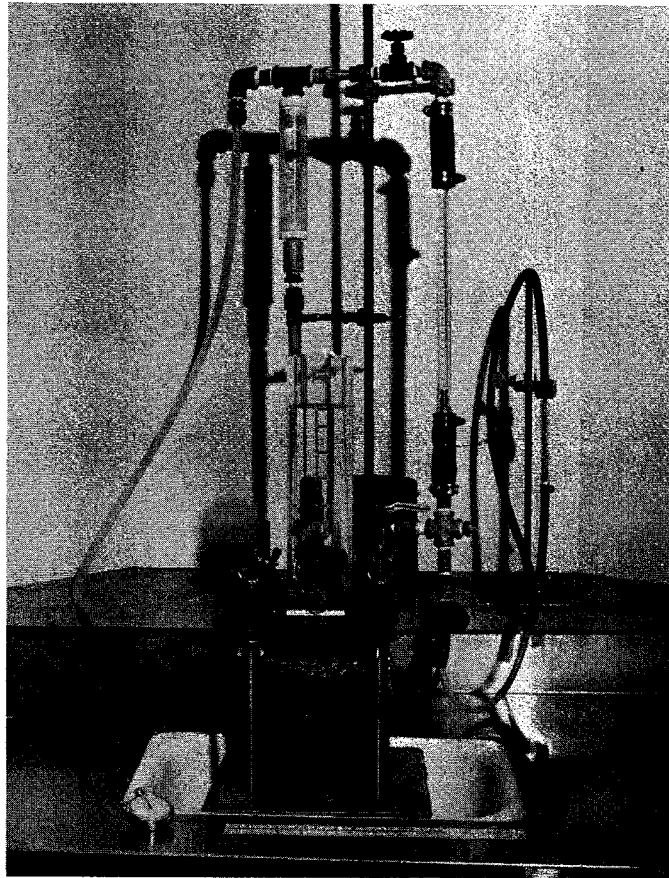


Figure      Permeability Test Apparatus

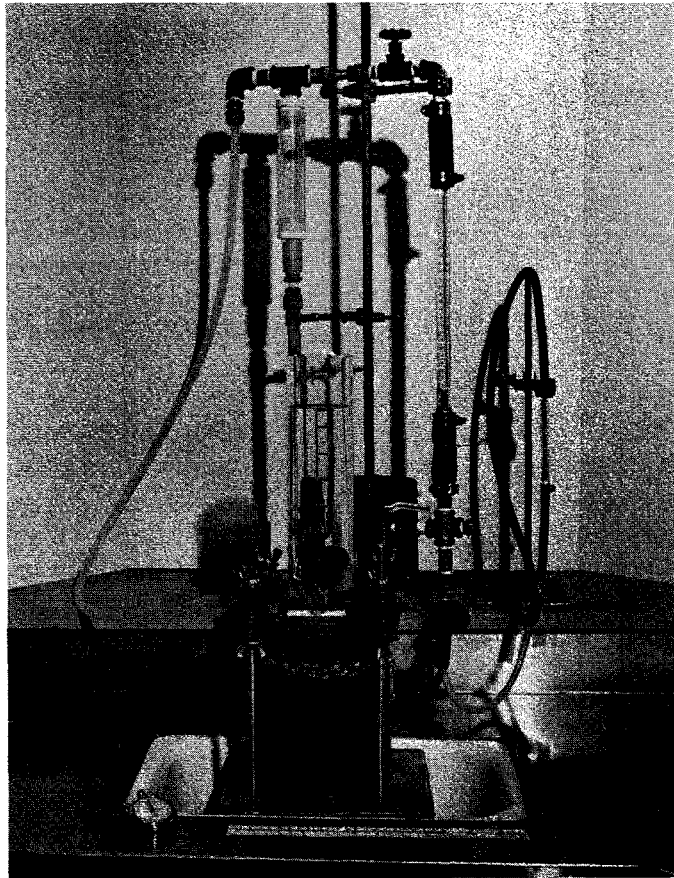


Figure      Permeability Test Apparatus