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FINAL REPORT ON
UTILIZATION OF LOCAL SANDS
IN HIGHWAY CONSTRUCTION

by
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FINAL REPORT

on

UTILIZATION OF LOCAL SANDS IN HIGHWAY CONSTRUCTION

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CHAPTER 1

INTRODUCTION

Crushed stone aggregates for use in building and highway construction are not present in the Coastal Plain Province of Georgia [1,2]. In this area which covers almost one-half of the state, all of the coarse aggregate presently used is shipped in from the Piedmont Province of the state which lies above the fall line. For example, during the construction of a portion of Interstate I-95, the crushed stone used in the asphalt concrete was shipped from Stockbridge, Georgia, to near Brunswick which is approximately 240 miles (460 km). Bringing in the large quantity of aggregate required for construction in the Coastal Plain has always been expensive. The cost of construction aggregates has risen steadily due to the present energy crisis and the rapidly increasing cost of rail freight. In some instances construction has been delayed by transportation problems such as strikes and shortages of rail cars. These factors have contributed to greatly increased construction costs in the Coastal Plain, and cause the aggregates presently used in construction to be highly energy intensive. An important need therefore exists for finding suitable sources of local aggregates and developing the required technology for using these aggregates in the construction industry.

Extensive deposits of sand and gravel are found throughout the Coastal Plain waterways and along the continental shelf. The types of materials available in the coastal waterways have been briefly described in earlier Sea Grant projects (Harding and Woolsey, 1972-1973 and 1973-1974, Hicks and Martin, 1975). Hicks and Martin [1] concluded that an abundant quantity of good quality sands are present in the coastal waterways of Georgia, and that these sands are suitable for construction and industrial purposes. The findings of Hicks and Martin were based on the examination of approximately 500 small samples of sand and gravel obtained by Harding and Woolsey [3] at various locations along the Georgia coast. If river and marine sources of the sand are utilized, dredging operations would have to comply with the applicable regulations of the Corps of Engineers [4].

The most important potential uses of natural alluvial and marine deposited sands found in the Coastal Plain are in (1) highway construction including asphalt concrete, sand asphalts, sand-cement and unstabilized sands or sand-stone blends, (2) concrete for use in general construction, and (3) the replenishment of beaches with new sand. In this investigation emphasis was given to studying the use of local sand deposits in flexible pavement construction, although use of portland cement concrete is also considered.

Flexible Pavement Design Considerations

A flexible asphalt concrete pavement should be designed to provide a durable, skid-resistant surface which is both (1) resistant to a fatigue-type failure under in-service conditions, and (2) does not undergo an excessive amount of rutting under the design traffic loading. Fatigue resistance is the ability to withstand cracking of stabilized layers due to repeated flexing of the surfacing that occurs with the passage of a large number of heavily loaded vehicles over the pavement. The structural design of a pavement consists of selecting compatible combinations of materials and layer thicknesses which minimize the occurrence of both fatigue and rutting failures in the pavement.

A fatigue failure of a stabilized surface or base course results in cracking which in turn allows water to enter the pavement structure. As a result, the load spreading capability of the pavement is reduced which, together with the detrimental effects of the water, can eventually lead to serious rutting in the base, subbase, and subgrade. Fatigue cracking of flexible pavement sections is the most important distress mode in the United States [5] and hence deserves careful consideration in the design of sand-asphalt mixes.

The asphalt concrete surface and base courses are critical components of a flexible pavement structure, and it is essential to minimize cracking and rutting in these layers. Laboratory investigations [6,7,8] have indicated that the fatigue and durability performance of an asphalt concrete mix is significantly influenced by the asphalt content, percent voids, mineral filler, the characteristics of bitumen binder, and to several other less important variables. The effects of these variables on the fatigue and rutting performance of sand-stone asphalt concrete mixes and sand-asphalt mixes have not been previously determined by laboratory studies for uniformly graded mixes such as those used in the United States. Rutting in sand-asphalt pavements has been found to be significantly greater than for pavements using asphalt concrete. Careful consideration of rutting, therefore, must be given in developing stable sand-asphalt and sand-stone mixes.

Primary variables affecting the performance of a soil-cement base are the type and gradation of soil, the percentage of portland cement used, and the compaction level. Important problems in the use of soil-cement bases which can greatly affect pavement performance are durability and shrinkage cracking which usually reflect to the surface. Durability usually can be handled by using a sufficiently high cement content, but the problem of shrinkage cracking has not yet been fully solved.

Project Objectives

The overall purpose of this project is to determine the feasibility of using coastal sands and gravels as construction or specialty materials. Specific objectives are as follow:

1. Locate sources of local sands and gravels,
2. Obtain bulk coastal sand and gravel deposits for use in detailed testing,
3. Identify potential uses of coastal materials,
4. Develop aggregate and aggregate-admixture combinations for the most promising of the potential uses and determine their engineering properties,
5. Develop material specifications for use in construction or as specialty products,
6. Evaluate the economic feasibility of using these materials for the indicated specific construction applications.

The above objectives were accomplished by obtaining over six tons of sand from the Altamaha and Ogeechee Rivers and also sand samples from selected sand pits. Samples were tested routinely for grain size, sand equivalent, and specific gravity. Further, fatigue tests were performed on a wide range of soil-cement, sand-asphalt and sand-stone asphalt blends utilizing local materials. The fatigue test consisted of placing a rectangular beam specimen of stabilized material on a rubber (elastic) subgrade. A repeated load was applied at the center of the beam until fatigue failure occurred. The fatigue tests were performed at 80°F (27°C) in an environmental chamber. The rutting characteristics of the asphalt-stabilized materials were determined using a repeated load tri-axial test. The rutting tests were performed at a temperature of 95°F (35°C) which is approximately the theoretical mean temperature in Georgia and Florida for rutting of asphalt concrete pavements that consist of approximately 10.5 in. (257 mm) asphalt concrete [6].

A comprehensive study was made of the variables affecting the fatigue and rutting performance of sand and sand-stone blend asphalt mixes having approximately 16, 40, 70, and 100 percent sand. Other variables studied included asphalt content, voids content, gradation and aggregate source. The fatigue performance of sand-cement specimens was evaluated for two different sands with fatigue tests performed on seven different mixes. Variables studied included gradation, crushed stone content and cement content. Soluble salt contents of the sands were also evaluated to determine if the salt concentrations were high enough to affect performance.

Preliminary design criterion and mix specifications were developed for utilizing local sands in sand-asphalt and sand-stone asphalt mixes and also in sand-cement bases of flexible highway pavements. The recommendations presented were based on the results of the detailed laboratory studies and also interviews and field inspections that were conducted in Florida, Georgia, Maryland and South Carolina. Most of the significant findings of the study are presented in Chapter 7.

CHAPTER 2

LITERATURE REVIEW USE OF SAND IN PAVEMENT CONSTRUCTION

Introduction

In 1976 the production of construction aggregates in the United States was in excess of 1 3/4 billion tons [9]. When compared with the annual demand, present construction aggregate production appears to be quite favorable, and projections for the next twenty years indicate that the supply of aggregates probably will meet the demand. Despite these facts, many areas of the southeastern United States are experiencing a severe shortage of high quality aggregates as illustrated in Figure 1. Shortages generally occur in areas which do not have developable supplies of construction aggregates [1,2,9,10]. In urban areas, zoning laws often prohibit or limit the size and number of stone quarries and gravel pits [14]. Probably the most extensive shortages are in the coastal plains and glaciated areas where rock suitable for producing crushed stone aggregates is not located close enough to the surface to be an economically viable alternative.

In recent years, the cost of energy has increased significantly. Greater energy costs have resulted in a significant increase in transportation costs associated with shipping gravel and crushed stone from the area of production to the construction site. The effect has been increased transportation costs and has doubled or tripled the cost of aggregates since 1974 in many areas [2]. The cost of shipping crushed stone aggregates to Brunswick and Savannah from the Piedmont area of Georgia was about 1.7 times the cost of the stone in 1978. Recent figures indicate that a shipping distance of 40 miles (70 km) can be considered the maximum possible before the adverse economics of transporting aggregates become serious.

High quality crushed stone and gravel are utilized in every phase of highway construction from the base course to the wearing surface. In 1977, almost 50 percent of all aggregates produced were utilized in the construction of highway pavements [10]. More extensive utilization of local materials in highway construction, therefore, is of great interest to local, state and federal governmental agencies responsible for constructing and maintaining public transportation systems at the lowest cost to the taxpayer.

Highways often are constructed many miles from approved or available stone sources. In Georgia, for example, construction of I-95 required the transportation of aggregates over distances usually exceeding 100

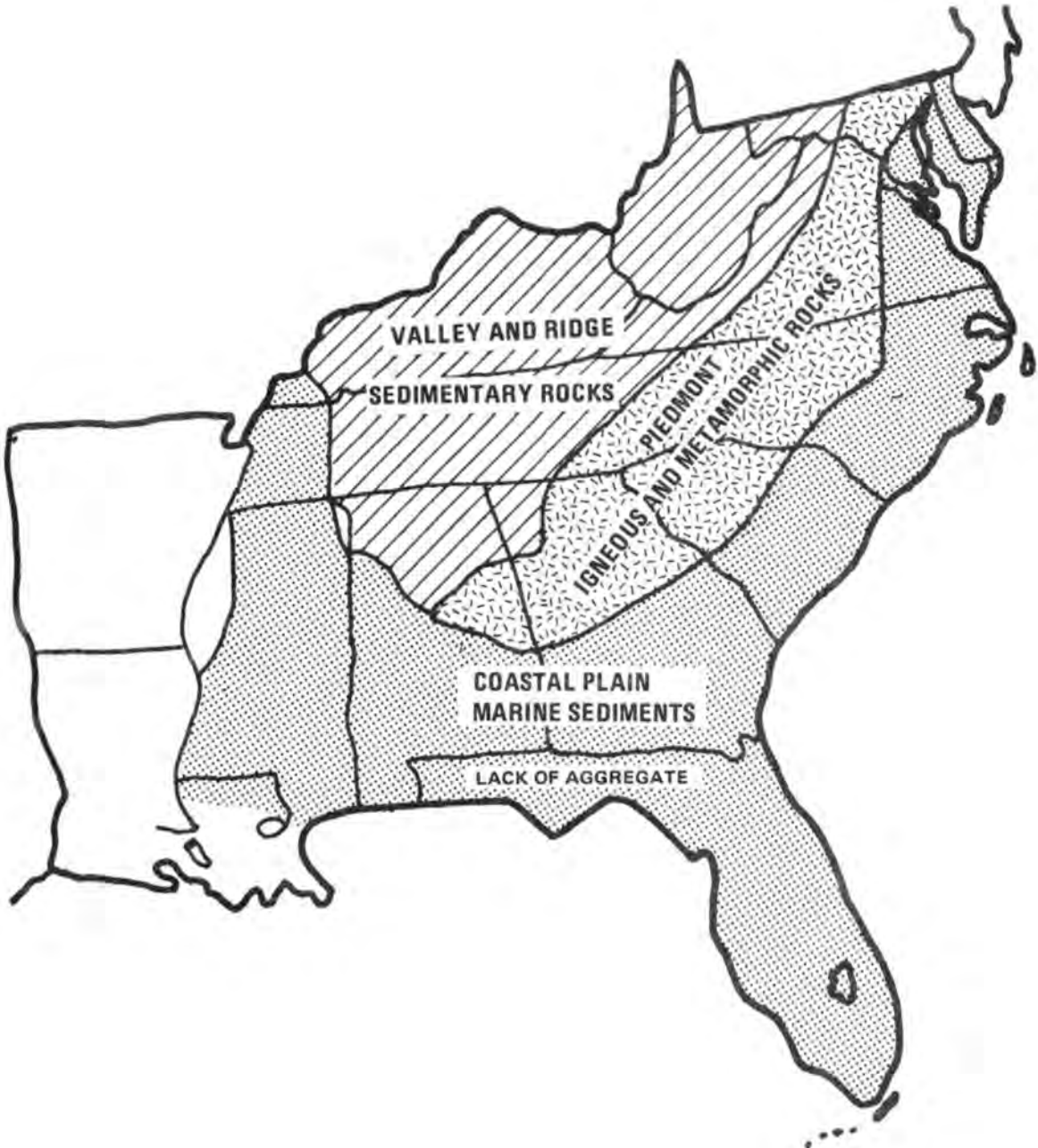


FIGURE 1. COASTAL PLAIN AREAS OF THE SOUTHEASTERN UNITED STATES GENERALLY HAVING A LACK OF CONSTRUCTION AGGREGATE (AFTER WALKER)

miles (170 km) and were as great as 250 miles (400 km) resulting in transportation costs far exceeding the cost of the aggregates. For these reasons, a number of state transportation organizations have become interested in determining the feasibility of more extensive use of locally available aggregates and waste materials as replacements for high quality crushed stone and gravel in highway construction.

The search for locally available materials has taken the following three diverse directions:

1. The utilization of marginal sands and gravels (and in some instances marginal crushed stone) which may not meet standard highway specifications but which have potential as substitutes when adequately stabilized for conventional base and subbase materials,
2. The use of domestic, industrial, and mining wastes as aggregate replacement, and,
3. The utilization of manufactured or synthetic aggregates.

In the literature review, only the use of natural materials in highway construction are considered since these materials are readily available in many areas and have a low-energy intensity.

Marginal sands and gravels are those materials which do not meet conventional accepted highway specifications. Usually sands in their natural state do not have sufficient strength to be used as base materials since the stress in the base is relatively great under heavy truck traffic. Therefore, stabilization of marginal materials using asphalt and portland cement are given primary emphasis in both the literature review and research investigation.

Performance of Sand-Asphalt

The most important advantage of using sand-asphalt bases compared with soil-cement is that shrinkage cracking is not a problem. Sand-asphalt mixes usually have much lower strengths than conventional asphalt concrete mixes, as indicated by conventional methods such as Marshall stability tests. As a result, if not properly designed and constructed, pavements using sand-asphalt mixes with low Marshall stabilities have the potential for excessive rutting and premature fatigue distress.

Marianna Test Road

During 1964, the Florida Department of Transportation constructed a sand-asphalt base test section at Marianna, Florida [61]. The test sections at Marianna have now been subjected to approximately 1.3 million equivalent 18 kip (80 kN) axle loads. The test pavement consisted of sixteen sections, each 100 ft. (30 m) in length. All test sections had a 3 in. (75 mm) thick asphalt concrete surfacing. Eight of the test sections were composed of a low-stability sand-asphalt base (566 lb.,

50 blow Marshall stability mix) 4 to 8 in. (100 to 200 mm) in thickness; the other eight sections had a high-stability sand-asphalt base (674 lb., 50 blow Marshall stability mix) of similar thicknesses. The sand used in the sand-asphalt base had 4 percent fines with essentially 100 percent passing the No. 4 sieve. The high-stability sections were stabilized with 6.4 percent asphalt, and the low-stability sections were stabilized with 6.9 percent. A summary of the basic material properties of the test pavement as constructed is given in Table 1.

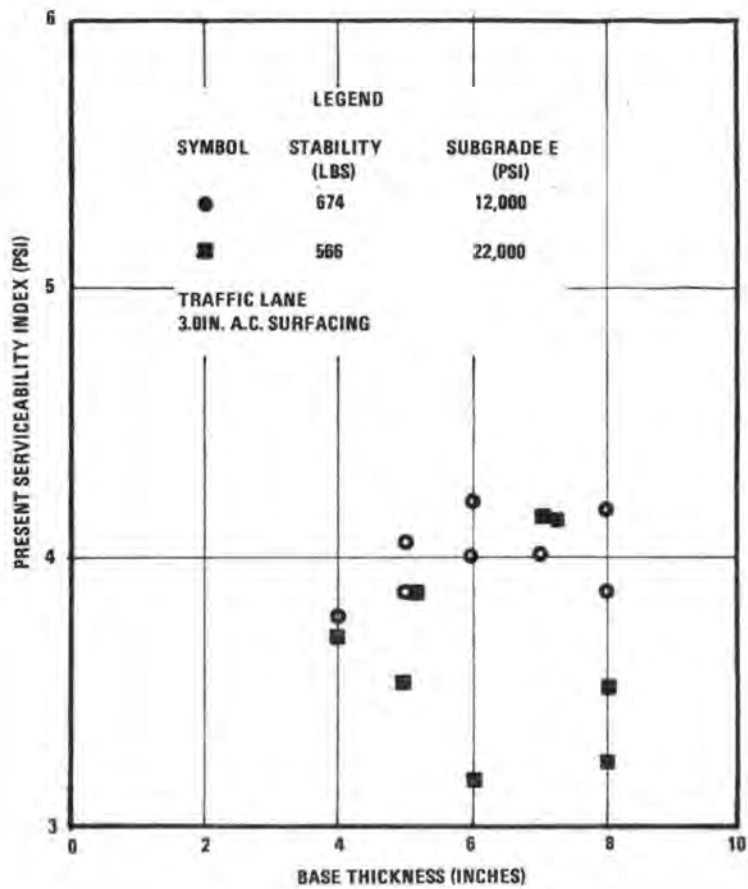
The natural soil subgrade (A-7 soil by the Bureau of Public Road Classification System) was undercut for a depth of 24 in. (0.6 m) and replaced with sand (A-2-4 Classification). The resulting modulus of elasticity of the prepared subgrade was found from field plate bearing tests to be approximately 12,000 psi (83,000 kN/m²) under the high-stability sections and 22,000 psi (150,000 kN/M²) under the low-stability sand-asphalt base sections.

Since construction, the surface roughness of the pavement has been measured using a CHLOE Profilometer and a Mays Ride Meter. Static surface deflection has been determined under a twenty kip static axle loading. Recoverable deflections have been found to decrease with time indicating a general stiffening of pavement structure as compaction and hardening occurs in the asphalt concrete surface and sand-asphalt base.

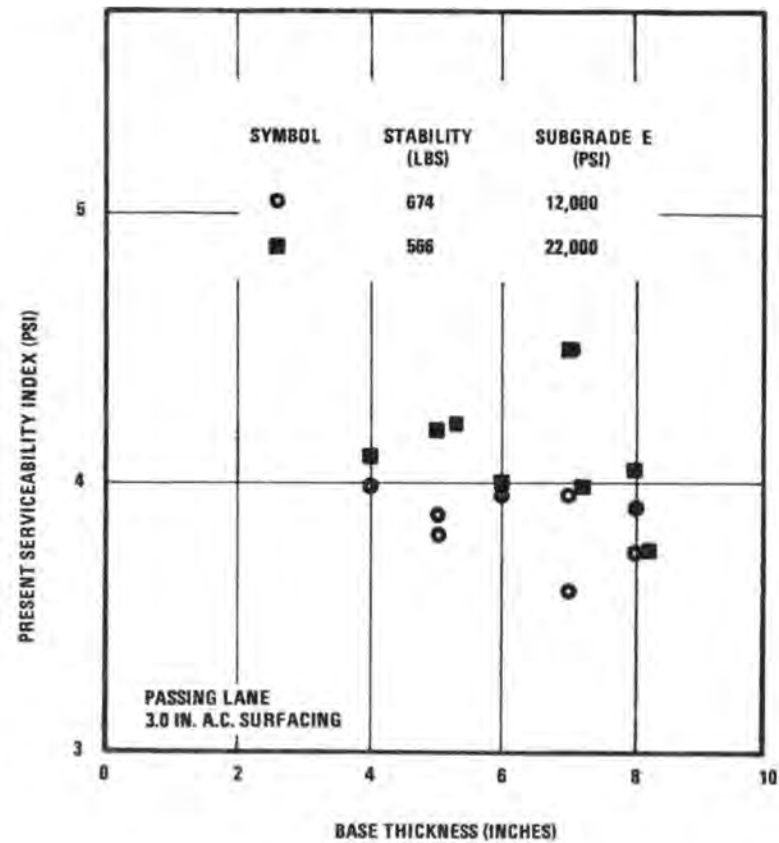
The observed pavement performance expressed in terms of the Present Serviceability Index (PSI value)¹ is summarized in Fig. 2 and Fig. 3. The average traffic lane PSI value for the high-stability sections was found to be 4.0 compared to 3.67 for the low-stability sections. In the passing lane, however, the average PSI value in high-stability sections was 3.86 compared with 4.10 for low-stability sections.

The observed variation in surface rut depth with sand-asphalt base thickness is shown in Fig. 4. The average rut depth in the traffic lane of high-stability sections was 0.25 in. (6 mm) compared with an average rut depth in low-stability sections of 0.43 in. (11 mm). On the average, a negligible difference in rut depth was found to exist between the passing and traffic lanes. High-stability sections show a decreasing rut depth with increasing sand-asphalt base thickness. In contrast, average rut depth in the traffic lane of low-stability sections appeared not to vary with base thickness, while rut depth in the passing lane increased with base thickness. Only one low-stability section reached a rut depth greater than 0.5 in. (13 mm). This section had an 8 in. (200 mm) base thickness and a rut depth of 0.6 in. (15 mm). Problems due to excessive rut depths have not been reported in this or other sections at the Marianna Test Road. Surface rut depth was found to increase almost linearly

¹The Present Serviceability Index (PSI) is a subjective rating developed at the AASHO Road Test. The PSI ratings vary from 0 to 5 with a rating of 5 indicating the theoretically best possible condition. The pavement is usually considered to have reached a condition of failure when the PSI value becomes 2.5.



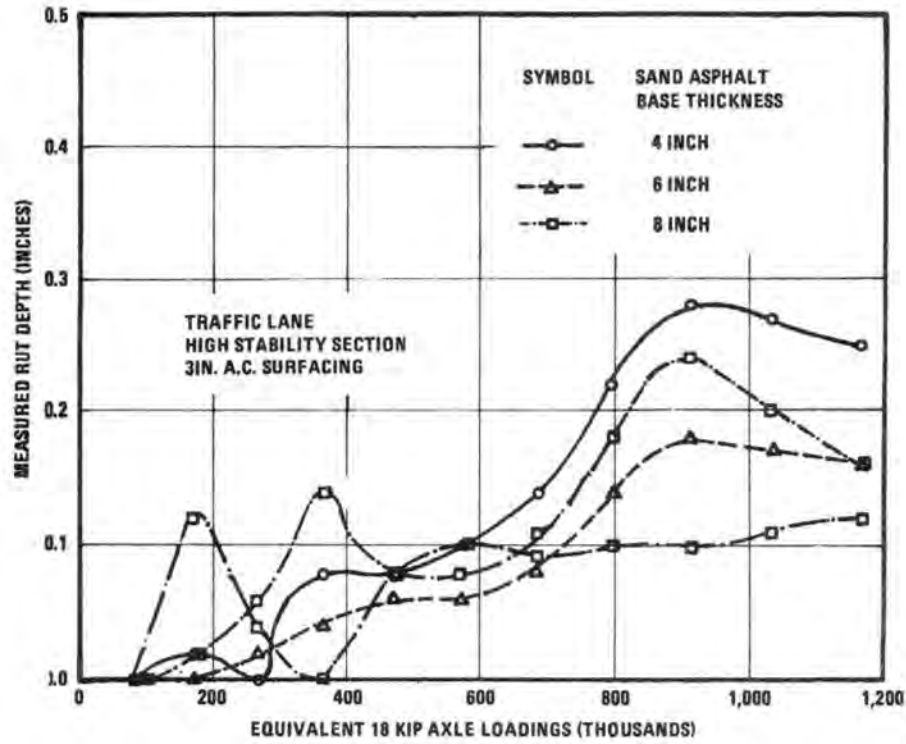
(a) TRAFFIC LANE



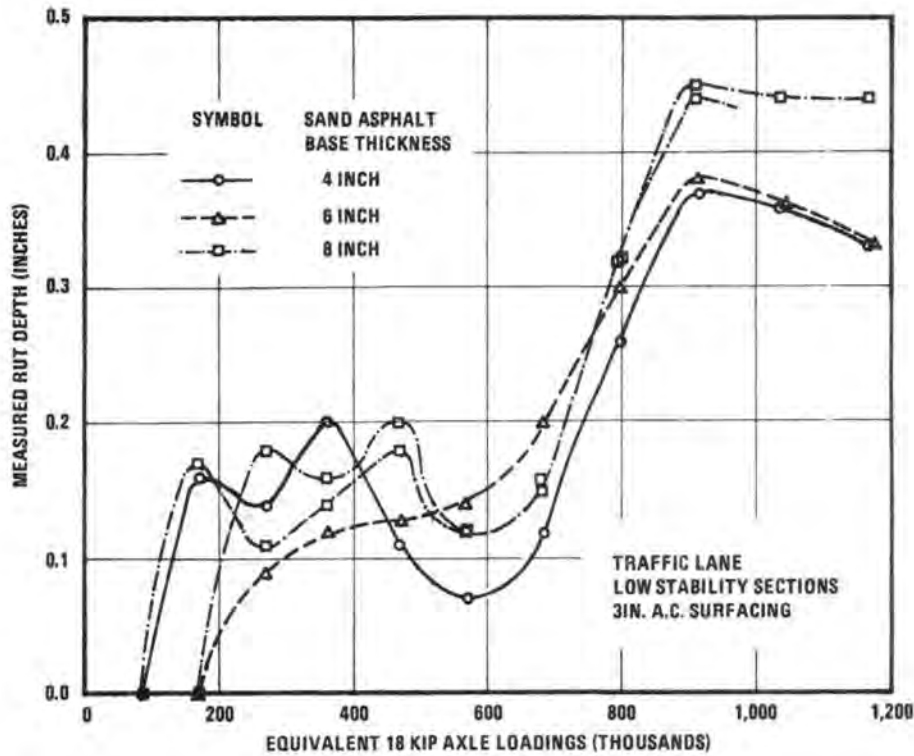
(b) PASSING LANE

MARIANNA TEST ROAD

FIGURE 2. VARIATION OF PAVEMENT PERFORMANCE WITH SAND ASPHALT BASE THICKNESS AND BASE STABILITY

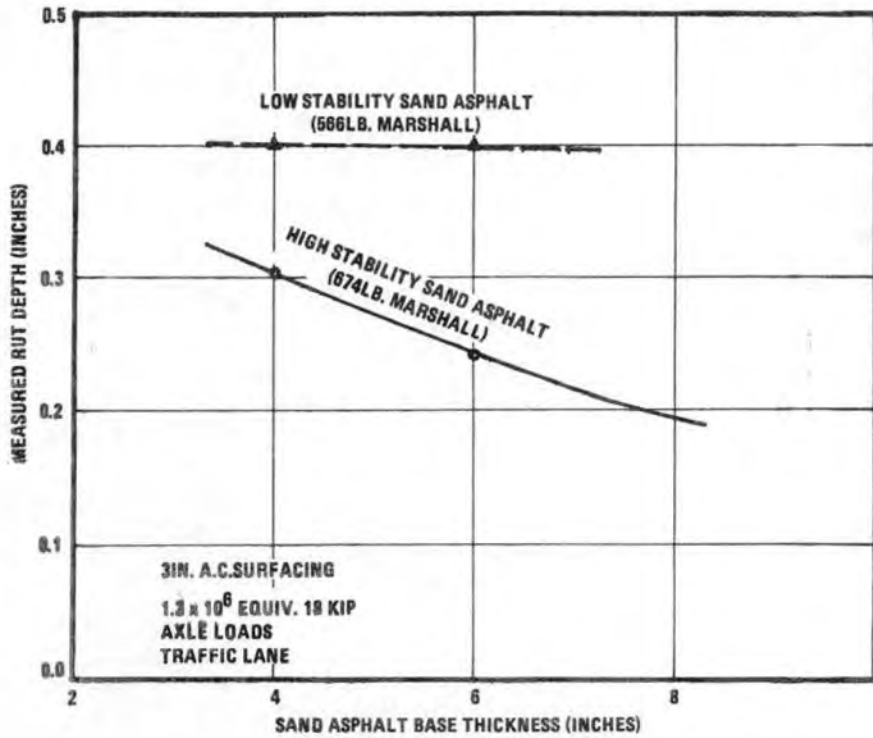


(a) HIGH STABILITY SECTIONS

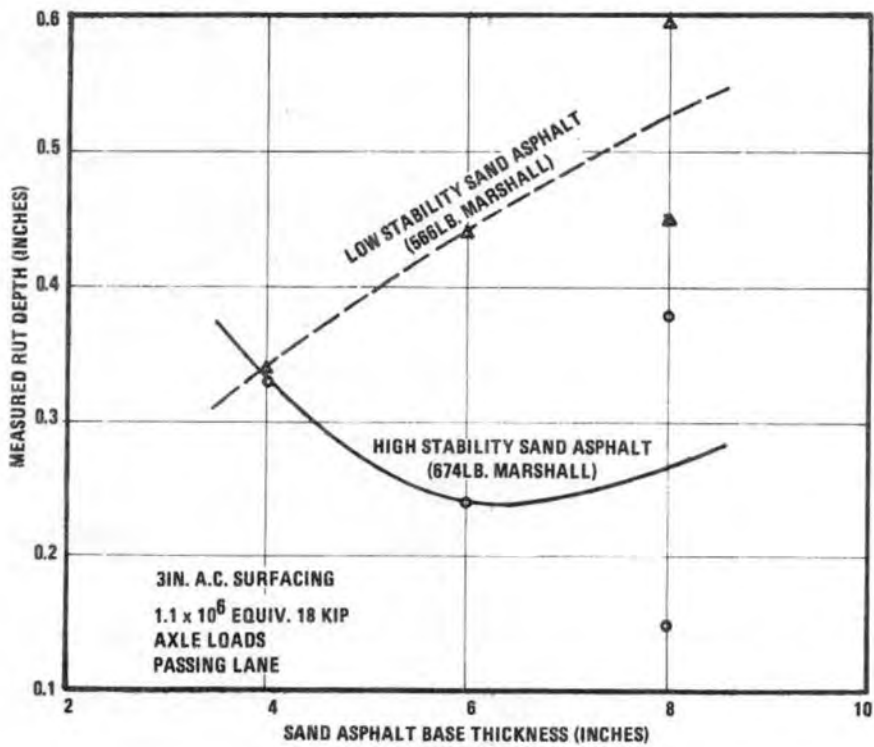


(b) LOW STABILITY SECTIONS

FIGURE 3. EFFECT OF TRAFFIC LOADING AND SAND ASPHALT BASE STABILITY ON RUT DEPTH AT MARIANNA TEST ROAD - TRAFFIC LANE



(a) TRAFFIC LANE



(b) PASSING LANE

FIGURE 4. RUT DEPTH AS AFFECTED BY SAND ASPHALT BASE THICKNESS, BASE STABILITY AND TRAFFIC/PASSING LANE - MARIANNA TEST ROAD

Table 1. Summary of Construction Materials Used at Marianna Test Road

| Gradation (% Passing) | Embankment ⁽¹⁾ | Subgrade ⁽⁴⁾ | Sand Asphalt | | Binder | Surface |
|------------------------------------|---------------------------|-------------------------|-------------------|------------------|--------|---------------------|
| | | | High Stability | Low Stability | | |
| 1-1/2 in. sieve | - | | | | 100 | |
| 1 in. sieve | - | | | | - | |
| 3/4 in. sieve | - | | | | 74.5 | |
| 1/2 in. sieve | - | | 100 | | - | 99.8 ⁽⁶⁾ |
| No. 10 sieve | 100 | 100 | 95 | | 21.4 | 34.5 |
| No. 40 sieve | 75.2 | 76.8 | 31 | | - | 20.9 |
| No. 60 sieve | 54.2 | 55.5 | 8 ⁽⁵⁾ | | - | - |
| No. 200 sieve | 21.8 | 18.9 | 4 | | - | 3.6 |
| Max. Dry Density (pcf) | 120.6 ⁽²⁾ | 122.5 ⁽³⁾ | | | | |
| Field Density (% max. dry density) | 99.8 | 103.6 | 125 (pcf) | 124.4 (pcf) | | 146 pcf |
| Asphalt ⁽⁷⁾ Content (%) | | | 6.4 | 6.9 | 4.5 | 5.0 |
| Marshall (lbs) Stability | | | 674 | 566 | 1694 | 1962 |
| Air Voids (%) | | | 11.4 | 16.0 | | |

1. Embankment Soil: 7.2 percent clay and 10.7 percent silt; Mean LBR value of 55.5.
2. AASHTO T-99 Maximum Dry Density
3. AASHTO T-180 Maximum Dry Density
4. Mean CBR values were as follows: High Stability Sections - 52.0; Low Stability Sections - 72.1
5. No. 80 U. S. Standard Sieve
6. No. 3/8 in. U.S. Standard Sieve
7. All asphalt cement was 60-70 penetration grade asphalt cement

with the number of 18 kip (80 kN) axle loads up to at least one million load repetitions.

Field observations from the Marianna Test Road indicate the following general trends:

1. All the sand-asphalt base sections at the Marianna Test Road performed well. Based on the PSI values, the sections with 8 in. (200 mm) sand-asphalt bases performed no better than sections with thinner bases and in several instances performed worse;
2. Average rut depth in low-stability sections was 0.43 in. (11 mm) and was 0.25 in. (6 mm) in high-stability sections. Therefore, for conditions existing at the Marianna Test Road, increasing a mix with a Marshall stability of 566 lbs. (2500 kN) to 674 lb. (3000 kN) resulted in a significant reduction in rut depth in the traffic lane;
3. The large difference in observed rut depths in low- and high-stability sections was not reflected in measured (Present Serviceability Index) values for the test sections;
4. Thickness of the sand-asphalt base for values between 4 and 8 in. (100 and 200 mm) did not significantly affect performance of the pavement as defined by the PSI; and
5. Rut depth was influenced by factors other than base thickness and Marshall stability, such as stiffness of the subgrade and other layers. As a result, no clear trend was observed between base thickness and rut depth.

The Marianna Test Road was constructed over a very stiff subgrade having a modulus of elasticity which was greater than the reported modulus of the sand-asphalt base. The presence of the stiff subgrade undoubtedly influenced the observed results and must be considered in extrapolating these results to other pavements. Rutting in low-stability sections was roughly proportional to the thickness of the base, while rutting in high-stability sections was inversely proportional to base thickness. This experimental finding indicated that a sand-asphalt base threshold strength may exist which separates widely varying rutting mechanisms. More field test results are needed to verify or disprove this finding.

Lake Wales Road Test

In late 1970, the Florida Department of Transportation completed construction of a flexible test pavement located at Lake Wales. The test road is composed of twenty test sections, half with sand-asphalt bases and half with limerock bases. Base thickness varied from 3 to 10 in. (75 to 250 mm). Both 1 1/2 in. (38 mm) and 3 in. (75 mm) thick asphalt concrete surface courses were used over the two bases. For comparable

sections, the rut depths in the traffic lane were found to be approximately twice as large in the sand-asphalt base sections as in the lime-rock sections as shown in Table 2 (on the average 0.43 in. compared with 0.24 in.). Nevertheless, PSI trends in the traffic lane of the sand-asphalt base sections were slightly greater than those with limerock bases as summarized in Table 3 (on the average 4.41 compared with 4.35). Figure 5 shows the PSI trends for the sections with sand-asphalt bases.

Total rut depths measured in the traffic lane of sand-asphalt sections were smallest for the section that had a 3 in. (75 mm) sand-asphalt base and reached a maximum value for sand-asphalt bases of approximately 4 in. (100 mm) in thickness for both the 1 1/2 in. (38 mm) and 3 in. (75 mm) asphalt concrete surface courses (Fig. 6). The rut depth became slightly less for base thicknesses greater than about 5 in. (125 mm). The variation in measured rut depth with number of 18 kip equivalent single axle loadings is shown in Fig. 7 for sections that had sand-asphalt bases. The rut depth in sections with sand-asphalt bases continued to increase significantly with wheel loadings up to at least 1×10^6 repetitions. At this time, the observed rut depths began to level off with additional wheel load applications.

The Lake Wales sand-asphalt base mix consisted of approximately 50 percent local sand and 50 percent crushed stone screenings. The original Hubbard Field mix design resulted in a recommended asphalt content of 7.5 percent with a corresponding Hubbard Field stability of 1283 (5.7 kN). The 50 blow Marshall laboratory density was 123 to 126 pcf (2.01 gm/cc) for all sections. The field nuclear density was relatively uniform for all sections varying from approximately 114.1 to 116.6 pcf (1.83 to 1.87 gm/cc) which corresponds to a density of only 92.7 percent of the Marshall design value. In the traffic lane of each section, mean Marshall stabilities varied from 340 to 528 lb. (1.5 to 2.3 kN), and mean asphalt content varied from 7.4 to 7.9 percent. The sand-asphalt mix had 50 to 57 percent passing the No. 40 sieve and 2 to 4 percent passing the No. 200 sieve.

Maryland Base Course Study

Stromberg [16,17] studied the performance of thirty-one pavements in Maryland with various base types including sand-asphalt, soil-cement, sand-aggregate, crushed stone, gravel and water-bound macadam. Pavement roughness, amount of cracked and patched areas, rut depth, and Benkleman beam deflections were determined in the field for each section. From these results the PSI was determined for each section and the Performance Index values were estimated by extrapolating PSI value trends. The Performance Index (PI) is defined as the logarithm of the number of 18 kip axle loadings tolerated by the pavement before the PSI value reaches 2.5.

The performance of pavement sections studied within Region II is summarized in Table 4 including structural information, cracking and patching, rutting, PSI, and the extrapolated number of 18 kip equivalent wheel loads that resulted in a PSI value of 2.5. Some of the properties of the sand-asphalt and soil-cement bases are given in Table 5. Extraction tests indicated that the sand-asphalt bases had an average asphalt content

Table 2. Measured Surface Rut Depths at the Lake Wales Test Road

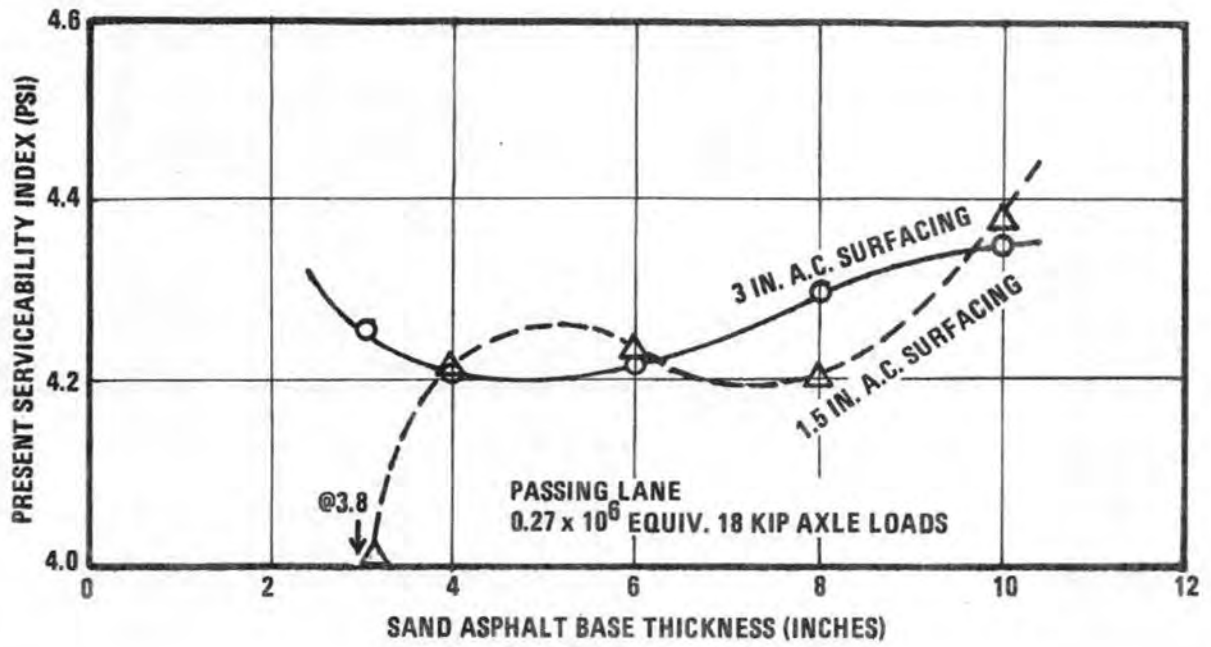
| Section Number | Layer Thickness | | Surface Def. (in.) ⁽¹⁾ | |
|----------------------------|-----------------|------------|-----------------------------------|--------------|
| | Surface (in.) | Base (in.) | Passing Lane | Traffic Lane |
| LIMEROCK BASE SECTIONS | | | | |
| 1A | 3 | 10 | .12 | .29 |
| 1B | 1.5 | 10 | .07 | .15 |
| 2A | 3 | 4 | .16 | .30 |
| 2B | 1.5 | 4 | .14 | .20 |
| 3A | 1.5 | 3 | .15 | .18 |
| 3B | 3 | 3 | .19 | .28 |
| 4A | 3 | 6 | .16 | .32 |
| 4B | 1.5 | 6 | .15 | .22 |
| 5A | 1.5 | 8 | .13 | .18 |
| 5B | 3 | 8 | .18 | .29 |
| Avg. | | | Avg. = 0.15 | Avg. 0.24 |
| SAND ASPHALT BASE SECTIONS | | | | |
| 6A | 3 | 3 | .24 | .38 |
| 6B | 1.5 | 3 | .21 | .25 |
| 7A | 1.5 | 6 | .17 | .43 |
| 7B | 3 | 6 | .24 | .53 |
| 8A | 3 | 4 | .25 | .53 |
| 8B | 1.5 | 4 | .18 | .42 |
| 9A | 1.5 | 8 | .18 | .41 |
| 9B | 3 | 8 | .22 | .51 |
| 10A | 3 | 10 | .27 | .42 |
| 10B | 1.5 | 10 | .27 | .38 |
| Avg. | | | Avg. 0.22 | Avg. 0.43 |

1. Rutting survey made December, 1974.

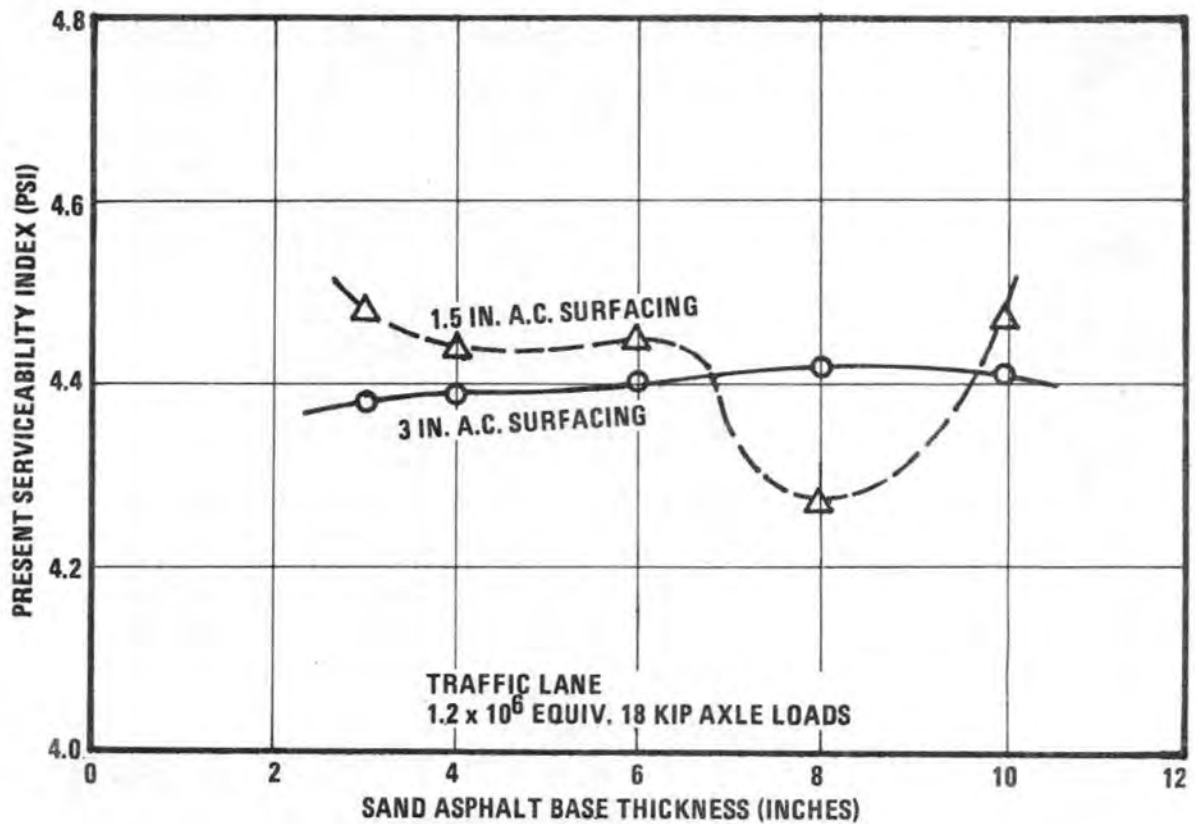
Table 3. Measured Present Serviceability Trends at the Lake Wales Test Road

| Section Number | Thickness (in.) | | PSI Value | | Psi Value ⁽¹⁾ | |
|-------------------|-----------------|------|--------------|--------------|--------------------------|--------------|
| | | | Passing Lane | Traffic Lane | Passing Lane | Traffic Lane |
| | Surface | Base | | | | |
| LIMEROCK BASE | | | | | | |
| 1A | 3 | 10 | 4.41 | 4.22 | 4.12 | 4.45 |
| 1B | 1.5 | 10 | 4.36 | 4.36 | 4.30 | 4.38 |
| 2A | 3 | 4 | 4.48 | 4.37 | 4.45 | 4.46 |
| 2B | 1.5 | 4 | 4.26 | 4.36 | 4.23 | 4.29 |
| 3A | 1.5 | 3 | 4.26 | 4.45 | 4.13 | 4.35 |
| 3B | 3 | 3 | 4.26 | 4.31 | 4.26 | 4.38 |
| 4A | 3 | 6 | 4.37 | 4.35 | 4.32 | 4.37 |
| 4B | 1.5 | 6 | 4.28 | 4.19 | 4.18 | 4.22 |
| 5A | 1.5 | 8 | 4.46 | 4.21 | 4.33 | 4.18 |
| 5B | 3 | 8 | 4.39 | 4.34 | 4.32 | 4.45 |
| Avg. | | | 4.36 | 4.32 | 4.26 | 4.35 |
| SAND ASPHALT BASE | | | | | | |
| 6A | 3 | 3 | 4.36 | 4.32 | 4.26 | 4.38 |
| 6B | 1.5 | 3 | 3.99 | 4.41 | 3.91 | 4.48 |
| 7A | 1.5 | 6 | 4.36 | 4.45 | 4.24 | 4.45 |
| 7B | 3 | 6 | 4.47 | 4.53 | 4.22 | 4.40 |
| 8A | 3 | 4 | 4.44 | 4.46 | 4.21 | 4.39 |
| 8B | 1.5 | 4 | 4.44 | 4.55 | 4.22 | 4.44 |
| 9A | 1.5 | 8 | 4.47 | 4.97 | 4.21 | 4.27 |
| 9B | 3 | 8 | 4.42 | 4.52 | 4.30 | 4.42 |
| 10A | 3 | 10 | 4.41 | 4.27 | 4.35 | 4.41 |
| 10B | 1.5 | 10 | 4.40 | 4.41 | 4.38 | 4.47 |
| Avg. | | | 4.38 | 4.49 | 4.23 | 4.41 |

1. The PSI values were determined in December, 1974.

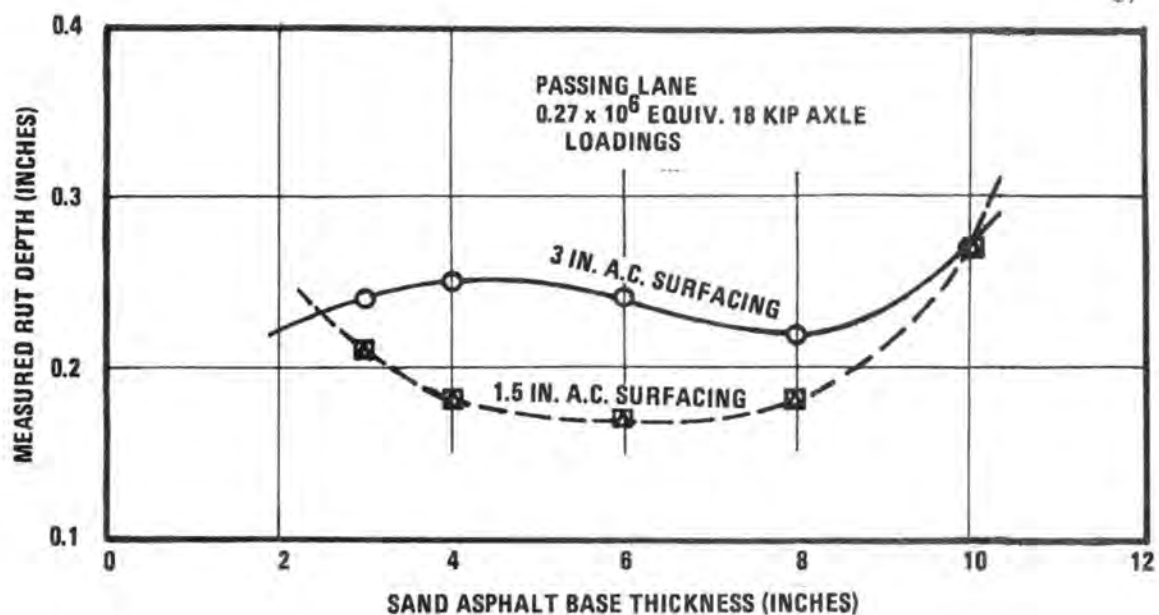


(a) PASSING LANE

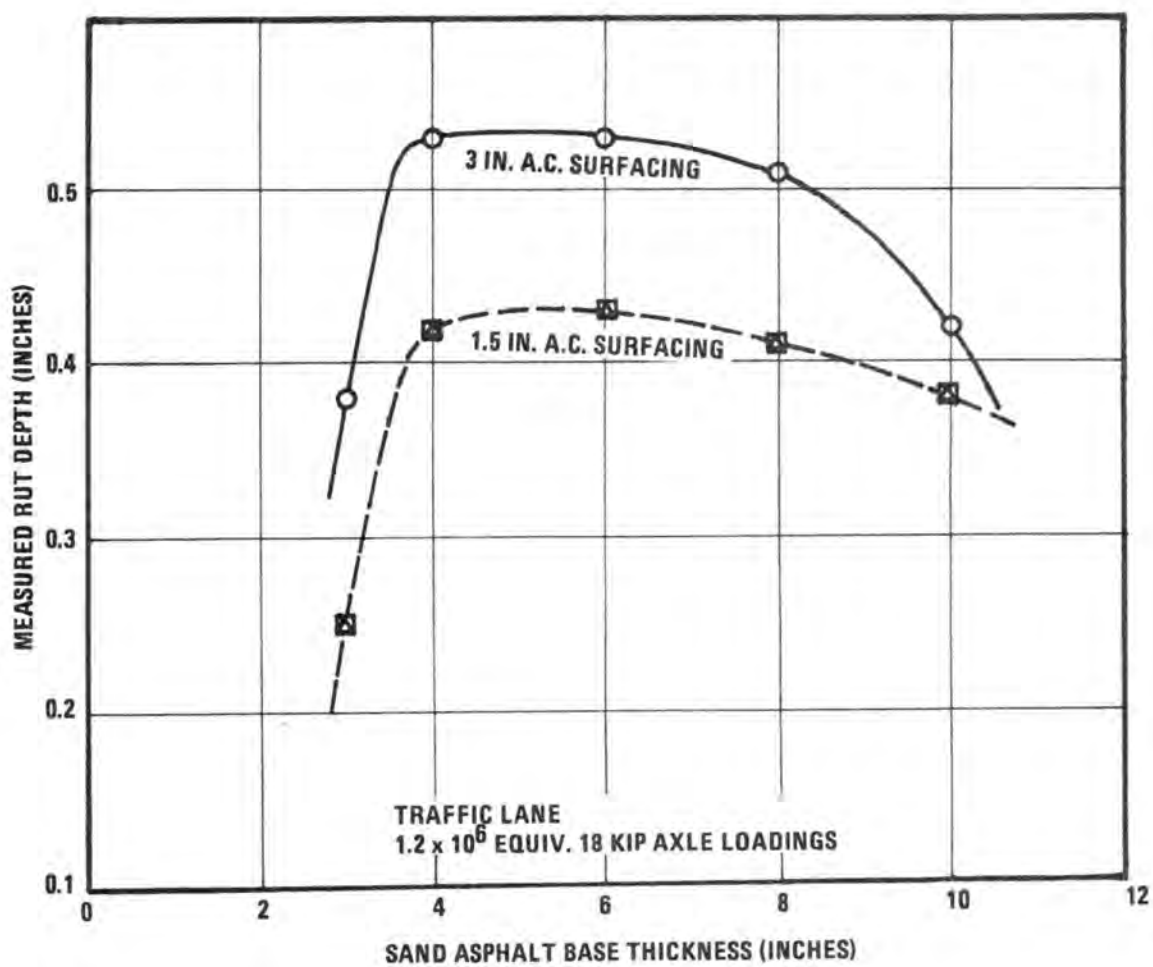


(b) TRAFFIC LANE

FIGURE 5. EFFECT OF ASPHALT CONCRETE SURFACE AND SAND ASPHALT BASE THICKNESS ON PERFORMANCE AS DEFINED BY THE PRESENT SERVICEABILITY INDEX - LAKE WALES TEST ROAD

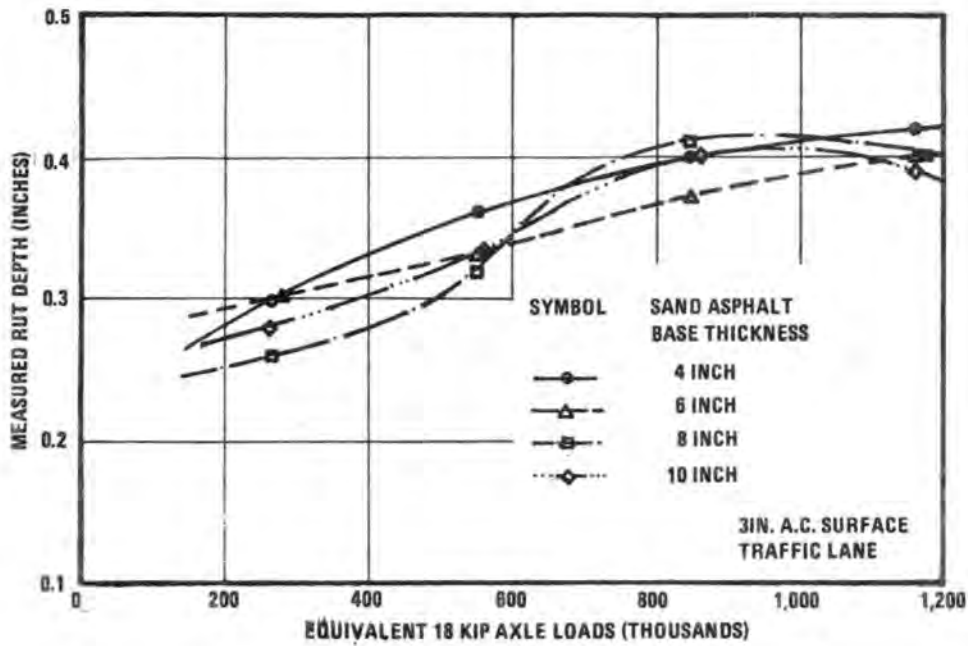


(a) PASSING LANE

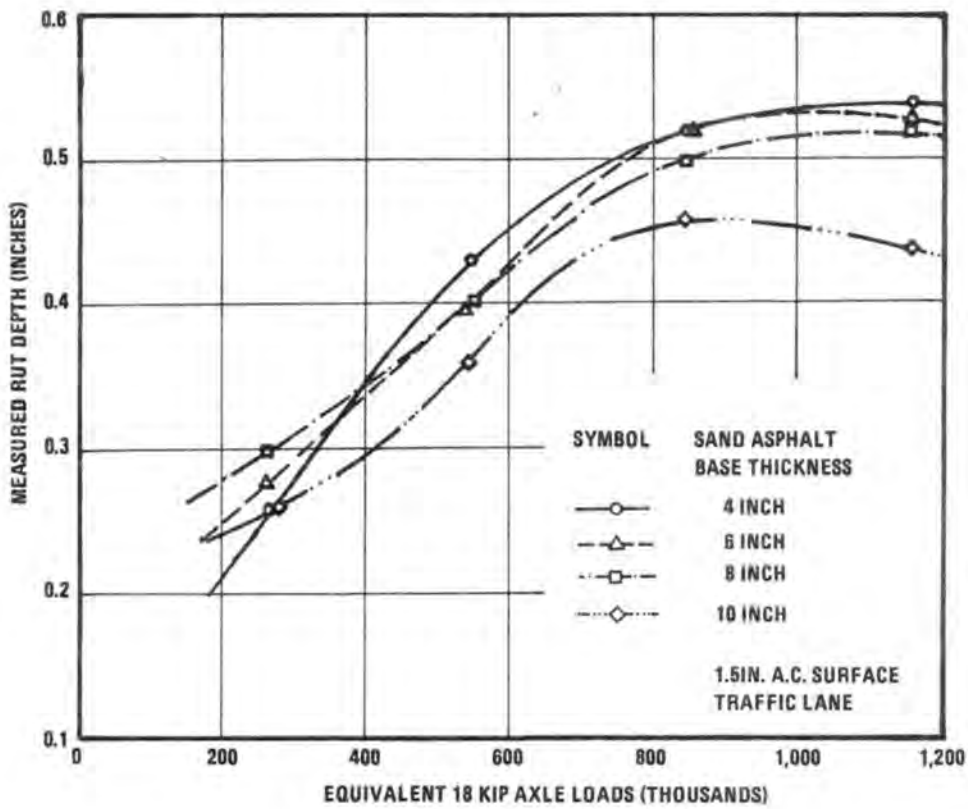


(b) TRAFFIC LANE

FIGURE 6. INFLUENCE OF ASPHALT CONCRETE SURFACE AND SAND ASPHALT BASE THICKNESS ON RUT DEPTH AT LAKE WALES TEST ROAD



(a) 1.5IN. ASPHALT CONCRETE SURFACING



(b) 3.0IN. ASPHALT CONCRETE SURFACING

FIGURE 7. EFFECT ON TRAFFIC LOADING ON RUT DEPTH AT LAKE WALES TEST ROAD - TRAFFIC LANE

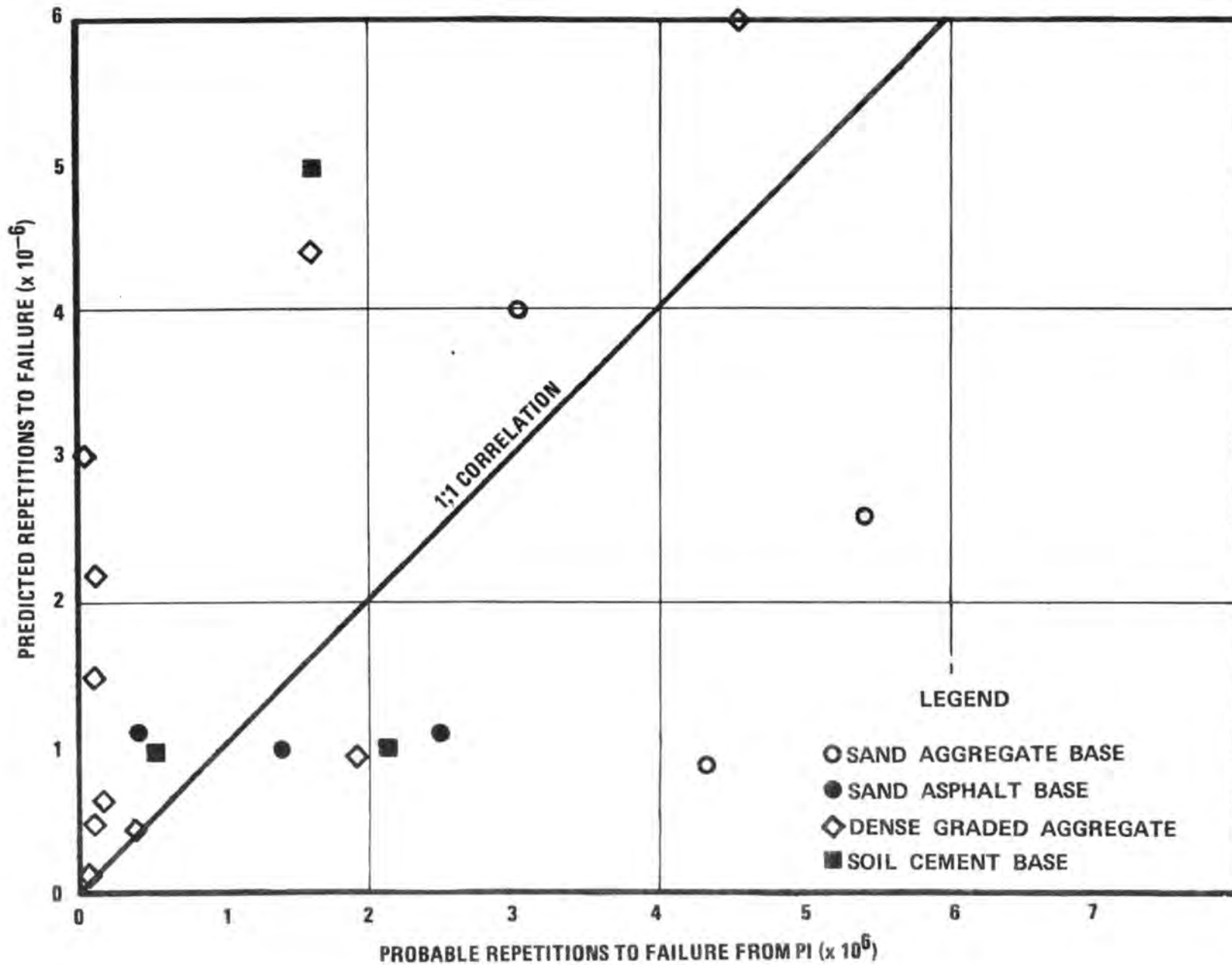


FIGURE 8. RELATIONSHIP BETWEEN OBSERVED AND PREDICTED PAVEMENT FAILURE FOR SELECTED MARYLAND PAVEMENTS

Table 4. Performance of Highway Pavements Constructed in Region II of Maryland [9]

| Sect. No. | Thickness | | | | CBR Subg. | C+P ¹ | Rut ² (in.) | A. C. Surface | | | Traffic | | AASHTO Design Guide | | | |
|-----------------|--------------------|------------------------|---------------|--|-----------|------------------|------------------------|----------------------|---------------|-----------------|-------------------|--|---------------------|----------------|---|---|
| | Surf. (in.) | Base (in.) | Subbase (in.) | Subgrade (in.) | | | | Voids (%) | I AC | Penet. (77°F) | Present PSI | Load ⁷ (X 10 ⁶) | SR ⁸ | S ⁹ | Pred. Load ¹⁰ (X 10 ⁶) | Extrap. Load ¹⁰ (X 10 ⁶) |
| 4 | 6.0 B | 6.0 SC | --- | A-1-b(0) A-4 | --- | 25 | 0.05 | 7.9 | 5.9 | 36 | 3.6(7) | 0.41 | 4.54 | | 8.8 | 1.6 |
| 17 ^B | 3.5 B | 7.0 SA | --- | A-4 A-2-4 | 27 | 38 | 0.04 | 6.2 | 5.6 | 33 | 3.8(7) | 0.02 | 3.50 | 6.6 | 7.3 | 0.04 |
| 18 | 3.0 B | 5.0 SA | --- | Varied | 10 | 5 | 0.03 | 6.1 | 5.7 | 33 | 3.7(9) | 0.15 | 2.58 | 4.9 | 0.31 | 1.4 |
| 19 | 3.5 B | 7.0 SA | --- | A-2-4 A-3 | 8 | 20 | 0.12 | 4.5 | 5.6 | 36 | 3.36 (6) | 0.49 | 3.72 | 4.4 | 1.8 | 2.5 |
| 8 | 4.0 B | 12.0 Stab. C | --- | Varied | 7 | 39 | 0.16 | 3.6 | 6.2 | 40 | 3.4(10) | 0.30 | 2.56 | 4.3 | 1.67 | 0.96 |
| 17 ^A | 3.5 B | 12.0 G | --- | A-2-4 A-4 | 27 | 5 | 0.05 | 6.2 | 5.6 | 33 | 3.46 (7) | 0.025 | 2.74 | 8.8 | 1.61 | 0.04 |
| 20 | 4.5 B | 12.0 G | --- | A-2-4 A-4 | 14 | 15 | 0.12 | 6.6 | 5.9 | 30 | 3.21 (7) | 0.10 | 3.04 | 5.5 | 1.31 | 0.22 |
| 21 | 3.0 B | 12.0 G | --- | A-4 A-2-4 A-1-b | 14 | 11 | 0.08 | 9.2 | 5.4 | 27 | 3.35 (8) | 0.11 | 2.45 | 5.5 | 0.35 | 0.16 |
| 5 | 3.5 B ³ | 10.0 WBM 2.0 screen | --- | A-2-4 | 10.5 | 3 | 3 | 3.0 | 5.4 | 28 | 2.5 (8) | 0.83 | 3.16 | 4.9 | 1.10 | 0.83 |
| 15 | 3.0 B 4.0 PM | 8.0 WBM 2.0 screen | --- | A-4 A-6 A-2-6 | 4 | 134 ⁴ | 0.16 ⁴ | 0.4 | 5.9 | 48 | 2.7(22) | 4.4 | 3.74 | 3.3 | 0.77 | 4.9 |
| 16 | 3.0 B 4.0 PM | 8.0 WBM 2.0 screen | --- | A-7-6 A-4 A-2-4 | 4.5 | 148 ⁵ | 0.22 ⁵ | 1.6 | 5.7 | 43 | 2.74 (22) 1969 | 4.8 | 3.41 | 3.5 | 0.48 | 5.4 |
| 9 | 4.75 B | 11.0 DGA | 6.0 CR | A-2-4 A-4(6) | 7.8 | | 0.15 | 2.2 | 5.7 | 50 | 3.58(10) | 1.5 | 5.18 | 4.3 | 13.9 | 7.6 |
| 11 | 5.0 B | 7.0 DGA | 6.0 CR | A-7-6(8) A-4 A-6 | 4.0 | | 0.10 | 2.3 | 5.8 | 45 | 3.63 (8) | 1.3 | 4.72 | 4.3 | 3.4 | 4.5 |
| 12 | 3.5 B | 12.0 DGA | 4.0 CR | A-7-6 A-2-4 A-4 A-1-b | 4.0 | | 0.22 | Avg. 3.9 (6.8) | 6.1 (5.88) | 41 (27) | 3.12(11) | 1.4 | 3.64 | 3.3 | 0.66 | 1.9 |
| 22 | 3.0 B | 7.0 DGA | --- | 6.0 CM A-7-5 A-4 A-1-b | 9.0 | 115 | 0.08 | 6.8 | 5.2 | 28 | 3.14 (7) | 0.03 | 2.79 | 4.7 | 0.38 | 0.05 |
| 23 | 3.5 B | 7.0 DGA | 4.0 CR | 6.0 CM A-2-4 A-4 A-5 A-7-6 | 4.0 | 0 | 0.06 | 3.7 | 5.8 | 38 | 3.75 (8) | 0.03 | 4.29 | 3.3 | 1.61 | 0.04 |
| 24 | 3.5 B | 7.0 DGA | 4.0 CR | 6.0 CM A-7-5 A-5 A-2-4 A-4 | 5.0 | 19 | 0.09 | 3.7 | 5.4 | 27 | 3.51 (7) | 0.07 | 3.39 | 3.6 | 0.55 | 0.10 |
| 25 | 4.5 B | 15.0 DGA | --- | A-2-4 A-1-b | 8.0 | 4 | 0.07 | 3.8 | 5.2 | 33 | 3.66 (6) | 0.11 | 3.94 | 4.4 | 2.48 | 1.6 |
| 26 | 3.5 B | 11.0 DGA | --- | 6.0 CM A-4 A-7-5 A-6 | 7.0 | 96 | 0.22 | 3.7 | 5.4 | 34 | 3.24 (7) | 0.09 | 3.08 | 4.3 | 0.55 | 0.17 |
| 27 | 6.0 B | 18.0 DGA | --- | A-2-4 A-4 A-1-b A-2-7 | 11.0 | 0 | 0.10 | 4.2 | 6.0 | 38 ⁶ | 4.04(3) | 0.06 | 5.23 | 5.1 | 22.5 | 0.33 |
| 28 | 4.0 B | 10.0 DGA | 4.0 CR | A-4 | 4.0 | 2 | 0.07 | 3.4 | 5.3 | 30 | 3.83(6) | 0.05 | 3.79 | 3.3 | 0.88 | 0.10 |
| 31 | 3.0 B | 4.0 DGA w/cement | 10.0 DGA | A-7-6 A-6 | 7.0 | 6 | 0.04 | 4.5 | 5.8 | 36 | 3.65(6) | 0.05 | 4.27 | 4.3 | 3.58 | 0.12 |

ABBREVIATIONS: B = ASPHALT CONCRETE SURFACING
SC = SOIL CEMENT
SA = SAND ASPHALT
G = GRAVEL
Screen = SCREENINGS
CR = CRUSHER RUN

WBM = WATER BOUND MACADAM
PM = PENETRATION MACADAM
DGA = DENSE GRADED AGGREGATE
CM = CEMENT MODIFIED

NOTES: 1. C+P = CRACKING PLUS PATCHING, SQ. FT./1000 SQ. FT. (1971).
2. AVERAGE RUT DEPTH IN 1971 (inches).
3. RESURFACED IN 1967.
4. OUT OF 10 PSI SITES, 4 WERE SURFACE TREATED BETWEEN 1969 AND 1970.
5. RUTTING, PATCHING, AND CRACKING MEASURED IN 1969; SURFACE TREATED IN 1970.
6. PENETRATION VALUE NOT INCLUDING TWO CORES WITH HIGH I AIR VOIDS.
7. EQUIVALENT 18 KIP SINGLE AXLE LOADS AT PSI RATING AND AGE IN PARENTHESES.
8. SR = AASHTO DESIGN GUIDE STRUCTURAL NUMBER.
9. S = AASHTO SOIL SUPPORT VALUE FROM SUBGRADE CBR.
10. LOAD PREDICTED BY AASHTO DESIGN GUIDE FOR PSI = 2.5; LOAD EXTRAPOLATED FROM PLOT OF PSI VS LOADINGS.

Table 5. Summary of Soil-Cement and Sand-Asphalt Materials Used in Region II of Maryland

| Section No. | Type Base | Fines (%) | Field Density (pcf) | Sand-Asphalt | | | | Sand-Cement |
|-------------|--------------|-----------|---------------------|--------------------|--------------------|--------------------------|-----------------|----------------------|
| | | | | A.C. (%) | Beam Flexure (psi) | Voids ⁽⁵⁾ (%) | Pa. A.C. (77°F) | Comp. Strength (psi) |
| 4 | Soil-Cement | - | 119.9 | - | - | - | - | 1653 |
| 17B | Sand-Asphalt | 3.7 | 132.1 | 4.4 | 245 ⁽¹⁾ | 13.8 | 40 | - |
| 18 | Sand-Asphalt | 3.9 | 134.0 | 3.8 | 269 ⁽²⁾ | 13.9 | 30 | - |
| 19 | Sand-Asphalt | 4.3 | 129.7 | 5.4 ⁽⁴⁾ | 338 ⁽³⁾ | 14.6 | 32 | - |

- Notes:
1. Asphalt concrete surfacing/binder had average flexural strength of 439 psi.
 2. Asphalt concrete surfacing/binder had average flexural strength of 500 psi.
 3. Asphalt concrete surfacing/binder had average flexural strength of 494 psi.
 4. Average of lower 3 of 5 asphalt contents was 4.1%.
 5. Obtained from extraction tests after traffic loading.

of 4 to 4.5 percent. Also, the sand-asphalt bases had 3.7 to 4.3 percent fines and void contents in the range of 13.8 to 14.6 percent. The soil-cement base had an average unconfined compressive strength of 1653 psi (11,400 kN/m²) which is high for this type base.

The relationship between performance predicted using the AASHTO Interim Guide [1] and the estimated performance obtained from extrapolating measured PSI values to 2.5 is given in Fig. 8. The AASHTO structural coefficients used in this comparison are given in Table 30 and were those used by the Maryland Department of Transportation during the base course study. A comparison of the predicted number of wheel load repetitions and those measured indicated that predicted pavement life was generally greater than life observed in the field. The soil-cement pavement sections typically consisted of a 6 in. (150 mm) soil-cement base placed beneath 6 in. (150 mm) of asphalt concrete. The sand-asphalt sections consisted of 3 to 3.5 in. (75 to 90 mm) of asphalt concrete above a 5 to 7 in. (130 to 180 mm) sand-asphalt base. Traffic loadings were on the order of 1 to 2 million 18 kip (80 kN) equivalent axle loads (Table 4). Two of the three sand-asphalt base pavements were found to perform extremely well, while the performance of the third section (17B) was below that predicted. The soil-cement sections showed reasonably good performance, although two of three pavements predicted the number of wheel load repetitions at about twice the probable number of repetitions (as predicted from the PI value) which gave a PSI value of 2.5. Due to shrinkage of the base, reflection cracks were present on the surface of soil-cement sections allowing water to penetrate the pavement, leading to more rapid deterioration. The performance of pavement sections that had dense-graded aggregate bases was less than predicted in almost every instance. In general, the section with sand-asphalt and soil-cement bases exhibited good performance when compared with the dense-graded aggregate base pavements.

After one million repetitions, rutting in the sand-asphalt base sections was found to be approximately the same as in the dense-graded aggregate base sections, and was approximately 0.15 in. (4 mm). Probably the relatively small amount of rutting in the sand-asphalt bases was due to:

1. an average asphalt content of only 4 to 4.5 percent,
2. a relatively high degree of compaction, and
3. a 3 to 3.5 in. (75 to 90 mm) asphalt concrete surface over the sand-asphalt base.

Of considerable practical importance is the fact that pavements with sand-aggregate bases were found to perform quite well. All three sand-aggregate bases studied performed either as well as or better than predicted. Also, the unstabilized sand-aggregate base sections had smaller rut depths (Fig. 9) than either sand-asphalt or dense-graded aggregate bases. This type base, which consists of local sands mixed with slag, should be studied more carefully for future use as a base material. Crushed stone could, of course, be used rather than slag. The sand-aggregate bases had about 88 percent of the aggregate smaller than 1/2 in. (13 mm) and 10 percent fines (Table 6). The two sections showing the best performance had an average field density of 95 percent of AASHTO

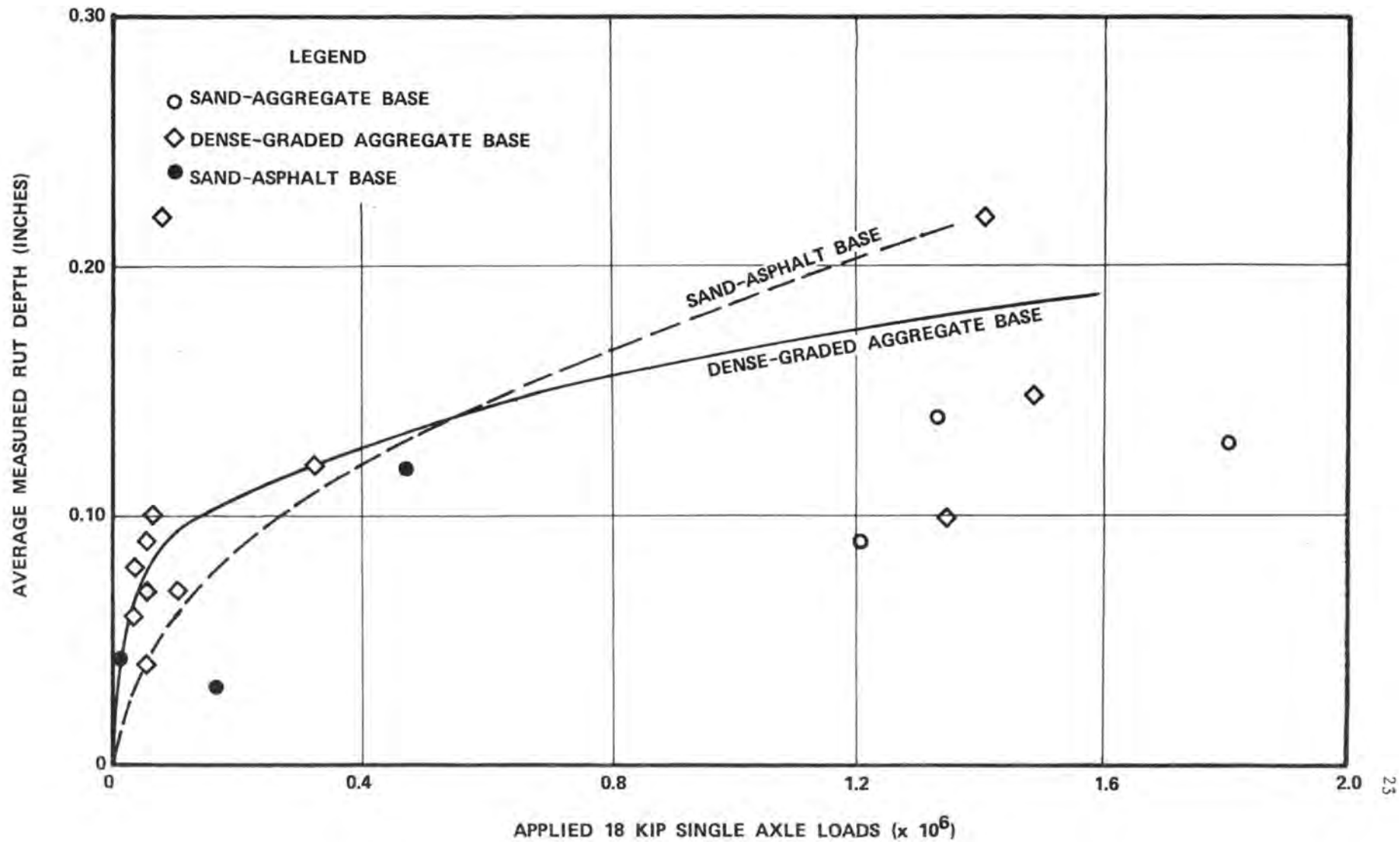


FIGURE 9. SURFACE RUTS DEVELOPED IN DIFFERENT TYPE BASES IN MARYLAND

Table 6. Summary of Material Properties of Sand-Aggregate Base Used in Region II of Maryland

| Section Number | Gradation | | | | | | | CBR Base | Compaction | | | |
|----------------|-----------|-----|----|----|----|----|-----|----------|-----------------------|------------|----------------|------|
| | | | | | | | | | Lab T-180 | | Field | |
| | 3/4 | 1/2 | 4 | 10 | 40 | 60 | 200 | | Max. γ_d (pcf) | Opt. w (%) | γ_d (%) | w(%) |
| 1 | 100 | 88 | 75 | 72 | 46 | 29 | 10 | 45 | 129.4 | 7.7 | 96 | 6.7 |
| 2 | 99 | 88 | 70 | 67 | 41 | 21 | 9 | 32 | 128.6 | 8.2 | 93 | 6.6 |
| 3 | 98 | 86 | 68 | 66 | 41 | 33 | 11 | 22 | 129.8 | 7.8 | 87 | 7.8 |

T-180 (modified Proctor) density and an average CBR value of 39; the poorer performing section had a base with an average AASHTO T-180 density of 87 percent and an average CBR of 22. Results indicated the use of unstabilized blends of sand and aggregate should be considered for use on low to moderately trafficked pavements as a low-energy intensive alternative to presently used stabilized base materials.

Washington State University Test Track

An extensive base course study was conducted at the Washington State University Test Track [19] involving:

1. sand-asphalt,
2. crushed stone base,
3. emulsion treated crushed stone,
4. special asphalt treated aggregate, and
5. asphalt concrete base.

A circular test track was used which had a diameter of 83 ft. (25 m). Each pavement section was 8 ft. (2.4 m) in width and varied from 18 to 43 ft. (5.5 to 13 m) in length. All sections had a 4 ft. (1.2 m) wide shoulder on each side. The pavement sections were subjected to a moving 10.6 kip (47 kN) dual wheel loading. The test sections had either a 3 or 4.25 in. (75 or 110 mm) thick asphalt concrete surface with base thicknesses varying from 2 to 12 in. (50 to 305 mm). Pavement sections were constructed over a clayey silt subgrade (A-6 Soil Classification) having a liquid limit of 35 and a plastic limit of 15 with the CBR value varying from 2.2 to 13.5.

A summary of properties of materials used in the test track is given in Table 7. The sand used in the sand-asphalt base was a clean, uniform white sand having 3 percent fines. The sand-asphalt was laid in 4 in. (100 mm) loose lifts at a temperature between 195°F and 250°F (90° and 121°C). The sand-asphalt base was compacted immediately using a steel wheel roller followed by a pneumatic roller, and then a steel wheel roller. The steel wheel roller caused tension surface cracks to develop if the mix cooled too much (Fig. 10).

A summary of base course equivalencies obtained from the test track is given in Table 8 and measured rut depths are summarized in Table 9. In the fall when the subgrade was not saturated, sand-asphalt was found to have approximately the same strength as the asphalt concrete base. However, in the spring when the clayey silt subgrade was saturated, approximately 1.6 in. (40 mm) of sand-asphalt was found to be equivalent to 1.0 in. (25 mm) of asphalt concrete. The untreated crushed stone base performed better when compared with the asphalt concrete base in the spring than the fall, having equivalencies of 2.38 and 4.76 respectively.

Different failure modes were found to occur in spring than fall. In fall, transverse fatigue cracks first developed in the thin sections which eventually led to a general alligator crack pattern. Cracking began after a wet, cold period followed by warmer weather. In spring, a punching (rutting) type failure occurred very rapidly due to the saturated subgrade conditions present. In the thickest sections, severe

Table 7. Summary of Materials Used at Washington State University Test Track [25].

| Item | Crushed Stone Base | Emulsion Treated Crushed Stone Base (2) | Asphalt ⁽¹⁾ Treated Special Aggr. (2) | Asphalt Concrete Base (Class F) | Sand Asphalt Base | Asphalt Concrete ⁽¹⁾ Surface Class B (2) |
|---------------------------------|----------------------|---|--|---------------------------------|---------------------|---|
| Aggregate Gradation (% Passing) | | | | | | |
| 1-1/2 in. sieve | | | 100 | 100 (3/4) | | 100 |
| 1 in. sieve | 100 | 100 | 95 | | | 100 |
| 5/8 in. sieve | 60 (1/4) | 99 | 89 | 93 | 100 (1/4) | 100 |
| 1/2 in. sieve | 32 | 59 | 41 | 36 | 100 | 42 |
| No. 10 sieve | 13 | 25 | 20 | 9 | 33 | 23 |
| No. 40 sieve | 4 | 9.2 | 7.6 | 5 | 3 | 9 |
| No. 200 sieve | | | | | | |
| Max. Dry Density (pcf) | 137.5 ⁽⁵⁾ | 133.5 | 143.2 | 146.6 | 128.1 | 149.4 |
| Field Density (% max. dry) | 103.4 | | 96.6 | 95.3 ⁽⁴⁾ | 88.6 ⁽³⁾ | 98.2 |
| Asphalt Content (%) | | 2.9 | 3.0 | 5.8 | 5.3 | 5.7 |
| Cement Content (%) | | | | | | |
| 7-day Comp. strength (psi) | | 17 | | | | 1.2 |
| Voids (%) | | | | | | |
| Fractures (%) | 100 | | 3 | 72 | | 75 min. |
| Sand Equivalent (specified) | 40 min. | 40 | 35 | 45 | 30 | 45 |
| Asphalt Grade | | SS-kh | 60-70 | 85-100 | 85-100 | 60-70 |

Materials Identification: Subgrade: clay-silt, LL=34.9, PL=14.7, A-6(10), Max. dry density=108 pcf, Optimum moisture=18.8% (ASTM D-698), R-value = 16, CBR = 2.2 to 13.5

1. 60-70 penetration grade asphalt
2. Values from Ring 3
3. % of specified 128.1 pcf density
4. % of specified 146.0 pcf density
5. Standard Proctor Test

Table 8. Base Course Equivalencies Determined at the Washington State Test Track [19]

| Base 1 Type | Ring 1 ⁽¹⁾ | | Ring 2-4 ⁽²⁾ | |
|--------------------------------|-----------------------|---------|-------------------------|--------|
| | Initial Cracking | Failure | Fall | Spring |
| Asphalt Concrete | 1.0 | 1.00 | 1.0 | 1.0 |
| Special Agg. Asphalt-Treated | 0.93 | 1.00 | 1.0 | 1.0 |
| Cement Treated Crushed Stone | 1.95 | 1.90 | - | - |
| Emulsion Treated Crushed Stone | | | 1.52 | 1.79 |
| Sand-Asphalt | | | 1.0 | 1.60 |
| Crushed Stone | | | 4.76 | 2.38 |

1. Saturated Subgrade Condition; 4.25 inches of asphalt concrete wearing course.
2. 3 in. of asphalt concrete wearing course

Table 9. Rut Depths Observed at Washington State University Test Track After 177,501 Wheel Load Applications⁽¹⁾

| Type Base | Base Thickness | Average Rut Depth (in.) |
|------------------|----------------|-------------------------|
| Sand-Asphalt | 6 | 1.36 |
| Sand-Asphalt | 8 | 0.675 |
| Asphalt Concrete | 5 | 0.663 |
| Crushed Stone | 12 | 0.438 |

1. For a 3 in. asphalt concrete surfacing; 143,370 wheel applications were applied in the fall with remaining applications applied in the spring.

Table 10. Recommended Relative Structural Coefficients
for Selected South Carolina Base and Subbase
Materials Δ [21]

| Pavement Components | Base Coefficients (a_2) Layer Thickness (in.) | | | | |
|---|--|------|---------------------|------|------|
| | 4 | 6 | 8 | 10 | 15 |
| I. Base (Under 3 in. Asphalt Concrete Surface) | | | | | |
| Bit. Stabilized Macadam (on sand-clay, soil) | 0.28 | 0.30 | 0.32 | 0.34 | |
| Bit. Stabilized Macadam (on Cem.-mod. soil) | 0.34 | 0.36 | 0.38 | 0.40 | |
| AASHO Dolomitic Limestone | 0.19 | 0.16 | 0.13 | 0.11 | |
| Type 2 Macadam - Crushed Granite-Gneiss (On weak support) ^x | 0.16 | 0.13 | 0.11 | 0.09 | |
| (On firm support) ⁺ | 0.11 | 0.09 | 0.07 | 0.06 | |
| Sand-Asphalt | 0.23 | 0.22 | 0.21 | 0.20 | |
| Soil Stabilized Aggregate | 0.23 | 0.21 | 0.19 | 0.17 | |
| Fossiliferous Limestone | 0.21 | 0.21 | 0.21 | 0.21 | |
| Cement Stabilized Macadam (3% cement) ^o | 0.34 | 0.36 | 0.38 | 0.40 | |
| Sand-Clay | 0.21 | 0.20 | 0.19 | 0.18 | 0.16 |
| Cement-modified Soil (4.6% cement) [*] | 0.23 | 0.25 | 0.27 | 0.29 | 0.34 |
| II. Subbase (Under 3 in. A.C. Surface and 5 in. Base) | | | | | |
| Sand-Clay | | | 0.16 | 0.14 | 0.12 |
| Cement-modified Soil (Under Bit. Stabilized Macadam Base) | | | 0.28 | 0.33 | 0.38 |
| (Under Type 2 Macadam Base) | | | 0.17 | 0.22 | 0.26 |
| (Under Other Flexible Bases) | | | 0.24 | 0.28 | 0.32 |
| III. Subbase (3 in. A.C. Surface and 10 in. Base) | | | | | |
| Sand-Clay | | | 0.12 ⁽¹⁾ | 0.10 | 0.08 |
| Cement-modified Soil (Under Bit. Stabilized Macadam Base) | | | 0.32 | 0.36 | 0.40 |
| (Under Type 2 Macadam Base) | | | 0.10 | 0.12 | 0.14 |
| (Under Soil Sta. Agg. or Sand-asphalt Base) | | | 0.16 | 0.18 | 0.20 |
| (Under AASHO or Fos. Limestone Base) | | | 0.20 | 0.25 | 0.30 |

1. These structural coefficients are for a base thickness of 5 in.

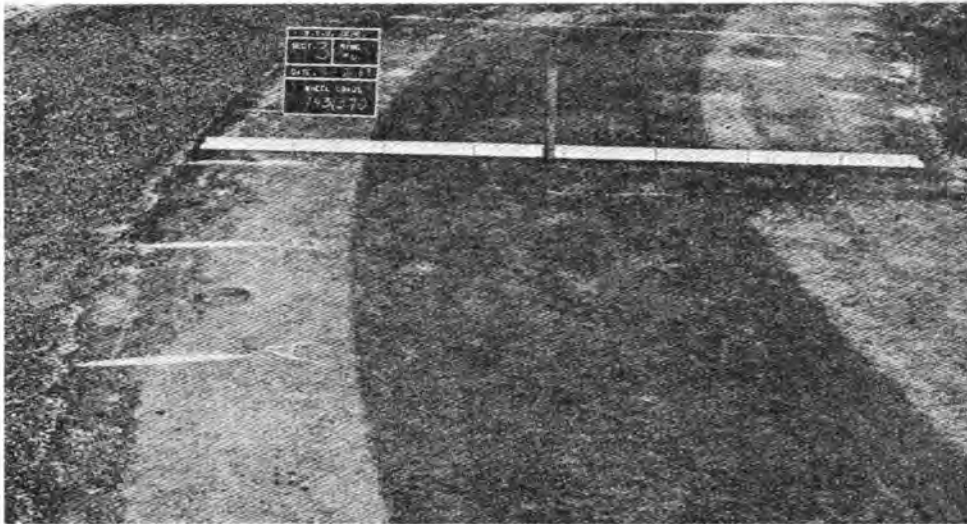
x Weak support is assumed if Structural Number of subbase plus one-third of subgrade Soil Support Value is 2.3 or less.

+ Firm support is assumed if Structural Number of subbase plus one-third of subgrade Soil Support Value is 3.0 or more.

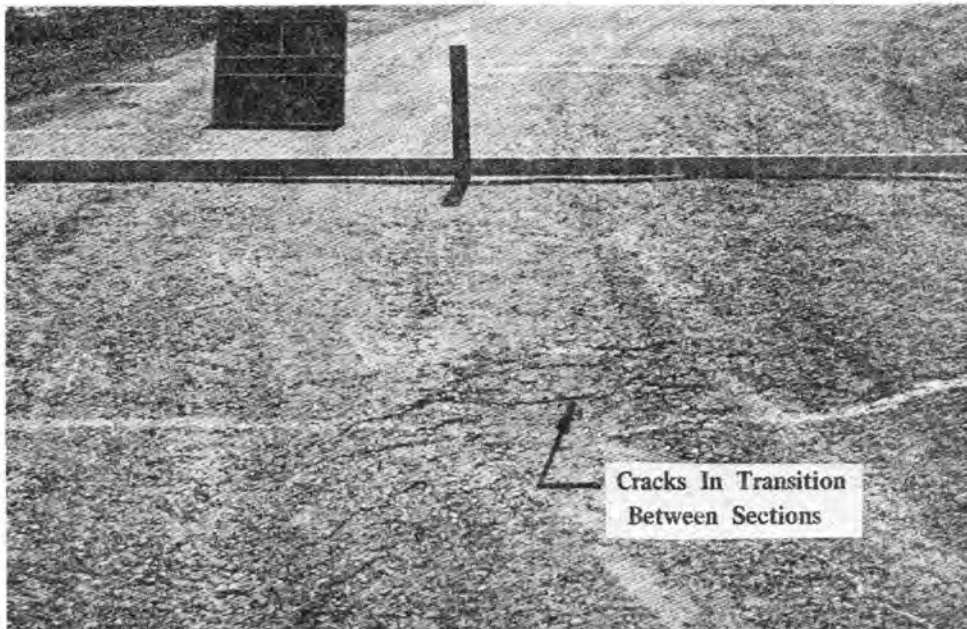
o Values have been reduced by 50% because of layer rigidity.

* Soil used was 50%/50% mixture of "B" and "C" horizon Cecil Sandy Loam.

Δ Relative strength values of similar materials having different plastic properties will differ from those recommended in this table.



(a) Condition in April After 143,370 Wheel Loads - Before Spring Loading



(b) Condition in May After 162,774 Wheel Loads

FIGURE 10. SAND-ASPHALT 6 in. BASE SECTIONS WITH 3 in. ASPHALT CONCRETE SURFACING - WASHINGTON STATE TEST TRACK

rutting developed but cracking was not observed in these sections. The average rut depth in the 8 in. (200 mm) thick sand-asphalt section was found to be 0.7 in. (18 mm) which was essentially the same as in the section with a 5 in. (130 mm) thick asphalt concrete base. The section with an 8 in. (200 mm) sand-asphalt base exhibited about one-half the rut depth as a section with a 6 in. (150 mm) sand-asphalt base. Probably the large difference in rut depths between 6 and 8 in. (150 and 200 mm) sand-asphalt base sections was due partially to variations in the very soft subgrade conditions encountered at the test track site. The difference in failure modes in fall and spring indicated the important effect that environmental conditions, including a saturated subgrade, had on pavement performance. The sand-asphalt base which had an asphalt content of 5.3 percent was placed at 88.6 percent of the 128.1 pcf (2050 kg/cm³) design density. In comparison, the asphalt concrete base which had an asphalt content of 5.8 percent was compacted to 95.3 percent of the 146.6 pcf (2350 kg/cm³) design density. The sand-asphalt base which consisted of a uniform, clean sand was compacted to a relatively low density. Therefore, the sand-asphalt would not be expected to perform as well as a better graded sand having a higher percent fines and compacted to 95 percent of the design density.

Three Rivers Test Road

In Minnesota, the U.S. Forest Service constructed twelve 1,000 ft. (300 m) long test sections using sand-asphalt [20]. The sand used in this investigation was uniformly graded and did not meet standard highway specifications. This sand had a dry density (AASHTO T-180) of 118 pcf (1890 kg/m³), and less than 2 percent passed the No. 200 sieve. Test sections having sand-asphalt surfacings 3.5, 5 and 8 in. (90, 130 and 203 mm) thick were stabilized with both a 250 to 300 penetration grade asphalt cement and a medium curing, MC-800 cutback asphalt. The test road was placed over a 24 in. (600 mm) prepared subgrade. The underlying natural subgrade was predominantly sand but some sections were underlaid by silty and clayey sands. Specimens used to design the pavement were prepared using 80 blows of the Marshall hammer on each side, with a resulting average density of approximately 120 pcf (1800 kg/m³). An asphalt content of 5 and 7 percent AC-250 to 300 asphalt-cement and 7 and 9 percent MC-800 was used in the test sections.

All sections were found to be in good condition after they were subjected to traffic for two years. The average Present Serviceability rating of the cutback sections was 3.1 compared with 3.78 for the sand-asphalt sections. Surface thickness was found to have minimal effect on observed rut depth for the MC and AC sand-asphalt sections. The 250 to 300 penetration sand-asphalt sections had slightly less rutting than the MC cutback sections with average values for all sections being 0.1 in. (2.5 mm) and 0.07 in. (1.8 mm), respectively.

Clemson University Base Course Study

Bushing *et al.* [21] studied the performance of six base course materials in a full-scale laboratory investigation. The following base materials were studied in this investigation: sand-asphalt, bituminous

stabilized macadam, cement stabilized macadam, dolomitic limestone, crushed granite-gneiss and soil stabilized aggregate. Tests were performed using a 3 in. (76 mm) thick asphalt concrete surfacing on two subgrades having effective moduli of subgrade reactions of 50 pci (1.4 kg/cm³) and 257 pci (7.1 kg/cm³). After a stationary dual wheel was subjected to 1,500 load repetitions, the deflection was measured as a function of load up to either 9,000 lbs. (40 kN) or 0.02 in. (0.5 mm), whichever was reached first.

Base course coefficients for use in the AASHO Interim Design Guide [18] were developed based on the deflection relationships observed for the test pavements. Recommended base course coefficients determined from this study are summarized in Table 10. For an 8 in. (200 mm) thick base, the base course coefficient was found to be 0.21 for the sand-asphalt, 0.27 for the cement modified soil (4.6 percent cement content), and 0.32 for the bituminous stabilized macadam. The base course coefficients developed during this investigation were based on the resilient deflection. Long-term effects such as rutting, fatigue, water and weathering effects were not directly considered.¹ Results of a comprehensive field study of pavements performed by Stromberg [16] indicated that a direct relationship does not appear to exist between deflection and long-term performance, although the results of the relatively short-term AASHO Road Test indicated that a relationship might exist. The results of this study, therefore, should be used with considerable caution in evaluating the general trends in performance between the different bases studied.

Laboratory Studies

Walker [2] investigated the use of pit sands in sand-asphalt bases. Marshall stabilities of the sand-asphalt mixes studied varied from approximately 200 to 600 lbs. (0.9 to 2.7 kN). Repeated load triaxial tests and fatigue tests were performed to evaluate potential performance of the material under simulated field stress conditions. Fatigue tests were performed on rectangular beam specimens supported only at the ends. Walker concluded that sands having stabilities of approximately 500 to 600 lbs. (2.2 to 2.7 kN) had fatigue properties which compared favorably with presently used crushed stone concrete base materials. Walker's results, however, were based on a direct comparison of fatigue curves which led to erroneous conclusions if the stiffness of the mix was not properly considered [6,54].

Gap-Graded Surface Mixes

Gap-graded surface mixes generally consist of a combination of coarse aggregate and sand in such proportions that the gradation curve

¹The small resilient deflections observed for the soil cement sections were apparently arbitrarily increased to crudely consider the long-term deterioration of this type pavement due to cracking and weathering.

has a gap in it. A mineral filler frequently is used in the gap-graded mixes with a relatively hard binder. In addition to utilizing local materials, gap-graded mixes generally have been reported [22-24] to have better workability and fatigue resistance than conventional mixes, and have an adequate level of stability and durability as well. In England, about 60 percent of the major highways are constructed with gap-graded mixes, and all of the freeways in South Africa are presently being built with this type mix [23,24].

Until recently, most gap-graded mixes were designed according to the well-known British Standard BS594:1961 [25]. Following the recipe approach given in BS594, wearing courses generally had the following composition:

| <u>Material</u> | <u>Content (%)</u> |
|-----------------|--------------------|
| Stone | 30 |
| Sand | 53 |
| Binder | 8 |
| Filler | 9 |

The binder commonly used in Great Britain consisted of a blend of bitumen with pitch or lake asphalt and had a penetration of about 50 at 25°C (77°F). In 1973 BS594 was changed to include a gap-graded mix design method based on the Marshall Stability test.

An angular, fine aggregate was found by Price, Please, Lees and Duthie as reported by Acott [22] to give higher stability mixes than rounded fine aggregates. The shape of the coarse aggregate apparently influenced stability and rutting less than the shape of the fine aggregate. Wheel tracking tests and Marshall Stability tests, however, did indicate that mixes that had a rounded gravel as the coarse aggregate underwent more rutting than mixes that used a crushed rock with a more angular shape.

Considerable research [26,27,28] has been devoted to developing ideal gradation curves for continuously graded mixes. Lees [29] concluded that an ideal grading curve did not exist since the minimum voids in a mix were influenced by particle shape, degree of compaction, and lubricating effects. Further, Hveem [27] found that in some instances irregular grading curves were associated with pavement that had shown good performance.

The gradation requirements for gap-graded mixes specified by BS594:73 are summarized in Table 11. The allowable amount of coarse aggregate varied with the layer thickness. Coarse aggregate stone contents up to 55 percent were permitted for a 2 in. (50 mm) surface course. For 30 percent stone mixes, Duthie [30] found that mix with the largest coarse aggregate had a lower optimum binder content. Further, an increase in stone content increased the density and stability, and decreased the optimum binder content. Tracking tests [24] indicated an increase in stone content resulted in a decrease in rut depth. Marshall stabilities were found to increase with stone content increases up to about 60 percent.

Table 11. Gap-Graded Surface Mix - Fine and Coarse Aggregate Grading Requirements for BS 594⁽¹⁾[22]

| DESCRIPTION | | WEARING COURSE MIXTURES | | | | |
|--------------------------------|-------------------|---|----------|----------|-------------------------|---|
| Nominal thickness of layer, mm | | 25 | 35 | 40 | 50 | |
| Coarse aggregate content, % | | 0 | 15 or 30 | 30 | 40 or 55 ⁽²⁾ | |
| Nominal size of aggregate, mm | | Not Applicable | 10 or 14 | 14 or 20 | 20 | |
| BS Sieve, (mm) | | Coarse Aggregate (% by Weight Passing) | | | | Fine Aggregate (% by Weight Passing) |
| Coarse Agg. | Fine Agg. | Not Applicable | | | | |
| 50 | 5.0 | | | | | 100 |
| 37.5 | 2.36 | | | | | 95 - 100 |
| 28 | 600 μm | | | 100 | 100 | 75 - 100 |
| 20 | 212 μm | | 100 | 85 - 100 | 85 - 100 | 15 - 60 |
| 14 | 75 μm | | 85 - 100 | 0 - 100 | 0 - 60 | 0 - 5 |
| 10 | | | 0 - 100 | 0 - 60 | - | |
| 6.3 | | 0 - 60 | - | - | | |

1. Material retained on a 2.36 mm sieve
2. 40 only for gravel or slag

BS594:73 required the amounts of soluble binder and mineral filler specified in Table 12 [22] for the wearing course. Further, BS594:73 required the mineral filler to consist of limestone or Portland cement with at least 85 percent of particles passing the 75 μ m sieve. For 55 percent stone, 6.4 percent soluble binder and 6.4 percent mineral filler (material passing the 75 μ m sieve) were required (Table 12). Further, BS594 required an increase in binder content as mineral filler increased. Several studies using the Marshall test indicated, however, that maximum Marshall Stability was reached at lower binder contents with increased filler content [29,30,31].

Acott [22] gave a thorough review of design methods for gap-graded mixes. Some methods reviewed include a modified vibrating hammer test, a rational gradation design method, and the Shell Method proposed by Brien [32]. The new BS549:1973 specification that was also reviewed used a modified Marshall procedure that tested only the fine aggregate and hence gave misleading results. Probably the best designed method presently available for gap-graded mixes was proposed by Marais [22,24]. The procedure was based on the Marshall Mix Design Method and used the complete gap-graded sand-stone mix. The design requirements proposed by Marais are summarized in Table 13. Requirements for 30 to 35 percent stone mixes included a minimum Marshall stability to flow ratio at 140°F (60°C) of 1.5 kN/mm, maximum Marshall stability of 2,500 lbs. (17,200 kN), 2 to 10 percent voids in the mix, a calculated film thickness of 6.0 μ m, and a minimum filler to bitumen ratio of 1.0.

The relationship between rutting and the stability/flow ratio obtained by Marais [24] for gap-graded mixes is shown in Fig. 11. Marais found that a gap-graded test pavement section underwent less densification than a continuously graded mix. Both mixes approached equilibrium densities close to the maximum density obtained in the 75 blow Marshall test.

Freeme [23] found that the most important indices influencing the sand used in 30 percent stone, gap-graded mixes are as follows:

1. the sand equivalent which insures cleanliness,
2. fineness modulus which insures reasonable grading, and
3. the shape factor which insures reasonable shape.

Table 14 summarizes the criteria proposed for accepting and rejecting sands used in gap-graded mixes. Category B sands should not be rejected if either the fineness modulus or shape factor criteria is not satisfied. One of the correlations upon which the design criteria given in Table 14 was based is presented in Fig. 12. For fineness moduli greater than approximately 1.5, the required binder content of the gap-graded mixes was generally less than 6.5 percent (Fig. 13).

Table 12. Composition of Gap-Graded Course Mixtures by Recipe Method - Crushed Stone Aggregate [22]

| Schedule No. | | | |
|--------------|--|----------------|---|
| | Coarse aggregate retained on 2.36 mm BS test sieve | Soluble binder | Aggregate passing ⁽¹⁾ 75 μ m BS test sieve |
| 1A | 0 | 10.3 | 13.0 |
| | 15 | 9.1 | 11.0 |
| | 30 | 7.9 | 8.9 |
| | 40 | 7.1 | 7.5 |
| | 55 | 5.9 | 5.4 |
| 1B | 0 | 10.8 | 14.0 |
| | 15 | 9.6 | 12.0 |
| | 30 | 8.4 | 9.9 |
| | 40 | 7.6 | 8.5 |
| | 55 | 6.4 | 6.4 |
| 1C | 0 | 11.3 | 15.0 |
| | 15 | 10.1 | 13.0 |
| | 30 | 8.9 | 10.9 |
| | 40 | 8.1 | 9.5 |
| | 55 | 6.8 | 7.4 |

1. Bulk density in toluene must be between 0.5 g/ml and 0.9 g/ml.

Table 13. Design Criteria Proposed by Marais for Gap-Graded Surfacing Mixtures with Stone Contents Between 30 and 50 Percent [22]

| Traffic Category | All | Limits | |
|--|--|--------------------|--------|
| No. of compaction blows each end of specimen | 75 | | |
| Test Property | Surface Property | Max | Min |
| Marshall stability/flow at 60°C (kn/mm) | Deformation or distortion | - | 1.5 |
| Indirect tensile strength at 40°C (kn/m ²) or Marshall stability | Toughness, fatigue resistance, and fracture strength | 680 11 | - - |
| Immersion index | Stripping by Water | - | 0.75 |
| Air permeability (cm ²) | Imperviousness durability | 1x10 ⁻⁸ | - |
| Voids in mix (%) | Balance in design | 10 | 2.0 |
| Film thickness of bitumen (µm) | Durability | - | 6.0 |
| Filler/bitumen ratio | Balance in design | | 1.0 |
| Bulk density in benzene of filler (g/ml) | Distortion toughness | 0.95 | 0.50 |

Table 14. Criteria Proposed by Freeme for Preliminary Acceptance of Gap-Graded Surface Mixes [23]

| Category | Description | Criteria | Mixes Accepted, (%) |
|----------|---|---|---------------------|
| A | Sands with a Sand Equivalent greater than 30 | SE > 30 | 78 |
| B | Sands with a Sand Equivalent between 15 and 30 but having either a Fineness Modulus greater than 1.0 and/or a Shape Factor greater than 0.7 | 15 < SE < 30 and FM > 1.0 or SF > 0.7 | 42 |
| C | Sands with a Sand Equivalent less than 15 | SE < 15 | 0 |

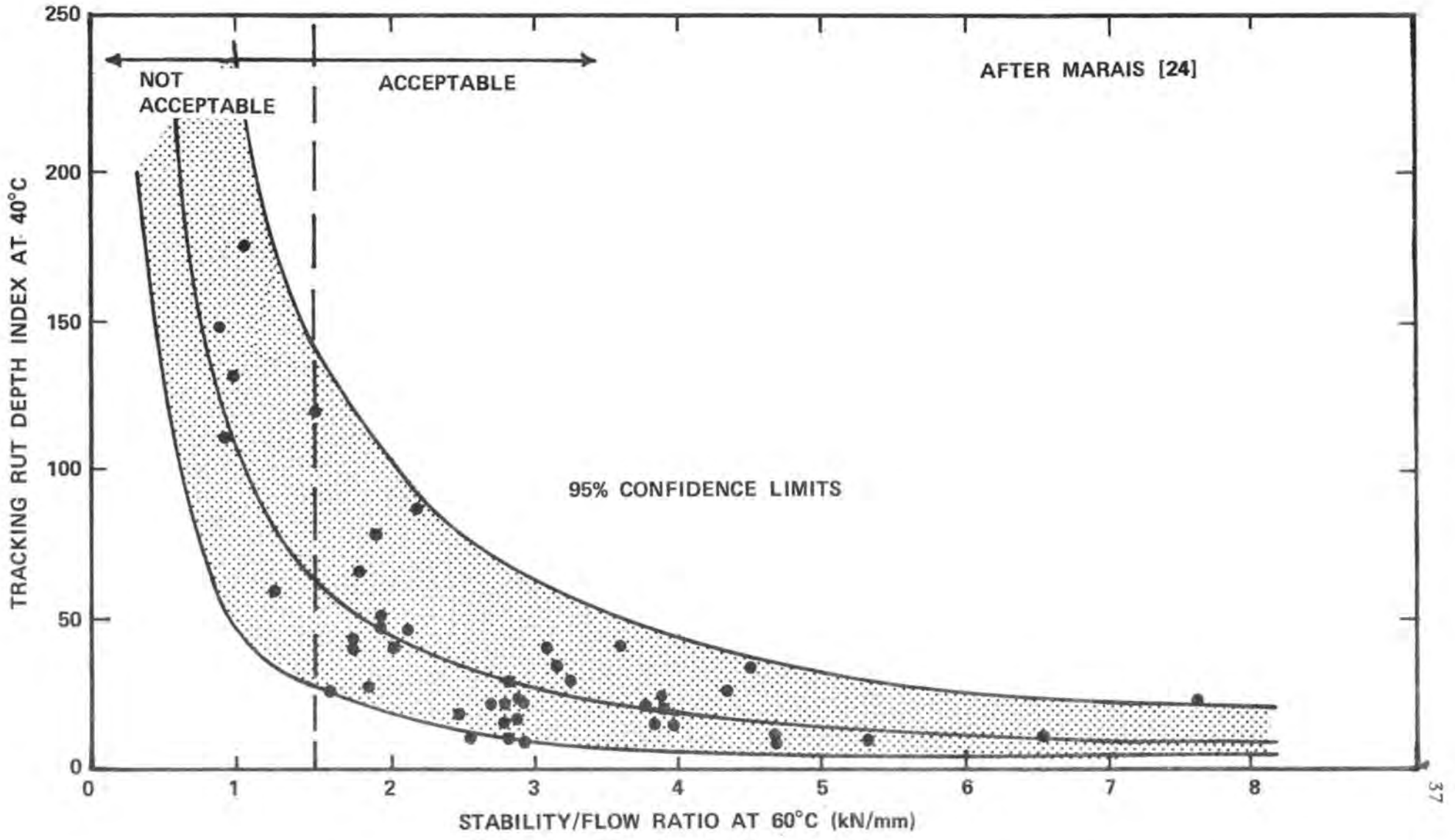


FIGURE 11. EFFECT OF STABILITY/FLOW RATIO ON TRACKING RUT DEPTH INDEX AT 40°C ON GAP-GRADED MIXES

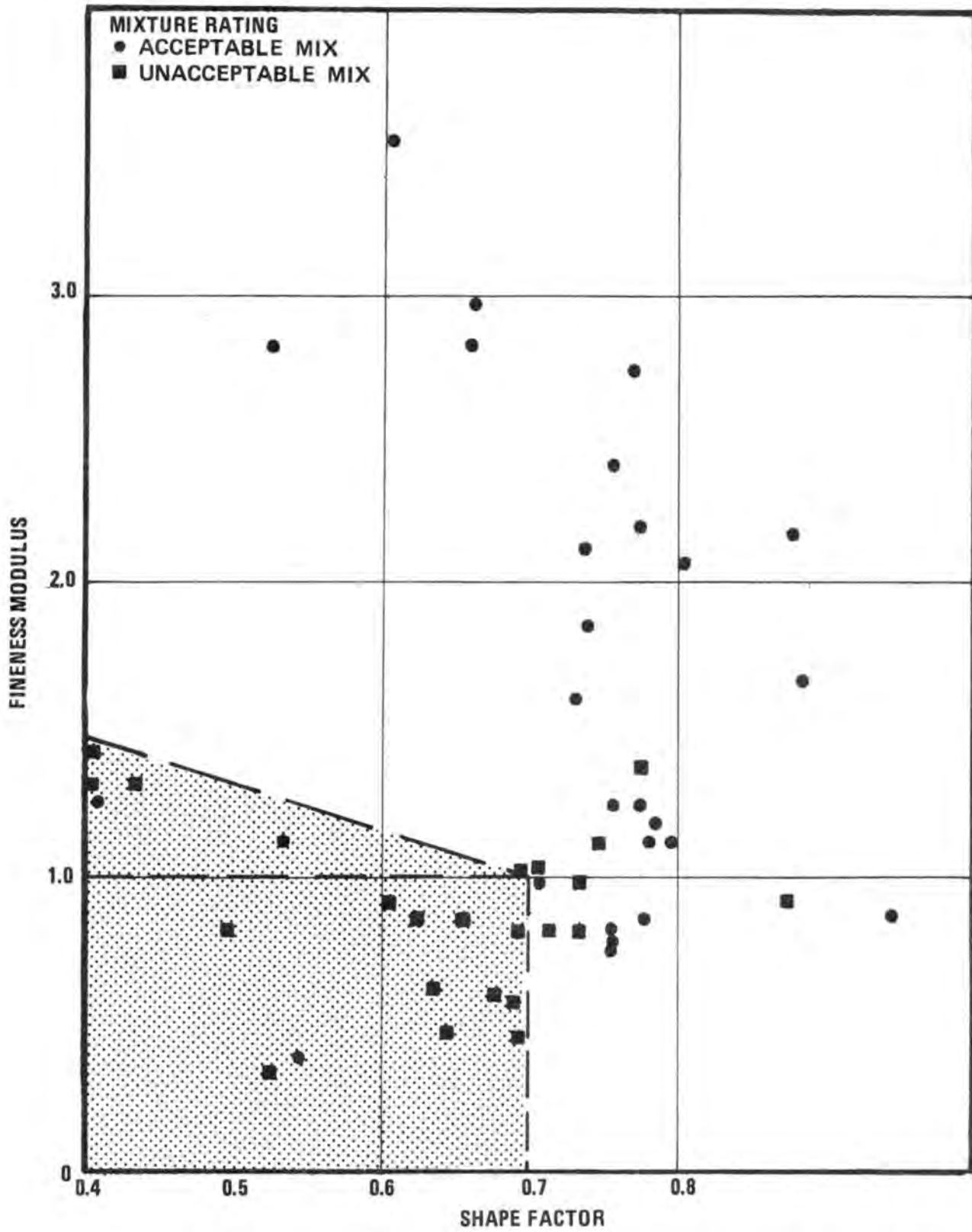


FIGURE 12. INFLUENCE OF SHAPE FACTOR AND FINENESS MODULUS ON ACCEPTABILITY OF GAP-GRADED MIXES

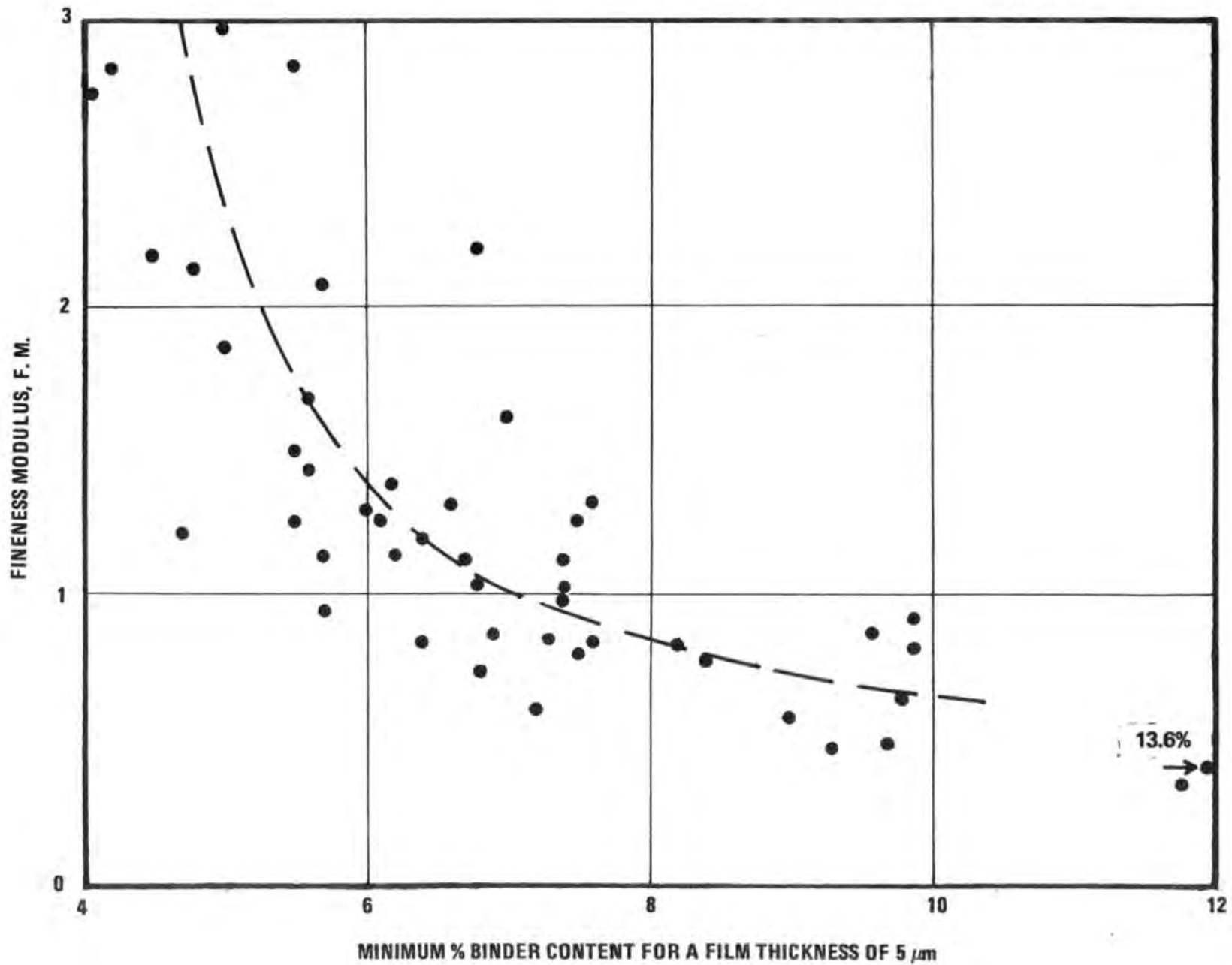


FIGURE 13. MINIMUM PERCENT BINDER TO OBTAIN A THEORETICAL FILM THICKNESS OF 5MM IN THE MIX (AFTER MARAIS)

Sand-Cement Stabilized Bases

Introduction

Portland cement has been used as a stabilizing agent since 1935 when the South Carolina Highway Department combined it with soil in five experimental test sections [33]. Soil cement is made by blending generally 6 to 10 percent portland cement with soil, adding the proper quantity of water and compacting the mixture to a high density [34]. Upon hydration, the soil cement mixture becomes strong and durable provided a suitable soil is stabilized with a sufficient quantity of cement. Sand is particularly suitable for stabilizing with cement since it is cohesionless, mixes readily, and gives high strengths upon hydration provided a sufficient amount of fines is present. Required cement contents for sand bases typically vary from 5 to 8 percent, with the quantity of cement required being generally less than for silts and clays but greater than for gravels and crushed stone.

A cement stabilized base must have a cement content sufficiently great to be both durable and have strengths capable of withstanding the applied wheel loads. Strength usually is expressed in terms of the 7-day unconfined compressive strength of cylindrical specimens of the mixture. Durability usually is evaluated by subjecting specimens to standard freeze-thaw and/or wet-dry cycles [35]. In areas with severe climates having considerable freeze-thaw cycles, the effective in situ long-term durability of the material is probably the most important criteria. In Alberta, Canada, for example, a simplified mix design technique [36] developed for soil cement blends uses a freeze-thaw test as the basis for selecting cement content.

Rutting in pavement sections having cement treated bases has been found to pose no problem [17]. However, the development of transverse reflection cracking is frequently an important problem resulting from pavements constructed with cement treated bases [17]. Surface reflection cracking results from shrinkage cracks that originally develop in a cement stabilized base upon hydration of the cement. These cracks gradually propagate to the surface through the asphalt concrete surfacing. Zube *et al.* [37] found that to minimize longitudinal and transverse reflection cracking, a minimum thickness was required of 3 1/2 in. (90 mm) of asphalt concrete surfacing. In Virginia, McGhee [38] found that transverse shrinkage cracks can reflect through a 3 in. (76 mm) bituminous concrete surface course in as little as 18 months and through 7 in. (180 mm) in less than 5 years. Research by Barksdale and Vergnoble [39] on a micaceous silty sand indicated that use of an expansive cement reduces shrinkage cracks in some cement stabilized silty sand bases, thus minimizing the problem of reflection cracking. Other methods attempting to reduce shrinkage cracks in cement stabilized bases include the addition of sodium chloride, calcium chloride, and sugar [40]. Also, use of lime and flyash has been found in some instances to reduce shrinkage cracks. Use of such additives as expansive cements should be studied as a means of controlling shrinkage cracks in cement stabilized sand bases.

Burns and Ahlvin [47] found at the Waterways Experiment Station that for heavy aircraft loadings (30 kip wheel loads), sections having an uncrushed gravel sand subbase stabilized with 6 percent cement were still in satisfactory condition after 10,460 coverages. In contrast, a similar section having a lean clay subbase stabilized with 10 percent withstood only 1,200 coverages. These results indicate that under some conditions, a cement stabilized granular material will perform better than a cement stabilized cohesive soil stabilized at an equal or higher level.

At the Brampton Road Test [48], the June rebound deflections were after three years close to twice the value measured the first year for a section having a 6 in. (150 mm) granular subbase treated with 10 percent cement by weight. Increase in rebound deflections also occurred elsewhere [43] and was caused partially by gradual cracking and deterioration with time of the cement treated base. Because of a high modulus of elasticity and brittleness, high tensile stresses are developed in a cement-stabilized base. As a result, small cracks may develop in the tensile zone which gradually become larger with the application of wheel loadings and result in an increase in deflection. In Japan, studies have shown that the increase in deflection is roughly proportional to the logarithm of the number of loads applied to the pavement. Fossberg, Mitchell and Monismith [44] found that if the deflection of a cracked base is 20 percent larger than an intact base, the stress transmitted to the soil is nearly 50 percent greater.

AASHO Road Test

Comparative performance of cement treated granular bases was studied at the AASHO Road Test [43]. The cement treated, gravelly sand base had 90 percent passing the 1/2 in. (13 mm) sieve, 71 percent passing the No. 4 sieve, and 7 percent passing the No. 200 sieve and hence was classified as a gravelly sand. The granular base was treated with 4 percent cement by weight resulting in an average seven-day unconfined compressive strength of 840 psi (5,800 kN/m²). Comparison in performance between the sand-cement, bituminous stabilized and crushed stone base sections is shown in Fig. 14 for a PSI of 2.5. For approximately one million 22.4 kip (100 kN) single axle loadings and environmental and loading conditions, and materials used at the AASHO Road Test, one inch (25.4 mm) of asphalt concrete base was found to be equivalent to 1.2 in. (30 mm) of cement treated base and 1.80 in. (46 mm) of crushed stone. Rutting in the cement treated sections was found to be significantly less than in the other two sections. Results indicate that the cement treated base section, which had an average unconfined compressive strength of 840 psi (5,800 kN/m²), performed quite favorable when compared with the asphalt concrete base sections if the aesthetics of transverse shrinkage cracks are not considered.

Rhode Island Test Pavement

Wang and Nacci [45] described the field performance of seven test sections constructed in Rhode Island using cement stabilized bases (Fig. 15). All sections had a 3 in. (75 mm) asphalt concrete surfacing

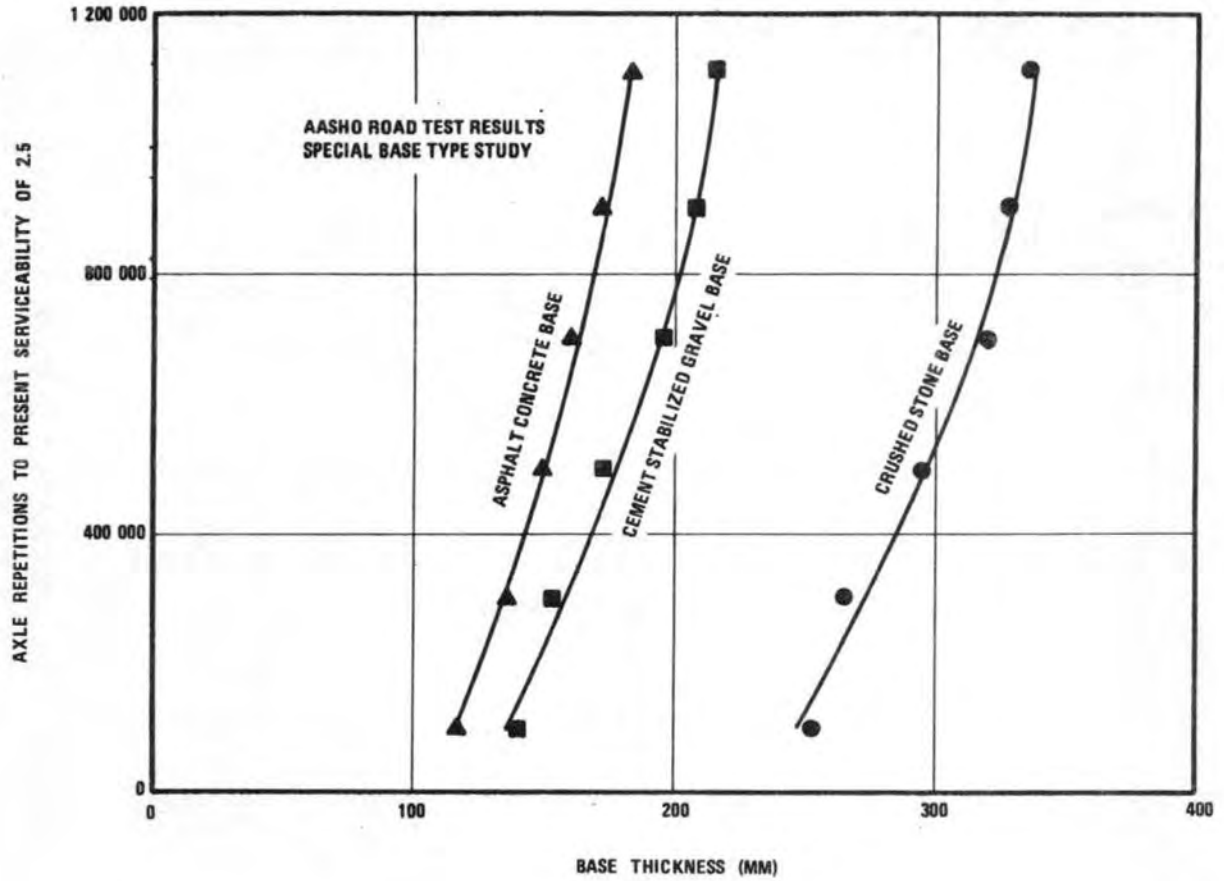


FIGURE 14. EFFECT OF BASE THICKNESS ON PAVEMENT LIFE FOR SELECTED BASE TYPES FOR 3IN. A. C. SURFACE AND 4IN. SUBBASE—AASHO ROAD TEST

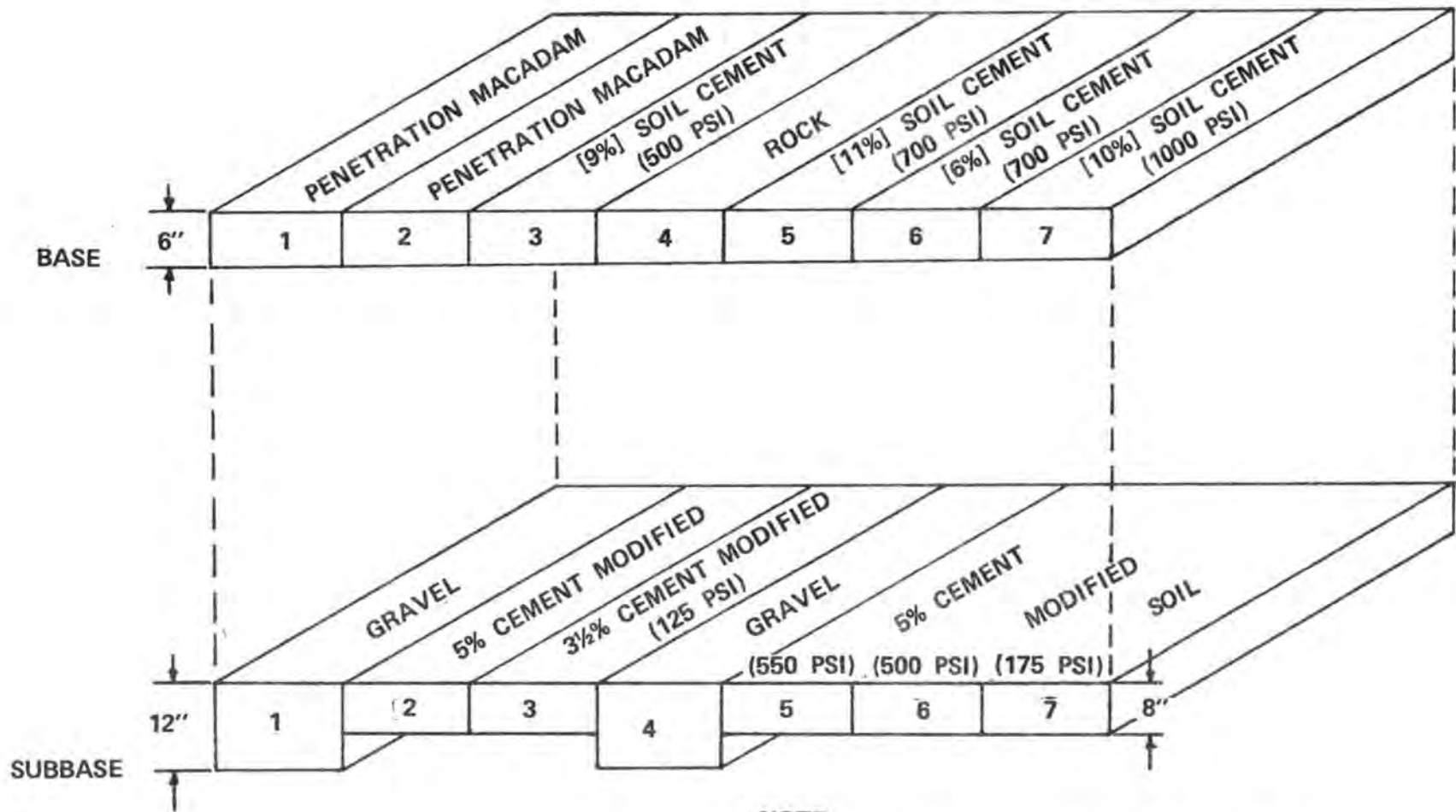


FIGURE 15. RHODE ISLAND TEST SECTION BASES AND SUBBASES

with a base thickness of 6 in. (150 mm). Five test sections were constructed with a non-plastic silty sand base [AASHTO A-4(3) Soil] stabilized with 6 to 11 percent Type I Portland cement; two of these sections also had 1 percent sodium sulfate. Two control sections were constructed with an asphalt macadam penetration rock base. Unconfined compressive strengths of the cement stabilized bases varied from approximately 500 to 1,000 psi (3,500 to 7,000 kN/m²) at 28 days, while the subbase strength varied from 125 psi to 550 psi (850 kN/m²) to 3,500 kN/m²).

After the application of approximately 60,000 equivalent 18 kip single axle loadings during the 4-year investigation, relatively extensive surface reflection cracking developed in the cement stabilized sections (Fig. 16). The measured surface roughness of the cement stabilized base sections, however, was only about 10 percent greater than for the penetration asphalt macadam control sections (Table 15). Therefore, the cracking which occurred in these sections did not significantly affect the overall riding qualities as reflected by the roughness index. An insignificant difference of less than 10 percent was noted in the various cement stabilized sections in total surface reflective cracking. Section 6 which was stabilized with 6 percent cement had the most transverse cracks, while section 3 which was stabilized with 9 percent cement had the most longitudinal cracking. As found by coring, cracking was found to extent through the entire base course to about the center of the subbase. Surface cracking was not observed in the penetration macadam control sections.

Controlling Reflective Cracking

Nordling [41] reviewed various methods for reducing reflection cracking in pavements constructed with cement treated bases. Nordling's pertinent recommendations include:

1. Using a granular base material having a minimum clay content,
2. Compacting the base material at or slightly below optimal levels to a high density,
3. Using the highest penetration (lowest viscosity asphalt cement in the surfacing that will meet stability requirements, and
4. Waiting until initial shrinkage cracking has occurred in the base before applying the surfacing.

Nordling found that double and triple surface treatments compared with asphalt concrete surfacings resulted in the development of fewer reflection shrinkage cracks of light to moderately trafficked pavements. A triple surface treatment gave good performance on 400 miles (670 km) of secondary roads in one Louisiana parish. For the first application, the triple surface treatment construction consisted of applying 0.40 gal. of asphalt and 0.0200 cu. yd. of 1/2 in. (13 mm) maximum size aggregate to each square yard of pavement; 0.30 gal. asphalt and 0.0111 cu. yd. of 3/4 in. (19 mm) maximum size aggregate for the second application; and 0.20 gal. of asphalt and 0.0075 cu. yd. of 1/2 in. (13 mm) aggregate for the third application. In New Brunswick, two surface treatment applications were placed the first year followed by a third application the second year. To minimize reflective surface cracking,

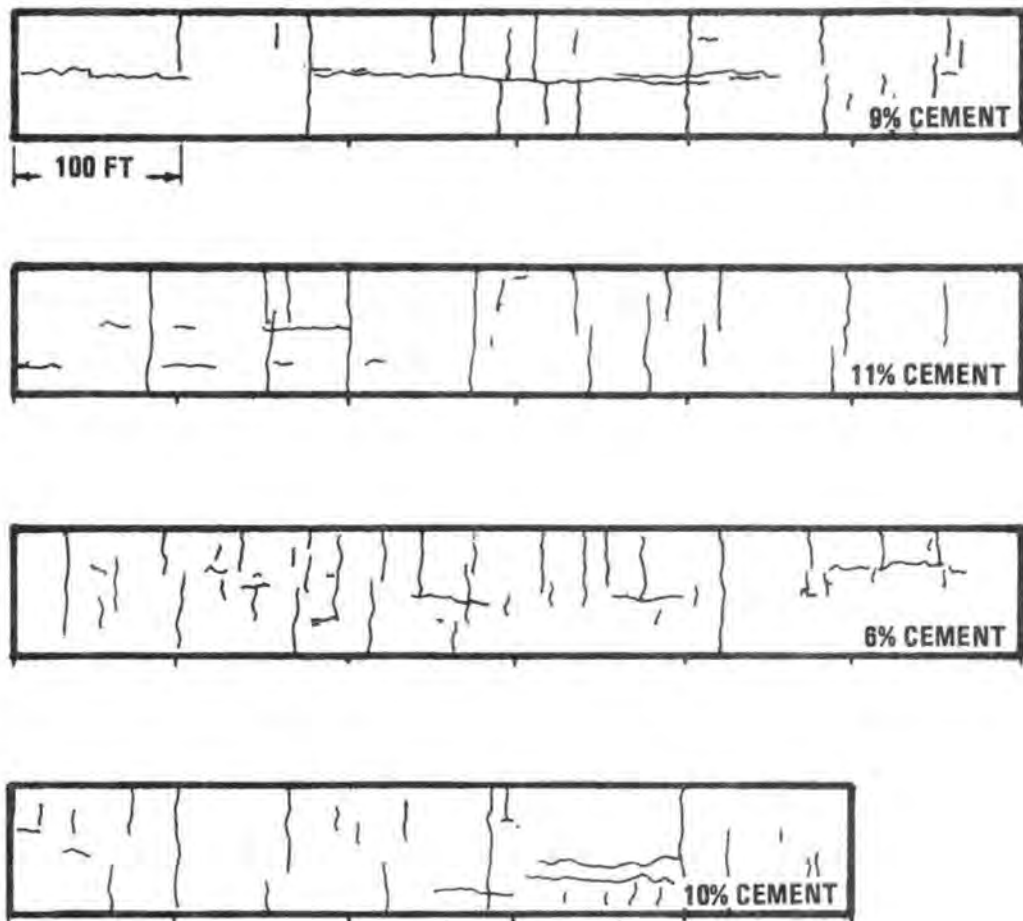


FIGURE 16. REFLECTION CRACK PATTERNS DEVELOPED IN SOIL-CEMENT BASES AT RHODE ISLAND TEST ROAD (AFTER WANG ET AL)

Table 15. Summary of Pavement Performance For Rhode Island Test Sections

| Section | Average Roughness (in./mile) | Total Cracking ⁽¹⁾ (ft.) |
|---------|------------------------------|-------------------------------------|
| 1 | - | - |
| 2 | 110 | - |
| 3 | 120 | 1090 |
| 4 | 111 | - |
| 5 | 123 | 590 |
| 6 | 120 | 900 |
| 7 | 110 | 725 |

1. Transverse and longitudinal crack length.

North Carolina developed a special treatment called a "prime, mat and seal" surface, and the Nova Scotia Highway Department began use of a special armor coat for soil-cement pavements.

In general, a minimum of 3 in. (75 mm) of asphalt concrete surfacing has been used in California and Washington to minimize reflection cracking in pavements having moderate to high volumes of traffic [41]. Oregon has used a 2 in. (50 mm) asphalt concrete surfacing over granular cement stabilized bases. The asphalt concrete surface course, however, with an asphalt content of 6 1/2 percent, made the surfacing relatively flexible and reportedly healed small surface reflection cracks [41]. To reduce reflective cracking in subdivisions in DeKalb County, Georgia, the soil-cement bases were first overlaid by a 1 in. (25 mm) binder course. The 1 in. (25 mm) surfacing was not placed until one week after the binder.

Cement Stabilized Stone Bases

Zube and Gates [46] described a comprehensive study involving 175 pavements constructed with cement stabilized granular stone bases which were subjected to 5 million or more equivalent 5 kip (22 kN) wheel loads. Although the study was not conducted on cement stabilized sand bases, the cement stabilized gravel and crushed stone bases used were similar enough that results should be generally applicable to pavements having sand-cement bases. Sixty-four percent of the 175 pavements studies gave excellent performance, 17 percent good performance, and 8 percent fair performance with only 11 percent requiring extensive early maintenance.

One very important cause of failure was found by Zube and Gates to be insufficient cement content. To perform satisfactorily, the minimum required in-place compressive strength of a cement stabilized base was found to be 500 psi (3,500 kN/m²). Class A cement stabilized bases constructed in California now require minimum 7-day compressive strengths of 750 psi (5,000 kN/m²). No significant increase in longitudinal or transverse cracking was found when compressive strength of the base was increased. Increased compressive strength did, however, significantly reduce block cracking in the pavement. Relatively rapid deterioration of some pavements was found to be due to nonuniform blending of cement during mixing and placing which resulted in localized areas having insufficient strength. Plant mixed cement stabilized bases showed considerably better uniformity than bases mixed in-place; however, studies on three projects using plant-mixed bases indicated a deficiency in cement content by more than the specified 0.4 percent in 3 to 8 percent of the cement stabilized base.

In 4 of the 35 projects studied by Zube and Gates, placing extremely thin lifts of cement stabilized bases to bring the layer up to grade, or reworking the surface after it had been compacted was found to be the probable cause of disintegration of the upper surface of cement stabilized bases. Bonding one layer of cement stabilized base to another was also found to be a problem; however, bonding asphalt concrete to a cement stabilized base presented no problem.

Additional conclusions reached by Zube et al. for conditions existing in California follow:

1. Extending the cement stabilized base at least one foot beyond the pavement edge into the shoulder can reduce block cracking, but does not affect longitudinal or transverse cracking.
2. Plant-mixed cement stabilized bases have less cracking than mixed-in-place projects.
3. Cement stabilized granular bases perform better when compacted in 4 in. (100 mm) thick layers rather than 8 in. (200 mm) layers.
4. The time of year when cement stabilized granular bases are placed has no significant effect on cracking.
5. Block cracking can be reduced by using Type 2 rather than Type 1 Portland cement.
6. Cement stabilized granular bases should be designed to have strengths 25 to 30 percent greater than desired at the end of construction to insure satisfactory in-place strength. This recommendation is justified since the cement stabilized bases with the highest compressive strength have the longest maintenance-free service life.
7. To prevent cracking of both the cement stabilized base and the surfacing, cement treated granular bases less than 6 in. (150 mm) thick should not be used.

Maine Test Pavements

In Maine, McNaughton and Rand [49] compared the performance of sections having soil cement, asphalt concrete and crushed gravel bases. The report did not describe the soil type that was stabilized with cement. All sections had a 3 in. (75 mm) thick asphalt concrete surfacing. Although the soil cement sections had a specified base thickness of 6 in. (150 mm), two of the three sections had measured thicknesses of only 4 and 5 in. (100 and 125 mm). Soil-cement base sections had an average 7-day compressive strength of 522 psi (3,600 kN/m²). Asphalt concrete sections had a 4 in. (100 mm) thick base and only about 3.85 percent asphalt cement as determined from extraction tests. The crushed gravel base varied in thickness from 6 to 9 in. (150 to 230 mm).

After 160,000 equivalent 18 kip single axle loadings, the PSI Ratings were 3.2 for the soil cement sections, 4.2 percent for the asphalt concrete and 4.3 for the crushed gravel base sections; visual ratings indicated a similar ranking. The PSI ratings remained almost constant during the study for asphalt concrete and crushed gravel sections. The PSI ratings for the soil cement base sections gradually decreased with time. In fact, by the end of four years, the surface deflections of sections with soil-cement bases had increased to almost

the same value as the crushed gravel sections. These results indicate insufficient levels of cement stabilization of the base achieved in the field to withstand severe environmental conditions experienced in Maine. Also, McNaughton and Rand found soil cement base sections to exhibit more expansion and contraction cracks than did other test sections. They did not have the ability to regain their PSI ratings after the spring thaw. Wire mesh reinforced asphalt concrete overlay 1.75 in. (45 mm) thick over some of the soil-cement sections was found to increase the observed amount of reflection cracking.

Inverted Construction

Successful use of cement stabilized subbase layers which were overlaid by unstabilized granular bases were reported by Johnson [42] in New Mexico in addition to a Virginia study reported by McGhee [38]. These so-called "inverted" or "upside down" sections were used successfully in New Mexico on urban projects having weak subgrades; a test section was also built in New Mexico which included this type construction. In New Mexico, a 6 in. (150 mm) thick granular subbase stabilized with 3 to 5 percent cement was used. This layer was overlaid by 4 to 6 in. (100 to 150 mm) of unstabilized granular base which in turn typically was overlaid by 3 1/2 to 4 in. (90 to 100 mm) of asphalt concrete. Frequently a 1/2 to 5/8 in. (13 to 16 mm) plant mix seal coat was also placed over the surfacing either at the time of construction or several years later [39]. British Columbia used a 3 in. (75 mm) unstabilized granular layer in "inverted" sections that had a maximum of 9 percent fines and a plasticity index no greater than five.

Advantages of inverted construction include:

1. Reduction in reflection cracking since the cement-treated layer is placed deeper in the section,
2. Deeper overall thickness of the structural section,
3. Better compaction of unstabilized materials placed over the stabilized layer which acts as a working platform where soft subgrades are encountered, and
4. Protection of the subgrade from moisture.

The inverted design makes optimum use of the good tensile strength properties of the cement stabilized subbase to bridge over weak subgrades through "beam" action. Optimal use is made of the excellent compressive characteristics of unstabilized crushed stone or sand by placement in the upper part of the pavement structure where stresses are compressive. Although this type of construction might tend to trap water in the upper layers, a degree of protection from moisture would be given to the subgrade. Based on the studies reported by McGhee [36] and Johnson [40], inverted construction appears to offer promise, particularly for construction of pavement over weak subgrades. This type of construction certainly deserves further study.

CHAPTER III

PREDICTING RUTTING IN PAVEMENTS WITH SAND-ASPHALT BASES

Introduction

Based on the results of pavement field inspections in Florida, Georgia, Maryland and South Carolina, rut depths from 0.25 to 0.5 in. (8 to 13 mm) are usually developed in pavements having sand-asphalt bases. Since rut depths greater than about 0.4 in. (10 mm) are sometimes considered excessive, a rational method is needed to predict rut depth in this type pavement. In this chapter a practical method is developed to predict rut depth that uses the plastic properties of the sand-asphalt and asphalt concrete mixes measured using the repeated load triaxial test and standard test conditions. The rut depth of the asphalt layers are then calculated considering the geometry of the section, stress level, and number of equivalent 18 kip axle load applications in addition to the plastic properties of asphalt layers,

Average Stress State for Rutting Tests - The Z Function Approach

Under the centerline of the wheel loading, according to layered theory, the bituminous layer is subjected to a compressive stress state in the top portion of the layer and tensile stress in the bottom. Application of a tensile stress state in the laboratory to reproduce the theoretical stress condition in the bottom of the layer is difficult and not suitable for use in a practical design method for evaluating the rutting characteristics of bituminous pavements. When an element of material in the tensile zone starts to deform laterally in the direction of the tensile stress, lateral passive resistance is encountered due to the presence of the surrounding sand-asphalt or asphalt cement. Therefore, the lateral tensile stress developed in the field may be less than predicted by layered theory.

Near the center of the asphalt concrete layer at the neutral axis, the lateral stress is zero, and an axial compressive stress state exists similar to that in the unconfined compression test. The existence of a simple unconfined compressive stress state near the center of the layer suggests that such a stress condition may be reasonably close for design purposes to the average of the tensile and compressive stress states within the layer.

Using the average stress condition within the bituminous layer for testing purposes to represent the stress conditions within the entire layer is a practical alternative to using considerably more complicated

Substituting equation (4) into equation (3) and rearranging gives

$$\sigma_{\text{avg}} = Z \cdot \sigma_0 \quad \dots \dots \dots (5)$$

Equation (5) gives the vertical stress that would have to be applied to an unconfined specimen to give the same elastic strain as that calculated to occur in the layer. The modulus of elasticity of the specimen and the layer are taken to be the same. Even though the theoretical prediction of average stress in the asphalt concrete layer (for no lateral confinement) is based on the elastic deformation of the layer, this admittedly idealized approach does consider the combined effect of tension and compression in a three-dimensional stress field. Hence, the Z-function method should give a reasonable first approximation of the stress state for use in testing to evaluate plastic deformation.

Mean Pavement Temperature for Rutting

A simplified method is used for estimating the mean pavement temperature at which rutting occurs for a given climatological region and representative traffic conditions [6,54]. Laboratory tests then only need conducting at the mean pavement temperature to evaluate the average rutting characteristics of a mix. Hofstra and Klomp [57] and Lister and Addis [58] established from model pavement studies the relationship between rutting and pavement temperature for selected asphalt mixes (Fig. 17). This figure shows that rutting occurring in the bituminous layer is sensitive to temperature and that essentially no rutting occurs below a pavement temperature of 75°F (24°C).

In this study, mean pavement temperature for rutting was developed for the climatological and traffic conditions representative of Marianna and Lake Wales, Florida where test pavements were located having sand-asphalt bases. Mean temperature for rutting was calculated for a section with 3 in. (75 mm) asphalt concrete surfacing and a 6 in. (150 mm) sand-asphalt base. A previous study [6,54] indicated that long-term monthly average temperatures could be used to determine the mean pavement temperature for rutting. The long-term climatological weather data for Marianna that was used to predict temperature profiles through the pavement is given in a report by Schroeder [64]. Mean daily air temperature from May through August was 79°F (26°C) with the long-term daily difference in temperature being 22°F (12°C). For Lake Wales, mean daily temperature from May through August was about 80°F (27°C) with the long-term daily difference in temperature being 19°F (10°C). Temperatures used in the analysis for Marianna and Lake Wales were obtained from weather stations located at Chipley and Winter Haven, Florida, respectively. All other climatological information was obtained for Marianna from Tallahassee and for Lake Wales from Tampa and included solar radiation, cloud cover, and wind speed. The climatological data used were long-term averages from weather data collected between 1931 and 1976.

The method proposed by Barber [59] was used to predict the temperature distribution within the asphalt concrete layers throughout the year. The Barber method assumes that the asphalt concrete is a

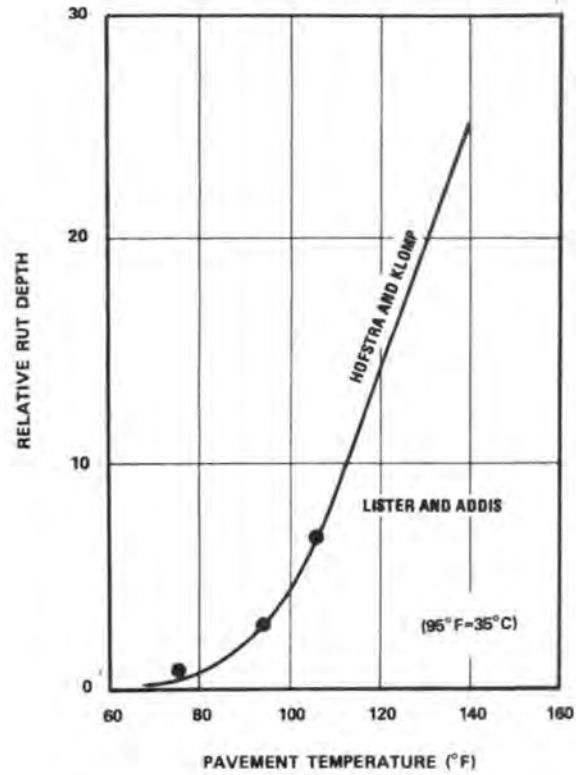


FIGURE 17 (a). EFFECT OF PAVEMENT TEMPERATURE ON RUT DEPTH

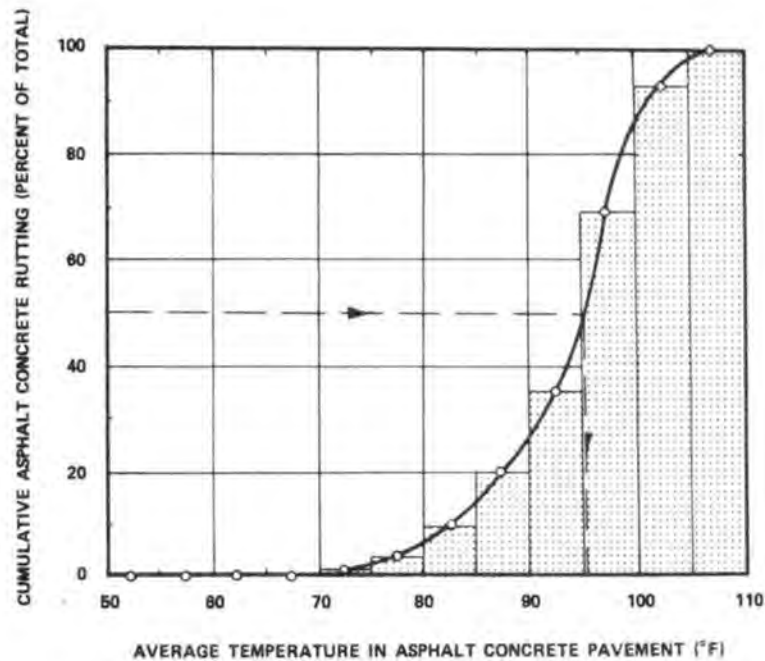


FIGURE 17 (b). PAVEMENT TEMPERATURE AS A FUNCTION OF CUMULATIVE PERCENT OF ASPHALT CONCRETE RUTTING IN GEORGIA—TYPICAL CONDITIONS

semi-infinite mass in contact with the air with temperature within the mass varying as a sinusoid with time. Consensual agreement has been reported between predicted and measured pavement temperatures using this method [60]. The thermal conductivity of the asphalt concrete was taken to be 0.70 BTU-ft/ft²-F-hr, the specific heat to be 0.22 BTU/lb-F, and the absorptivity of the asphalt concrete to be 0.90. In addition to the monthly mean air temperature, meteorological data used included diurnal range, solar insolation, cloud cover and wind speed. For the Marianna pavements, a weighted average unit weight of bituminous layers of 132 pcf [61] was used and for Lake Wales, 124.5 pcf. These densities were weighted averages of each layer measured in the field. Extraction tests indicated average measured air voids in each mix to be 8.4 percent.

Hourly temperature profiles were calculated using the above data and a computer program developed by Kasianchuk [6] for the average day of each month for July through December at depths from 0 to 8 in. (200 mm) in 2 in. (50 mm) increments. Only data for six months was used in the analysis because climatological information for the following months were similar: Dec./Jan., Nov./Feb., Oct./Mar., Sept./April, Aug./May, and July/June. Amount of temperature data to be analyzed was reduced further by dividing the day into six time intervals with average pavement temperature calculated within each time interval for each month.

In a similar analysis by Ngowtrakul [62], the relative amount of rutting during each of six time intervals was calculated using average monthly distributions of heavy truck traffic occurring throughout the day as obtained by mechanical and manual traffic counts. Although the hourly traffic distributions used were for middle Georgia, the overall error introduced in the analysis by using these distributions for the locations in Florida was not considered significant.

The analysis to determine the mean pavement temperature for rutting was performed using the relative values of rutting observed by Hofstra and Klomp [57] using a test track as previously described by Barksdale [6]. Our assumption was that the relative rut depth due to each wheel load passage in the field was equal to that observed in the test track for the same temperature. The relative rut depth for each time interval was then determined using the distribution of truck traffic and calculated average temperature in the asphalt concrete. By tabulating the relative rut depth occurring in each 5°F (3°C) temperature interval from 50°F to 110°F (10° to 44°C), the relationship shown in Fig. 18 was established between average pavement temperature and cumulative rut depth for the Marianna and Lake Wales pavements. From this figure, the mean pavement temperature at which rutting develops was approximated to be 92.5°F (33.6°C) for the Marianna pavements and 93.5°F (34.2°C) for Lake Wales for the conditions used in the analysis. Almost all pavement rutting occurred between an average pavement temperature of 75°F and 100°F (24° and 38°C) which corresponds to typical surface temperatures between about 62° and 87°F (17° and 30°C). This analysis, of course, assumed the relative relationship between temperature and rut depth for the mixes studied by Hofstra and Klomp to be similar to that for the sand-asphalt mixes studied in this investigation.

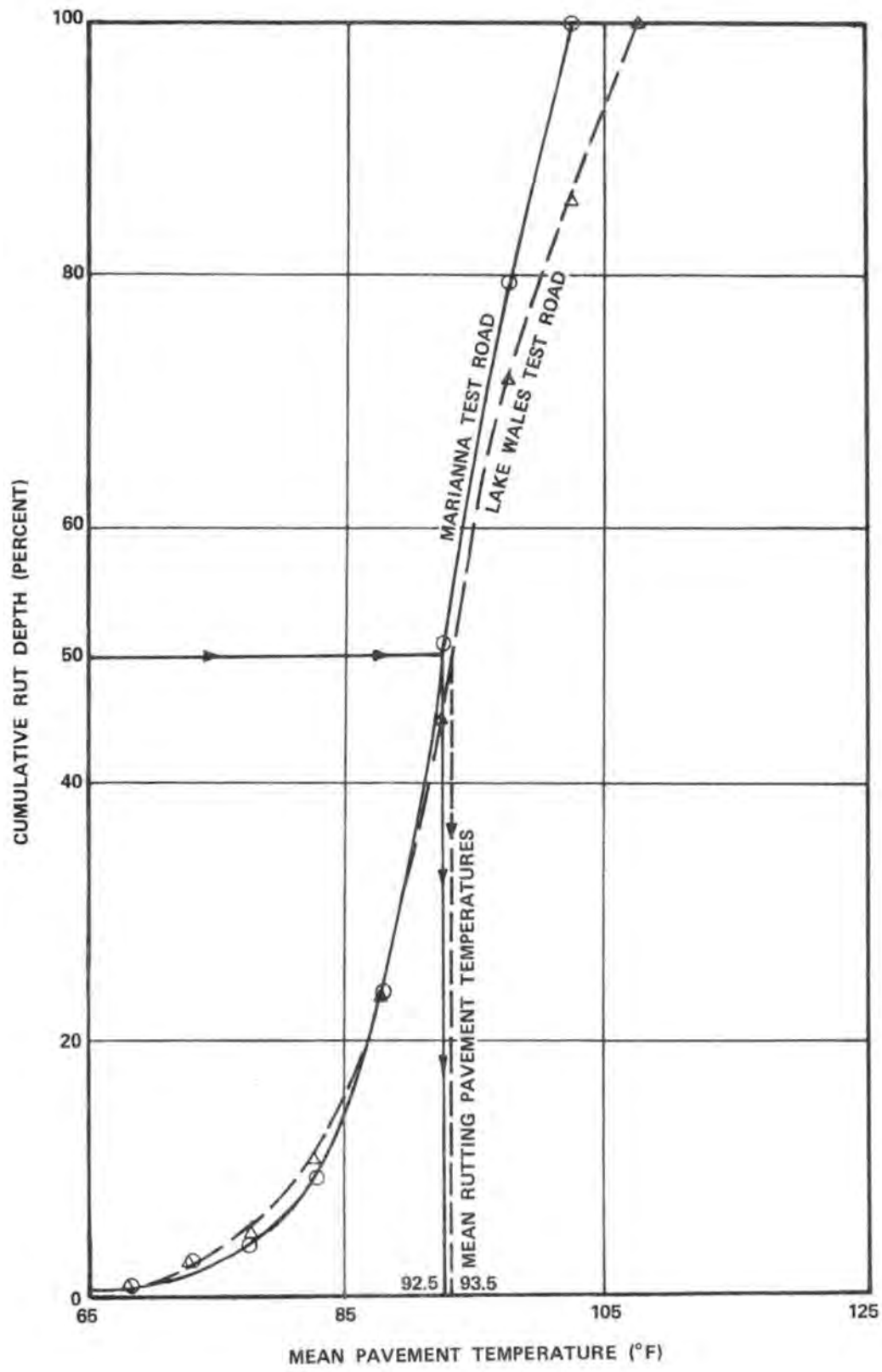


FIGURE 18. VARIATION OF RUT DEPTH WITH PAVEMENT TEMPERATURE AT MARIANNA AND LAKE WALES TEST ROAD SITES—MEAN PAVEMENT TEMPERATURES FOR RUTTING

A similar study [6,62] found that the mean pavement temperature for rutting in the central portion of Georgia was 95.8°F (35.4°C). A standard laboratory testing temperature of 95°F (35°C) was used in this earlier study. Since the mean temperatures for rutting found at both locations in Florida were only slightly less, 95°F (35°C) was also used as the standard testing temperature to evaluate rutting (plastic) properties of the sand-asphalt and sand-stone blends so that the properties of the various mixes studied could be directly compared. Since relatively little variation in the mean pavement temperature for rutting was found from central Georgia to middle and northeast Florida, a testing temperature of 95°F (35°C) was considered valid for most of the Coastal Plain areas of South Carolina, Georgia and Florida.

Theoretical Prediction of Rut Depth in Sand-Asphalt Pavements

Introduction

The rut depth occurring in an asphalt concrete mix composed of crushed stone can be estimated using the results of the repeated load triaxial test and the following simplified engineering expression recently developed by Barksdale [6]:

$$\delta = \left\{ 1.4 \frac{Z \cdot \sigma_o - 5}{65 - Z \cdot \sigma_o} \right\} \left\{ \frac{1.25 N}{300 + N} \right\} \lambda \cdot \epsilon_p \cdot H \quad (6)$$

where Z = the Z-function which has been previously defined

σ_o = tire contact pressure in psi

N = number of 18 kips (80 kN) equivalent axle loads expressed in thousands

ϵ_p = measured axial plastic strain in a specimen

H = layer thickness in inches

σ_3 = constant confining pressure used in the repeated load triaxial test

λ = correlation factor

The correlation factor λ varies from approximately 2.5 to 3.2 when the plastic strain ϵ_p is evaluated for 100,000 load repetitions from the repeated load triaxial test for the following conditions:

1. a constant confining pressure, σ_3 of 5 psi (34 kN/m²),
2. a deviator stress $\sigma_1 - \sigma_3$ of 25 psi (170 kN/m²), and
3. the average temperature at which rutting occurs in the pavement (approximately 95°F (35°C) in Georgia and Florida).

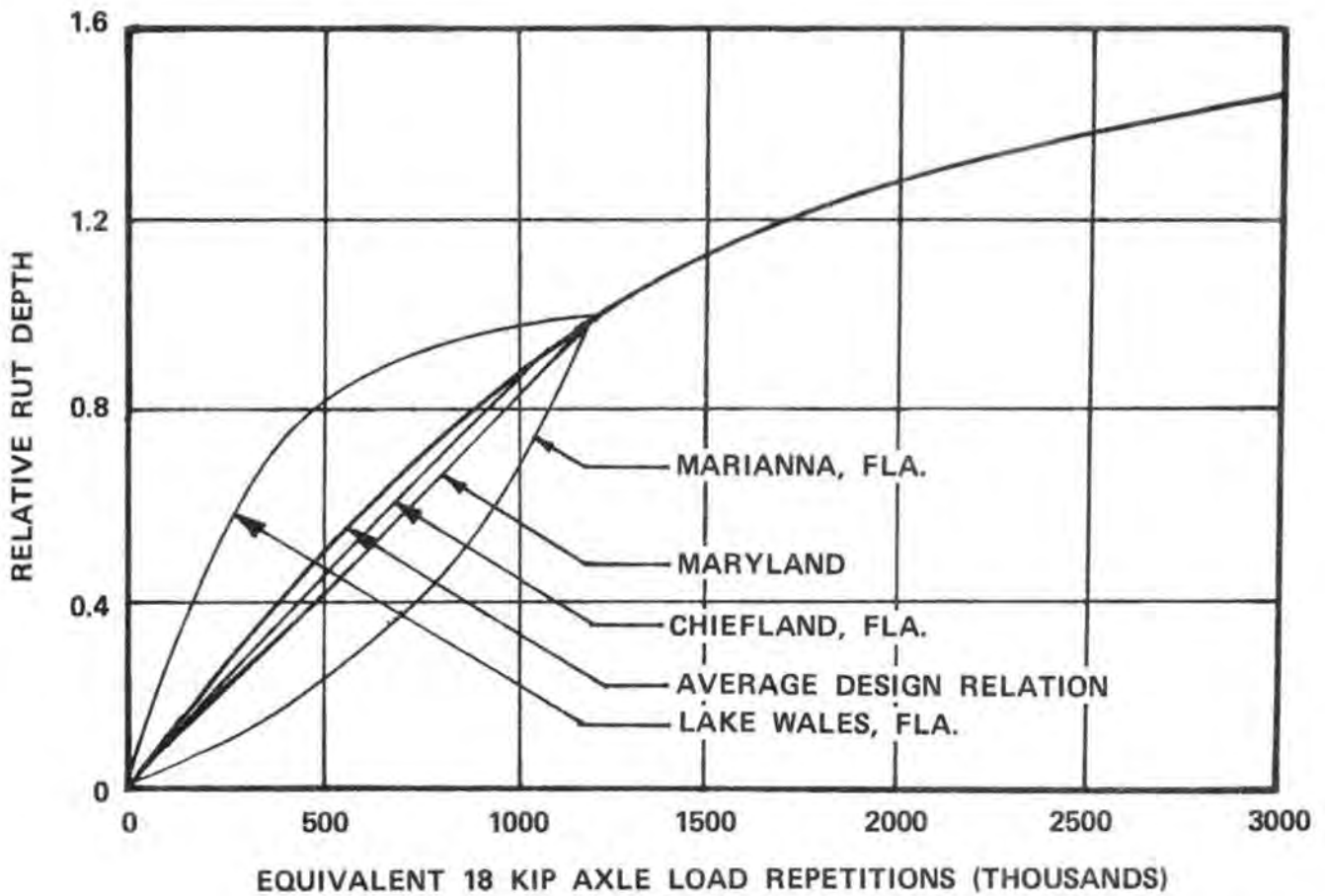


FIGURE 19. RELATIVE ACCUMULATION OF RUTTING IN DIFFERENT SAND-ASPHALT BASE PAVEMENTS

For axle loadings up to about 3×10^6 repetitions, for a best curve fit using trial and error values of δ_u and a , the set of δ_u and a values which gave a satisfactory curve fit was found to be 2.1 and 1320, respectively, giving:

$$\delta_r = \frac{2.1N}{N+1320} \quad \dots \quad (9)$$

Equation (9) corrects the calculated rut depth which is for 1,200,000 load repetitions for the actual number of applied repetitions for pavements with sand-asphalt bases. This expression is valid for up to approximately 3 million equivalent 18 kip axle loadings.

The correction term used to adjust the plastic strains measured in the repeated load test to consider the actual stress level in the pavement is developed using the Z-function concept previously introduced. The laboratory established relationship between axial stress and permanent (plastic) strain for a typical asphalt concrete and sand-asphalt base course mix have an asphalt content of approximately 4.8 and 5.5 percent, respectively, which is shown in Fig. 20. These relationships are very nearly hyperbolic and deviator stress as a function of percent strain can be approximated for the sand-asphalt mix by the following hyperbolic expression:

$$\sigma_1 = \sigma_3 = \frac{\epsilon_p}{1 + \epsilon_p} 50 \quad \dots \quad (10)$$

The total vertical stress σ_1 in a layer is, from the basic definition of the Z-function, equal to

$$\sigma_1 = Z \cdot \sigma_0 \quad \dots \quad (11)$$

and hence the deviator stress can be expressed as:

$$\sigma_1 - \sigma_3 = Z \cdot p - 5 \text{ psi} \quad \dots \quad (12)$$

for the standard 5 psi (35 kN/m^2) used in the triaxial test to evaluate the plastic properties of the mix. The correction term for stress level can then be expressed as

$$\text{Stress Level Correction} = \frac{\epsilon_p @ \text{field } \sigma_1 - \sigma_3}{\epsilon_p @ \sigma_1 - \sigma_3 = 25 \text{ psi}} \quad \dots \quad (13)$$

The denominator for equation (13) which is the plastic strain for $\sigma_1 - \sigma_3 = 25 \text{ psi}$ (170 kN/m^2) is obtained from Fig. 20 and equals 1.0 percent plastic strain. For any pavement geometry the applied vertical stress σ_1 can be calculated from (12). The corresponding deviator stress is then obtained by subtracting the standard 5 psi (35 kN/m^2) confining pressure used in testing from σ_1 . Now substitute this deviator

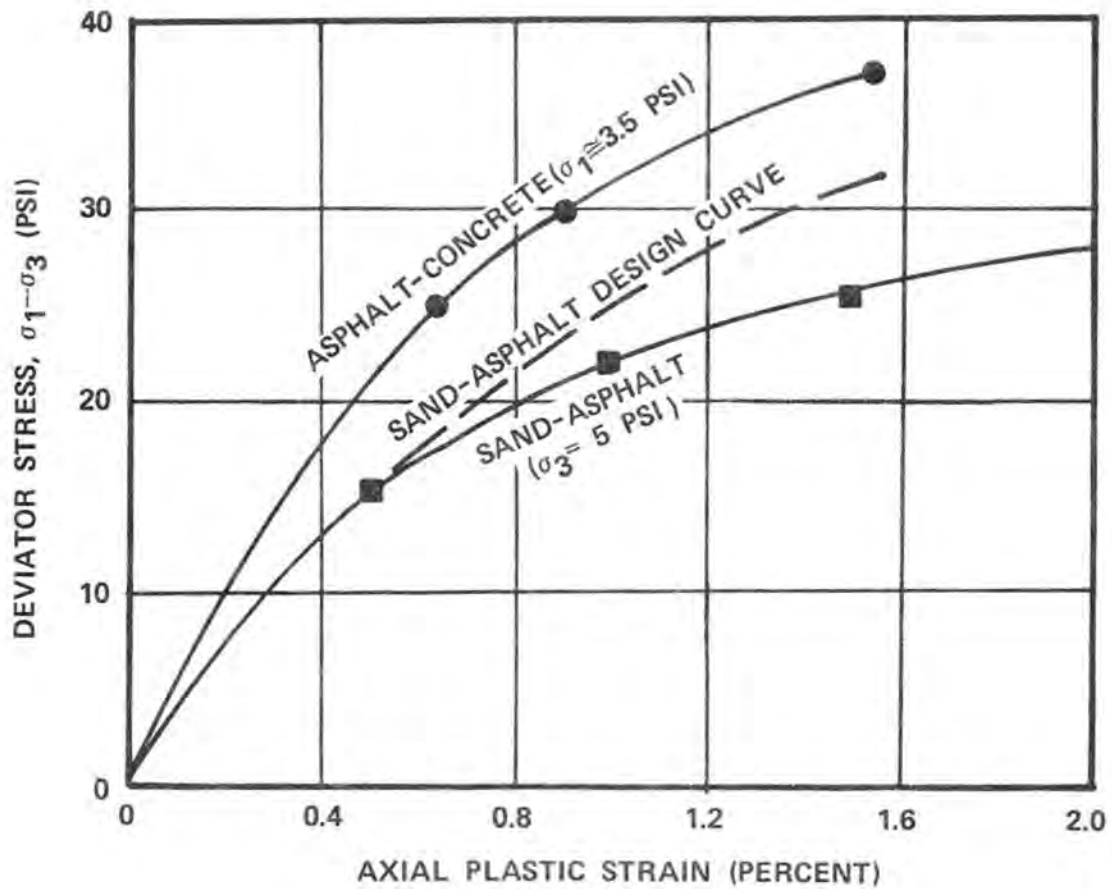


FIGURE 20. VARIATION OF AXIAL PLASTIC STRAIN WITH DEVIATOR STRESS IN ASPHALT-CONCRETE AND SAND-ASPHALT BASE MIXES

stress into the equation analogous to (6), and solve for plastic strain ϵ_p . Substituting the above terms for plastic strain into equation 13 gives the correction factor for stress level:

$$\left\{ \frac{Z \cdot \sigma_o - 5}{55 - Z \cdot \sigma_o} \right\} \dots \dots \dots (14)$$

Substituting equation (9) and (14) into (6) gives

$$\delta = \left(\frac{Z \cdot \sigma_o - 5}{55 - Z \cdot \sigma_o} \right) \left(\frac{2.1 N}{N+1320} \right) \lambda_1 \cdot \epsilon_p \cdot H \dots \dots (15)$$

This equation is suitable for use in calculating rut depths in pavements using sand-asphalt mixes and may also be applicable for some pavements having asphalt contents greater than about 6 to 6 1/2 percent.

Marianna Z-Functions

Appropriate Z factors are now determined for use in calculating the rut depth in typical pavements having sand-asphalt and sand-stone blend bases. Since the proposed expression given above for estimating rut depth is correlated with the results observed at the Marianna and Lake Wales test roads, Z-functions are determined representative of these pavements. The Marianna Test Road had a subgrade modulus of elasticity that varied between 10,000 and 20,000 psi (70,000 and 140,000 kN/m²) as determined in the field by plate load tests. A modulus of 10,000 psi (70,000 kN/m²) was taken to be a conservative value and hence was used in the analysis. Using the results from the temperature distribution analysis for the Marianna pavement sections, the ratio of the modulus of elasticity of a combined 3 in. (75 mm) asphalt concrete surface course and 6 in. (150 mm) sand-asphalt base course to the subgrade modulus was found to be approximately 25.5. The generalized relationship between the Z-function and layer for this modular ratio is shown in Fig. 21. The layer thickness is expressed in the dimensionless form a/H, where a is the radius of loading and H the layer thickness. For a typical Marianna pavement section having a 3 in. (75 mm) asphalt concrete surfacing and 6 in. (150 mm) or 8 in. (200 mm) sand-asphalt base, the corresponding Z factors were found from Fig. 21 to be 0.370 and 0.352, respectively. Linear elastic layered theory was used to calculate the above Z-functions for a single axle dual wheel loading. Each layer was assumed to have a Poisson's ratio of 0.5, although this variable was found not to have a significant effect on the Z-function.

As shown on the figure, the Z values for the Marianna sections are somewhat lower than those found by Ngowtrakul [62] for pavements having a 4,000 psi (27,600 kN/m²) subgrade modulus and temperature conditions that exist at Marianna, Florida. Since the temperature distributions and mean rutting temperatures at Macon, Lake Wales and Marianna were

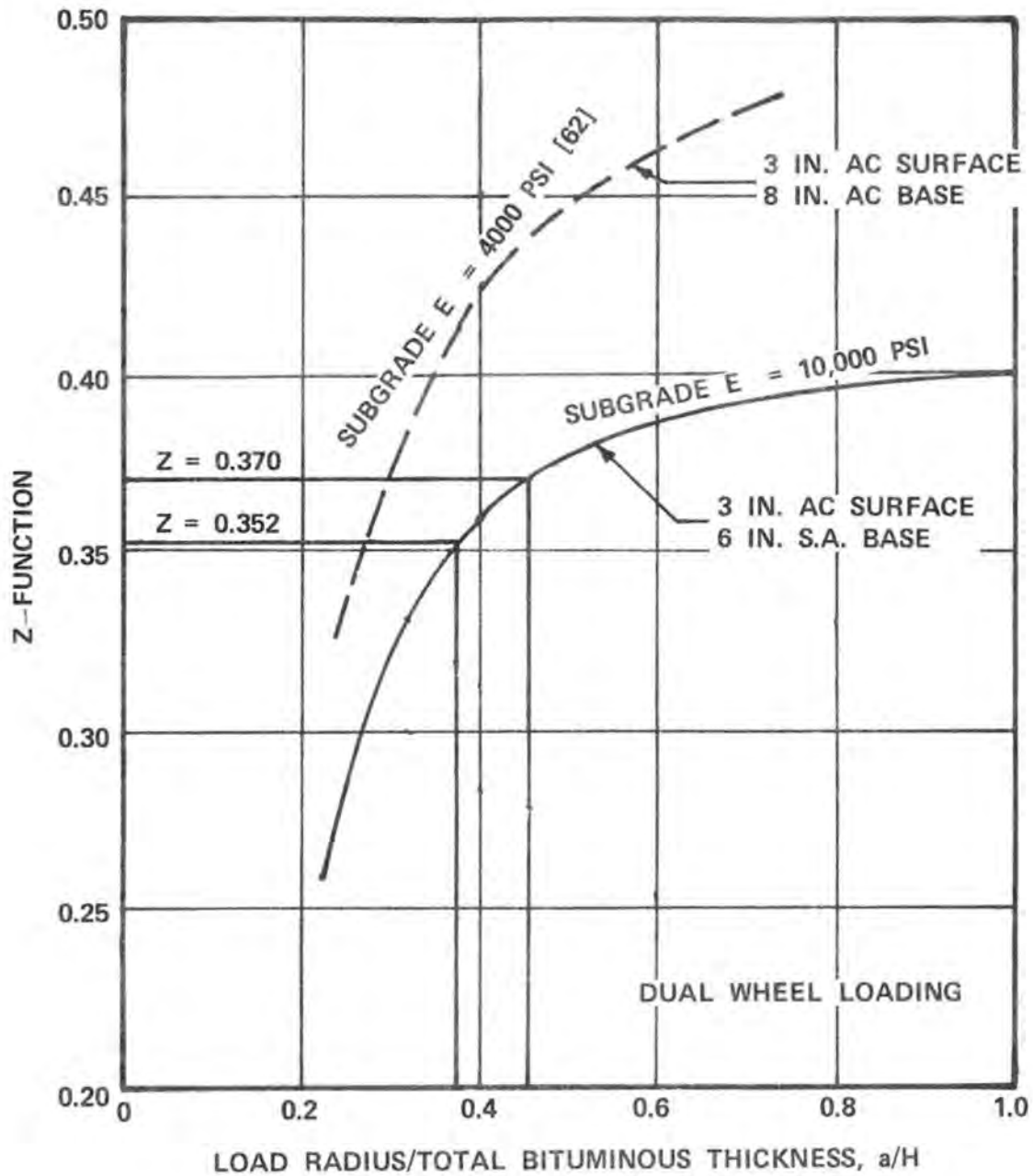


FIGURE 21. VARIATIONS OF Z-FUNCTION WITH SUBGRADE MODULUS, LAYER THICKNESS AND BASE TYPE

all nearly the same, the Z-functions determined for Marianna were also applicable for pavement sections at the other locations.

Development of λ_1 for Sand-Asphalt Pavements

Rut depths in pavements having sand-asphalt bases can be calculated using equation (15). However, before computations can be carried out, appropriate values of λ_1 in this expression must be developed using observed rut depths in pavements with sand-asphalt bases and plastic strain properties of the asphalt layers evaluated using the repeated load triaxial test.

Solving equation (5) for λ_1 gives

$$\lambda_1 = \frac{\delta}{\epsilon_p H} \left\{ \frac{N + 1320}{2.1 N} \right\} \left(\frac{55 - Z \cdot \sigma_o}{Z \cdot \sigma_o - \sigma_3} \right) \quad \cdot \quad (16)$$

For a typical asphalt concrete mix having an asphalt content of 6.5 percent, the results of Barksdale and Miller [54] indicate that the plastic strain is approximately $\epsilon_p = 0.0073$ in./in. for the standard test conditions of 100,000 repetitions, $\sigma_3 = 5$ psi (35 kN/m²) and a temperature of 95°F (35°C). The Marianna test section was subjected to approximately 1,200,000 repetitions of an equivalent 18 kip (80 kN) wheel load, while the Lake Wales section was subjected to approximately 1,160,000 repetitions.

The Marianna sand-asphalt bases had a typical asphalt content of 6.5 percent. For an asphalt content of 6.5 percent, the average of the two larger measured plastic strains out of three tests was 0.0087 in./in. Using these values of plastic strain which are for the standard testing conditions, the weighted average plastic strain in a section with a 3 in. (75 mm) asphalt concrete surface course and a 6 in. (150 mm) sand-asphalt base was 0.00823 in./in. For a corresponding pavement with an 8 in. (200 mm) sand-asphalt base, $\epsilon_p = 0.00832$ in./in. The Marianna test sections had an average measured^p rut depth of 0.32 in. (8 mm) in the 6 in. (250 mm) base sections and 0.26 in. (7 mm) in the 8 in. (200 mm) base sections. For these conditions, the calculated λ_1 were 3.85 and 2.86, respectively.

For the Lake Wales test sections having a 3 in. (75 mm) asphalt concrete surface course and 6 and 8 in. (150 and 200 mm) sand-asphalt base, the average measured rut depth was approximately 0.47 in. (12 mm). The repeated load triaxial tests performed at a confining pressure of 5 psi (35 kN/m²) and a deviator stress of 25 psi (170 kN/m²) failed, and hence had an infinitely large plastic strain. Therefore, the test was repeated using a deviator stress of 20 psi (140 kN/m²) and the same confining stress giving a plastic strain of $\epsilon_p = 0.0091$ in./in. The ratio of plastic strains at selected number of load repetitions for the two stress conditions before failure of the specimen subjected to a 25 psi (170 kN/m²) deviator stress was found to be approximately 5.6. To correct the plastic strain measured at 100,000 repetitions using a deviator stress

of 20 psi (140 kN/m²), this plastic strain was multiplied by two, which is approximately one-third of the 5.6 correction factor. The plastic strain obtained was $\epsilon_p = 0.0182$ in./in. The average plastic strain in the section with the 6 in. (150 mm) base was therefore 0.0146 in./in. and for the 8 in. (200 mm) base section was 0.0152 in./in. For the sections with the 6 in. and 8 in. (150 and 200 mm) sand-asphalt bases, the calculated values of λ_1 were 3.24 and 2.88, respectively.

Tentative values of λ_1 for both asphalt concrete pavement sections and sand-asphalt pavements are given in Table 16. These values of λ_1 were determined by correlating equation 6 and equation 15 with the measured rut depth in pavement test sections. Using the values λ_1 given in Table 16 should give approximately the average total rut depth of pavements having approximately 8 to 12 in. (200 to 300 mm) of asphalt concrete or sand-asphalt. The λ_1 function has been found for the sections studied to apparently decrease somewhat as the pavement thickness increases. Also λ_1 for sand-asphalt pavements is about 20 percent greater than for asphalt concrete pavements.

Table 16. Tentative Recommended Values of λ_1 Obtained by Correlation with Test Pavement Sections

| Total Thickness (in.) | Sand Asphalt Base, λ_1 | Asphalt Concrete Base, λ_1 |
|--------------------------|--------------------------------------|--|
| 9 | 3.9 | 3.3 |
| 11 | 2.9 | 2.5 |

CHAPTER 4

SAND AND GRAVEL SAMPLING PROGRAM AND BASIC MATERIAL PROPERTIES

Introduction

The coastal plain has an abundant supply of natural sedimentary sands which were deposited in relatively recent geologic time in alluvial or marine environments. Presently, sand sources are located on land and in rivers and marine coastal areas. Martin and Hicks [1] proposed alluvial deposits of sand and gravel as one possible source. Swift moving streams and rivers carry a heavy load of sediment including sands from the Piedmont and mountain regions in the northern portion of Georgia to the coast. As the rivers approach the ocean, velocities are significantly reduced causing the particles to settle out. As a result, almost 10,000 cubic yards of coarse and fine sands are deposited in the coastal region each year [1]. Since deposition of sands has occurred for centuries, extensive natural sand deposits are found throughout the coastal region. These are evident both inland, where former deposition created ancient coastlines and old stream channel deposits, as well as in present day river channels where active deposition still occurs.

Walker [2] conducted a preliminary investigation of inland sands obtained from sand pits and concluded that the sands could be used for highway construction when carefully selected and stabilized with asphalt cement. Harding and Woolsey [3] conducted an extensive sampling program of river and marine sands. Approximately 500 one-pound (454 gm) samples obtained by Harding and Woolsey were tested in a preliminary study by Martin and Hicks [1] to determine the feasibility for use of these sands in highway construction. Each sample was subjected to basic tests including specific gravity, grain size analysis, and sand equivalency. Test results are summarized in Table 17 for each of the rivers and marine locations sampled. Although some samples were unsuitable because of high shell content, Hicks and Martin concluded that the sands studied generally could substitute for high-quality aggregates in selected applications.

The one-pound (454 gm) samples tested by Martin and Hicks were too small to perform extensive tests including general mix designs, fatigue and rutting tests to evaluate the sands for use in highway construction. Therefore, the first phase of this investigation was to obtain bulk samples of sand from promising locations as determined by the preliminary investigation conducted by Hicks and Martin.

Table 17. Summary of Physical Properties of Sands Measured by Martin and Hicks [1]

| Location | Gradation Percent Passing | | | | Sand Equivalent | Fineness Modulus | Specific Gravity |
|----------------------|------------------------------|-----|-----|------|--------------------|---------------------|---------------------|
| | #8 | #16 | #50 | #200 | | | |
| Upper Altamaha River | 98 | 89 | 6 | 0 | 92 - 100 | 2.31 - 3.01 | 2.62 |
| Lower Altamaha River | 93 | 76 | 25 | 0 | 80 - 93 | 2.40 - 3.19 | 2.60 |
| Ogeechee River | 100 | 95 | 23 | 0 | 85 - 95 | 1.49 - 2.22 | 2.59 |
| Satilla River | 100 | 100 | 90 | 0 | 76 - 90 | 0.71 - 1.08 | 2.51 |
| Doboy Sound | 85 | 65 | 7 | 0 | 52 - 95 | 2.94 - 3.38 | 2.57 |
| Wassau Sound | 88 | 82 | 60 | 0 | 52 | 1.91 | 2.57 |
| St. Catherines Sound | 98 | 88 | 13 | 0 | 47 | 2.47 | 2.56 |
| Sapelo Sound | 77 | 61 | 4 | 0 | 83 | 3.46 | 2.56 |
| St. Marys River | 96 | 91 | 71 | 0 | 85 | 1.38 | 2.61 |
| Saint Simons Sound | 70 | 59 | 23 | 0 | 73 | 3.31 | 2.49 |
| Continental Shelf | 95 | 81 | 11 | 0 | 98 | 2.66 | 2.58 |
| Ossabaw Sound | 60 | 38 | 4 | 0 | 95 | 4.07 | 2.59 |

Sampling

Sampling the sands in the coastal region was conducted over shallow water by means of a combined air-water uplift sampling system mounted on a low draft barge. The sampling system used was originally developed by Harding and Woolsey [3] in an earlier Georgia Sea Grant Program research project. The combined air-water uplift system was found by Harding and Woolsey to be extremely effective at depths up to 20 ft. (7 m) and performed better than either a water jet or air jet sampling system.

Air-Water Uplift System

The air-water uplift system used in this study is shown in Fig. 22 and 23 and consists of two concentric pipes driven to the desired sampling depth. The inner pipe concentrates a stream of air and water under pressure at the sand by means of a jet bit located at the bottom of the pipe (Fig. 23). The combined air-water jet scours a hole around the bit and forces the sand-water slurry up through the outer pipe to the surface. The sand-slurry mixture is discharged from the sampler at the top of the pipe through the wash head. The sand is removed from the slurry by simple sedimentation principles. Using this system, approximately 3 lbs./min. (1.4 kg/min) of a representative sample of sand is obtained from the depth being sampled.

The basic sampler consisted of an outer pipe 1.5 in. (38 mm) in diameter and 20 ft. (7 m) long enclosing a 0.5 in (12 mm) diameter inner pipe. Other important components of the air-water uplift system were as follows:

- Hammer: The hammer was utilized to drive the pipes to the sampling depths and to remove them after sampling was complete. The 100 lb. (0.4 kN) steel hammer was lifted by a pulley system attached to a 5 horsepower cathead winch located on the barge deck. To advance the drill pipe, the hammer was dropped on the anvil above the washhead. To raise the drill pipe, the hammer was lifted repeatedly against the shock rods causing them to react against the washhead thus lifting the washhead and drill pipe.
- Compressor: The compressor supplied air at a pressure of 60 psi (400 kN/m²) to the drill pipe. A Sears (Model 30 H 17516N) air compressor was used which had a rated capacity of 17 SCFM at 175 psi (1200 kN/m²).
- Water Pump: The water pump was a 1.25 in. (31 mm) Jabsco pump rated at 30 GPM at a pressure of 40 psi (280 kN/m²). The water pump was connected to the air compressor by a hose which in turn led to the drill pipe.
- Spuds: The spuds provided the anchoring system for the barge and consisted of 3 in. (75 mm) dia. steel pipes which were dropped through guides to the bottom of the river

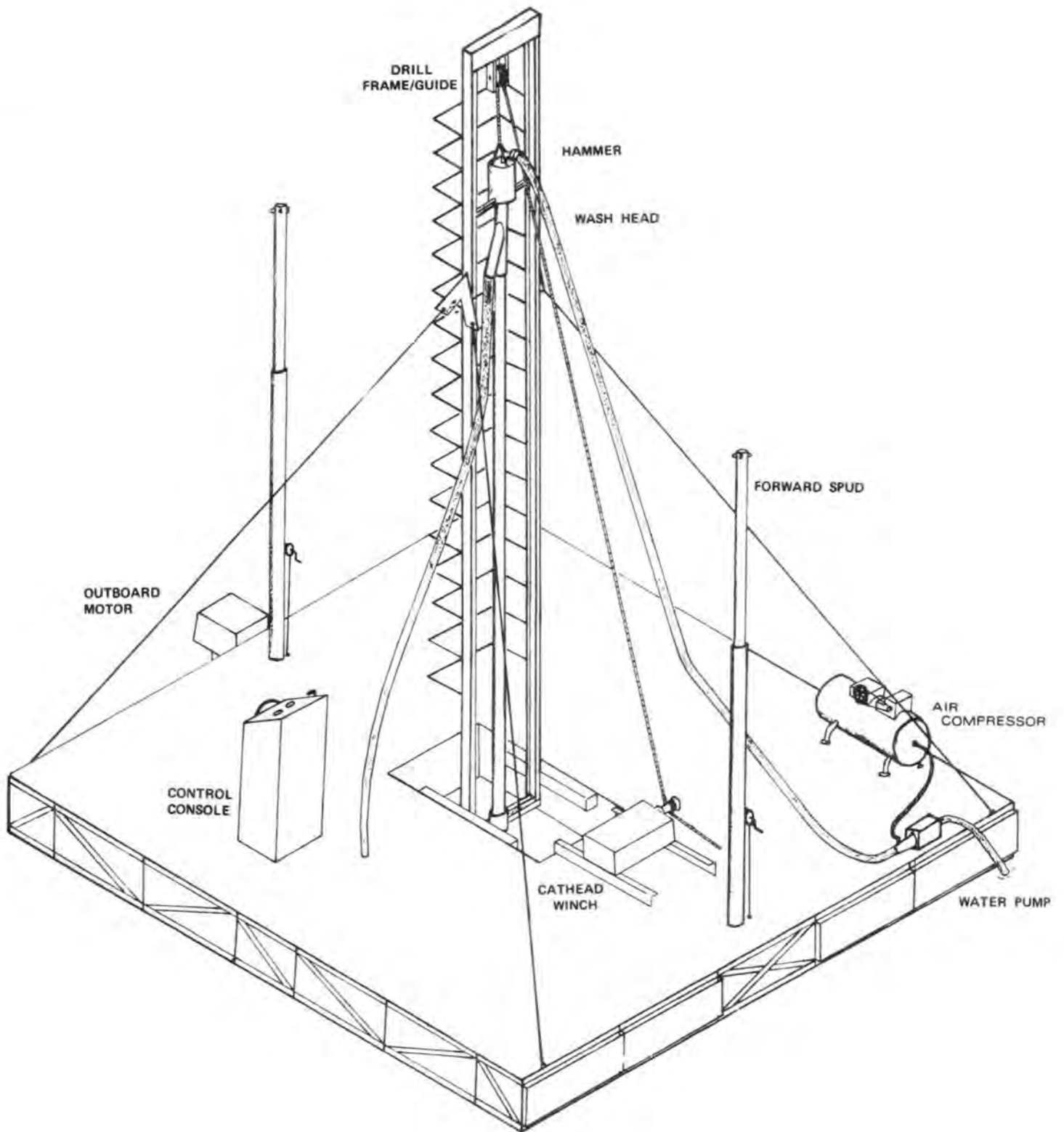


FIGURE 22. DRILL BARGE USED FOR SAMPLING THE ALTAMAHA RIVER

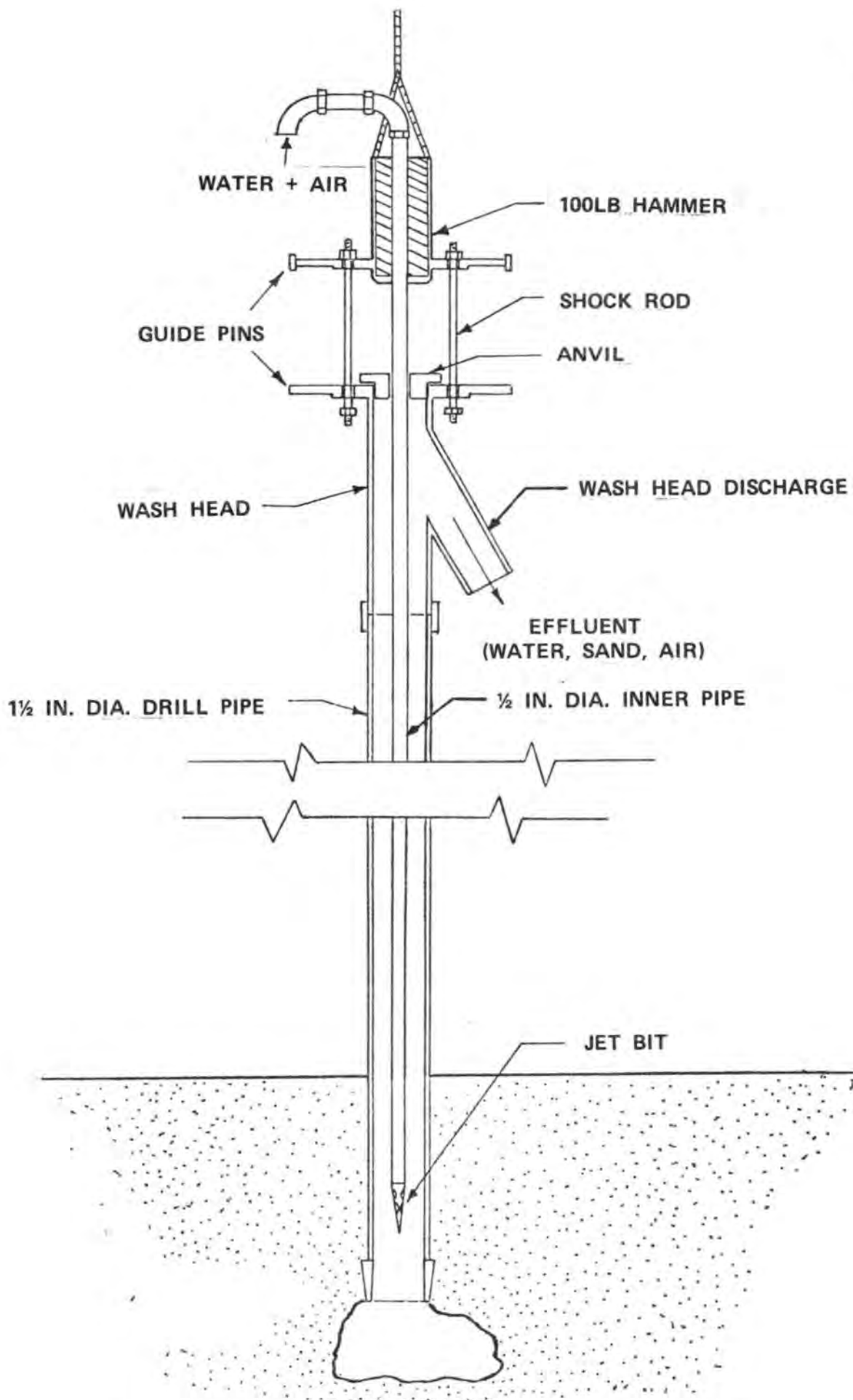


FIGURE 23. AIR-WATER UPLIFT SAMPLING SYSTEM

channel. The ends of the spud pipes were left open and penetrated the soft sediment on the river bottom. Upon completion of drilling operations, the spud pipes were pulled up to the deck level using a hand operated, 0.5 ton (4.4 kN) winch attached to each spud.

The general sampling procedure was as follows:

1. a site was selected and the barge was maneuvered over it,
2. the spuds were dropped to stabilize the barge,
3. the air compressor and water pump were started,
4. the drill pipe was driven to the sampling depth by means of the hammer/cathead winch system,
5. sand samples were collected on the surface of the barge through the washhead discharge,
6. steps 4 and 5 were repeated for each desired sampling depth,
7. the drill pipe was removed by the upward force of the hammer against the shock rods,
8. the water pump and air compressor were turned off, and
9. the spuds were winched to deck level.

Sampling Locations

Preliminary tests conducted by Martin and Hicks [1] indicated that the lower reaches of the Ogeechee and Altamaha Rivers contained sands relatively free of shell and clay and generally had good sand equivalent values. Therefore, bulk samples of sand were obtained from the Ogeechee and Altamaha Rivers in the vicinity of Richmond Hill and Darien, Ga. at the locations shown on Figs. 24 and 25. Sand samples were obtained from the Ogeechee at 24 locations (Fig. 24). At each location, approximately 50 lbs. (0.22kN) of sand were obtained at depths of 5, 10, 15 and 20 ft. (1.5, 3, 4.5, and 6 m). The barge Harding and Woolsey utilized was refurbished and used for sampling on the Ogeechee River. The nearly 5,000 lbs. (22 kN) of sand obtained using this barge made operation and maneuverability difficult. For this reason, the barge was redesigned and assembled specifically for the bulk sampling operations (Fig. 22). A 415 HP Johnson outboard motor made this version of the barge self-propelled. In addition, a new guide system composed of two steel channels connected together to form a rigid frame was used and greatly improved the ease of bulk sampling by holding the drill pipe vertical. Bulk sand samples were obtained from the Altamaha River at 15 locations in the general vicinity of Darien, Ga. as shown on Fig. 25. Due to the tremendous sediment load, the Altamaha River became highly braided in this area resulting in the Darien, Butler, Champney, and South Altamaha Rivers. The South Altamaha River is maintained as a navigable waterway by dredging and is relatively free of sand deposits. The Darien, Butler and Champney Rivers are not dredged, and as a result considerable deposits of sand are found in these rivers. Sampling was carried out from depths of 5 to 20 ft. (1.5 to 6 m) on the Altamaha River. Additional bulk sand samples were collected near the surface to insure adequate quantities for testing. Approximately 3 tons (27 kN) of sand were obtained from the Altamaha River.

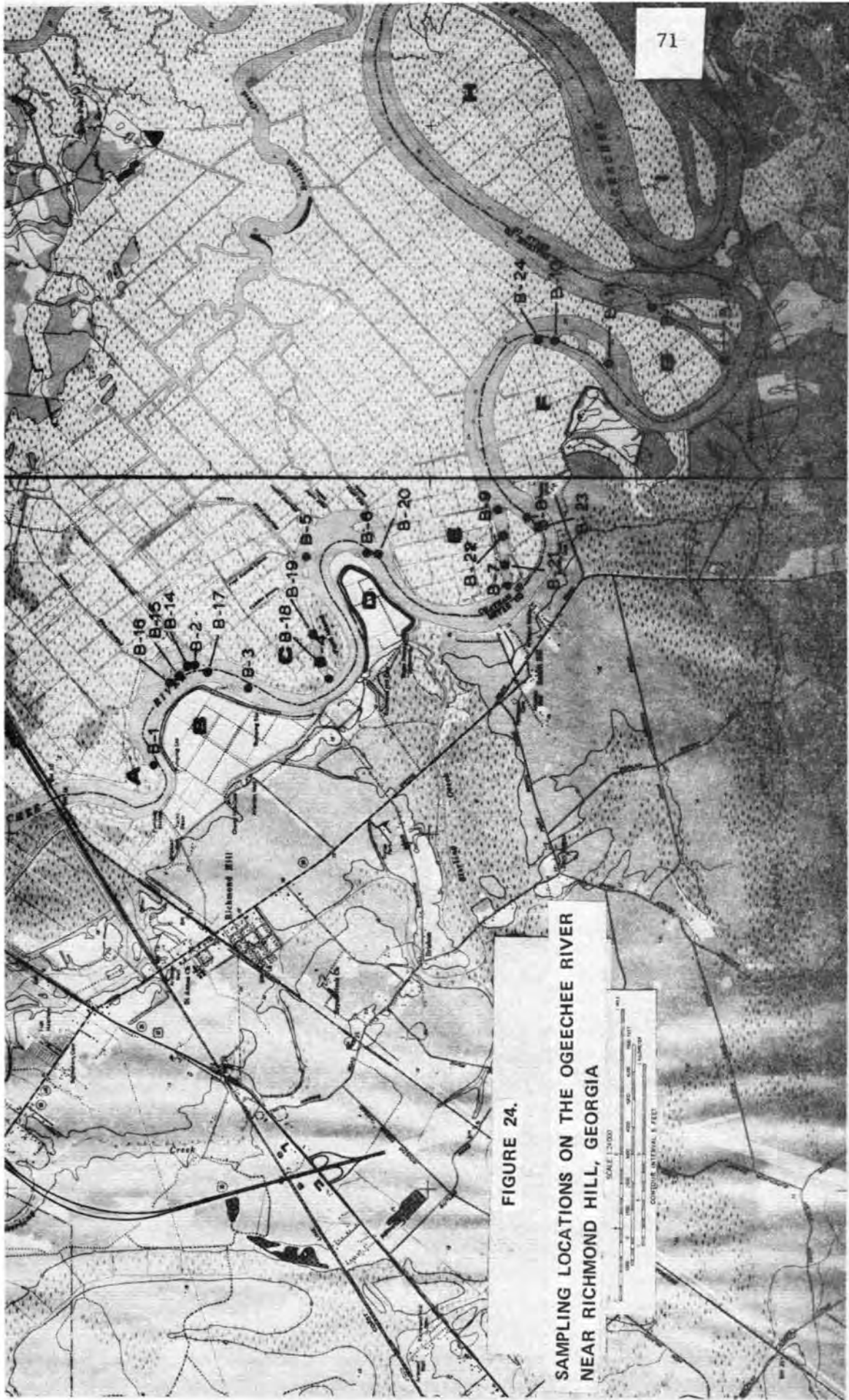


FIGURE 24.
SAMPLING LOCATIONS ON THE OGEECHEE RIVER
NEAR RICHMOND HILL, GEORGIA

FIGURE 25. SAMPLING LOCATIONS ON THE ALTAMAHA RIVER NEAR DARIEN, GEORGIA



Basic Material Tests

After sampling operations were completed, the sands were transported to the Geotechnical/Materials Laboratory at the Georgia Institute of Technology for testing. To obtain a general evaluation of the materials sampled, basic material tests were performed including grain size analysis (ASTM C33)¹, specific gravity (ASTM C128), and sand equivalency (GHD63)².

In addition, a new shape factor test described by Freeme [50] was performed to evaluate the overall effect of shape and surface texture of the sand. This test consisted of determining the time required for sand to flow through a standard size orifice so that light could be seen through the hole. The specific procedure used for determining the shape factor follows [50]:

1. Sieve the sand through a 5.75 mm sieve to remove material which could clog the hole in the test apparatus.
2. Pour the sand retained on the sieve into a lucite cylinder (Fig. 26).
3. Level the top of the sand without causing compaction of the sand in the cylinder, and place a collection pan beneath the plugged hole. Remove the cork from the bottom of the cylinder while at the same time starting a stop watch.
4. Allow the sand to flow from the cylinder until light is seen through the bottom hole, at which time the flow of sand and the watch are stopped.
5. Weigh the sand which has flowed out the hole during the timed period.
6. Calculate the shape factor from the following expression:

$$\text{Shape Factor} = Q / \{ D^{2.5} G_s g^{0.5} (\bar{d} - 5)^{.25} (0.163) \} \quad . . . (17)$$

where: Q = weight of sand which flows from the container divided by the time recorded on the stop watch (gm/sec.)
 D = Diameter of the hole in the bottom of the cylinder (cm)
 G_s = Specific Gravity of the sand
 g = Acceleration due to gravity (cm/sec.²)
 \bar{d} = Diameter D_{50} of the sand at which 50 percent of the sand is smaller (cm)

The Altamaha and Ogeechee River sands sampled were generally a clean, uniform, subrounded to subangular, fine to medium sand. Typical micrographs magnified to 7 times the original size of selected sands

¹American Society for Testing Materials, Standard Test Procedures.
²Georgia Department of Transportation, Standard Test Procedures.

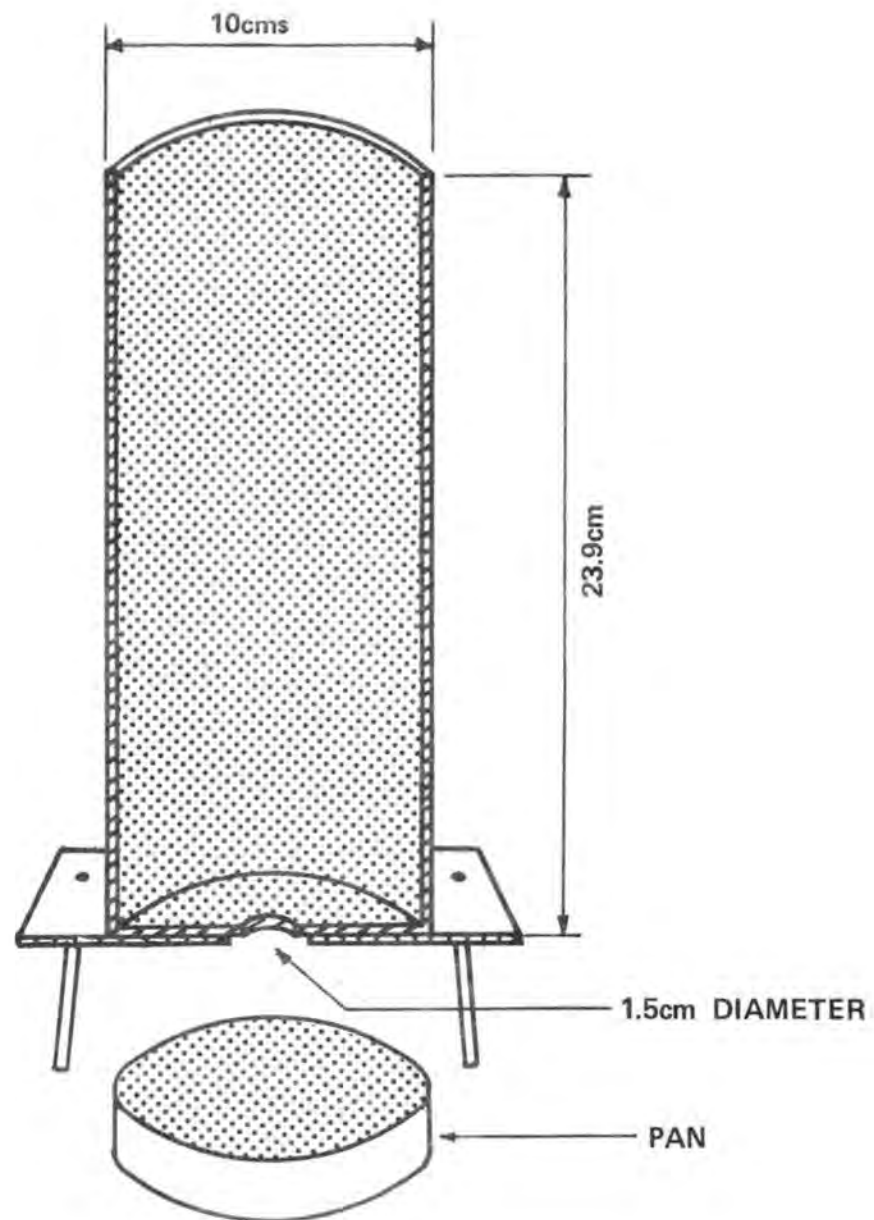
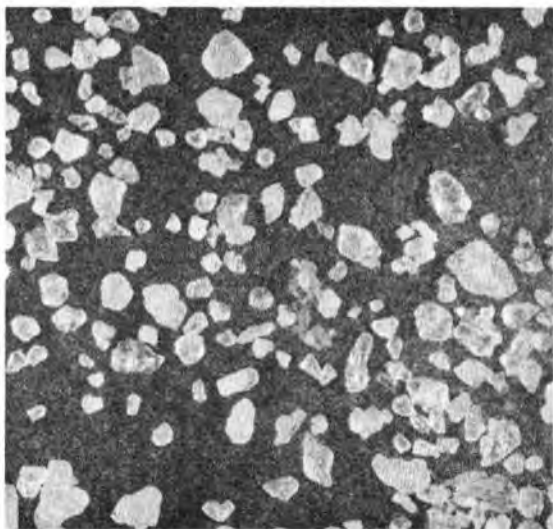
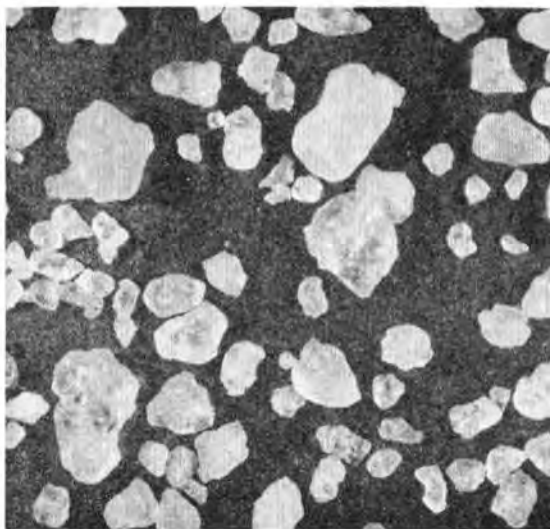


FIGURE 26. APPARATUS FOR DETERMINING THE SHAPE FACTOR FOR SANDS (After Freeme)

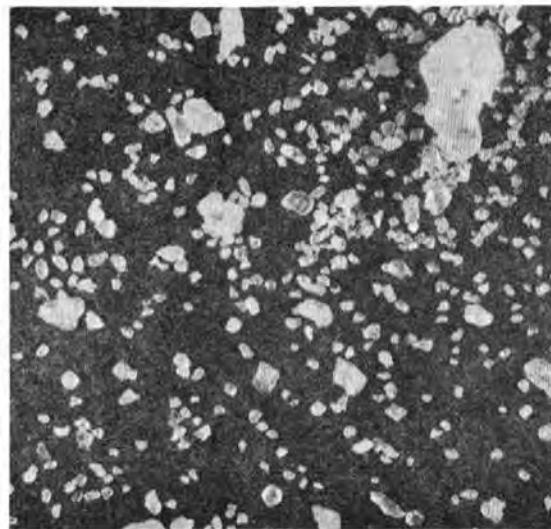
are shown in Fig. 27. The average percent passing the No. 200 sieve for the Ogeechee River sand was 0.5 percent, and one percent for the Altamaha River sand. Trace to moderate amounts of organics usually consisting of fibrous material were found in many of the sand samples. The sand equivalent values of the Ogeechee River sands were generally between 80 and 100, while those for the Altamaha River sands varied from 60 to 100. Descriptions of the bulk samples obtained from the Ogeechee and Altamaha Rivers are given in Tables 18 and 19, respectively, and results of the basic tests performed are summarized in Table 20. Average gradation and gradation limits are shown in Figs. 28 and 29, respectively.



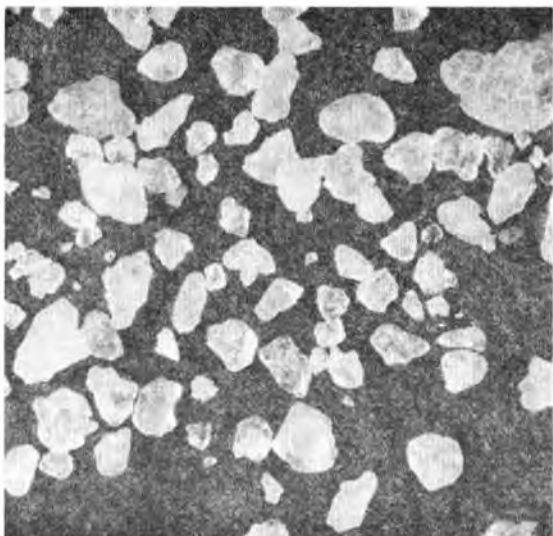
(a) Boring B-2 at 5 ft.



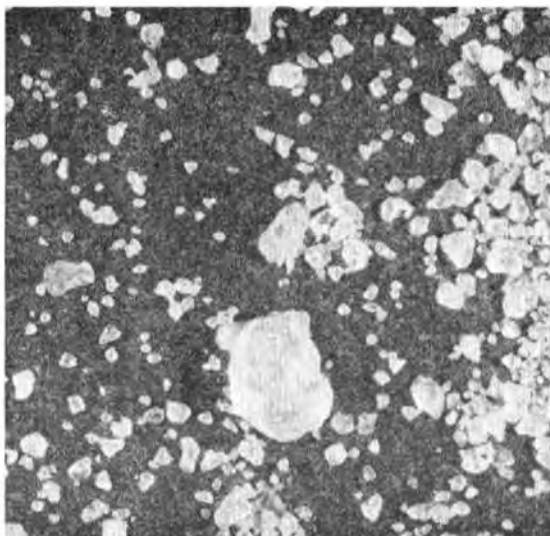
(b) Boring B-3A at 5 ft.



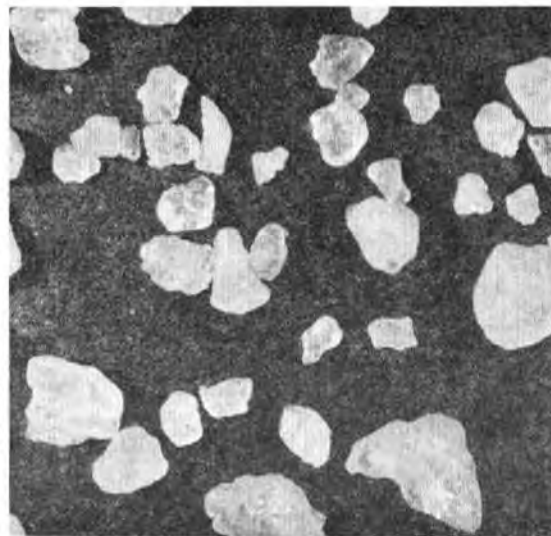
(c) Boring B-3A at 10 ft.



(d) Boring B-7 at 5 ft.



(e) Boring B-7 at 10 ft.



(f) Boring B-7 at 20 ft.

FIGURE 27. SELECTED SAND SAMPLES OBTAINED FROM THE OGEECHEE RIVER

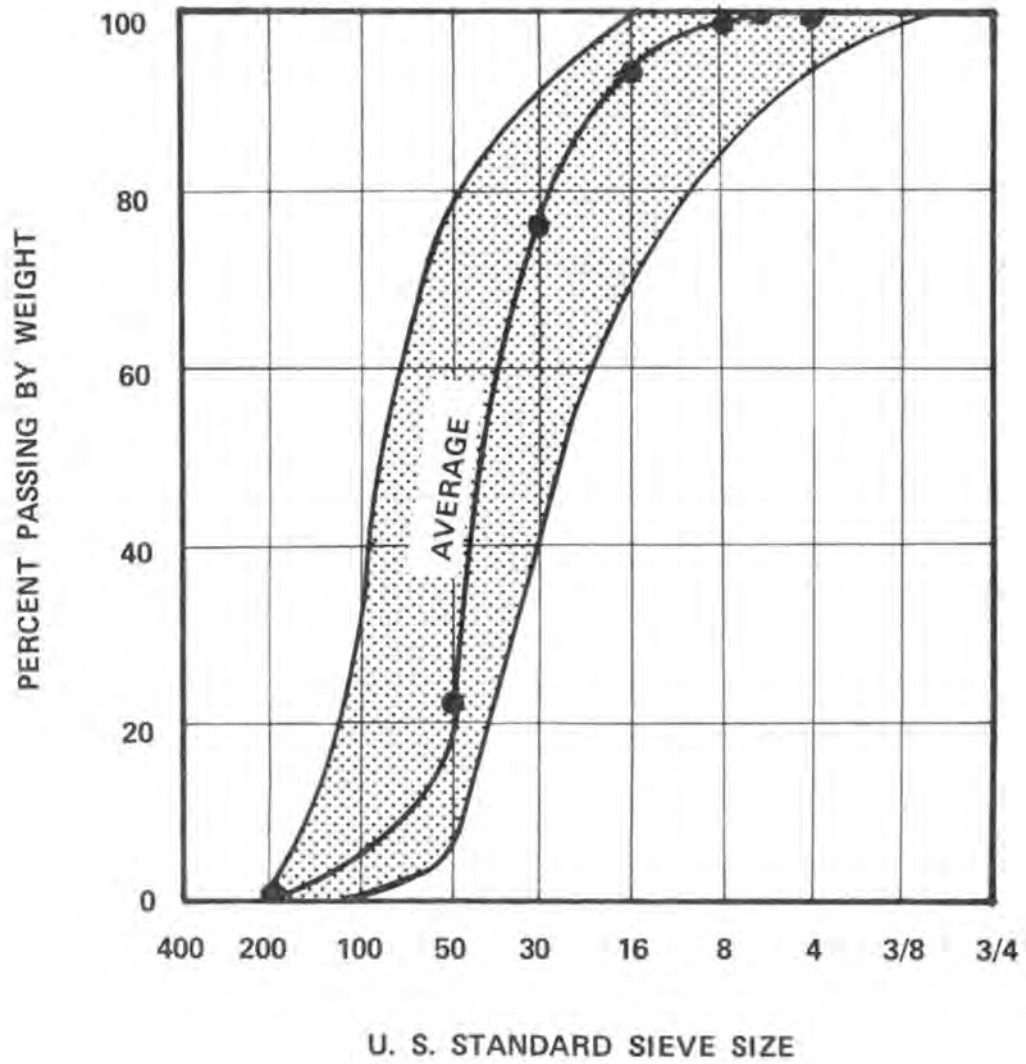


FIGURE 28. GRADATION OF OGEECHEE RIVER SAND

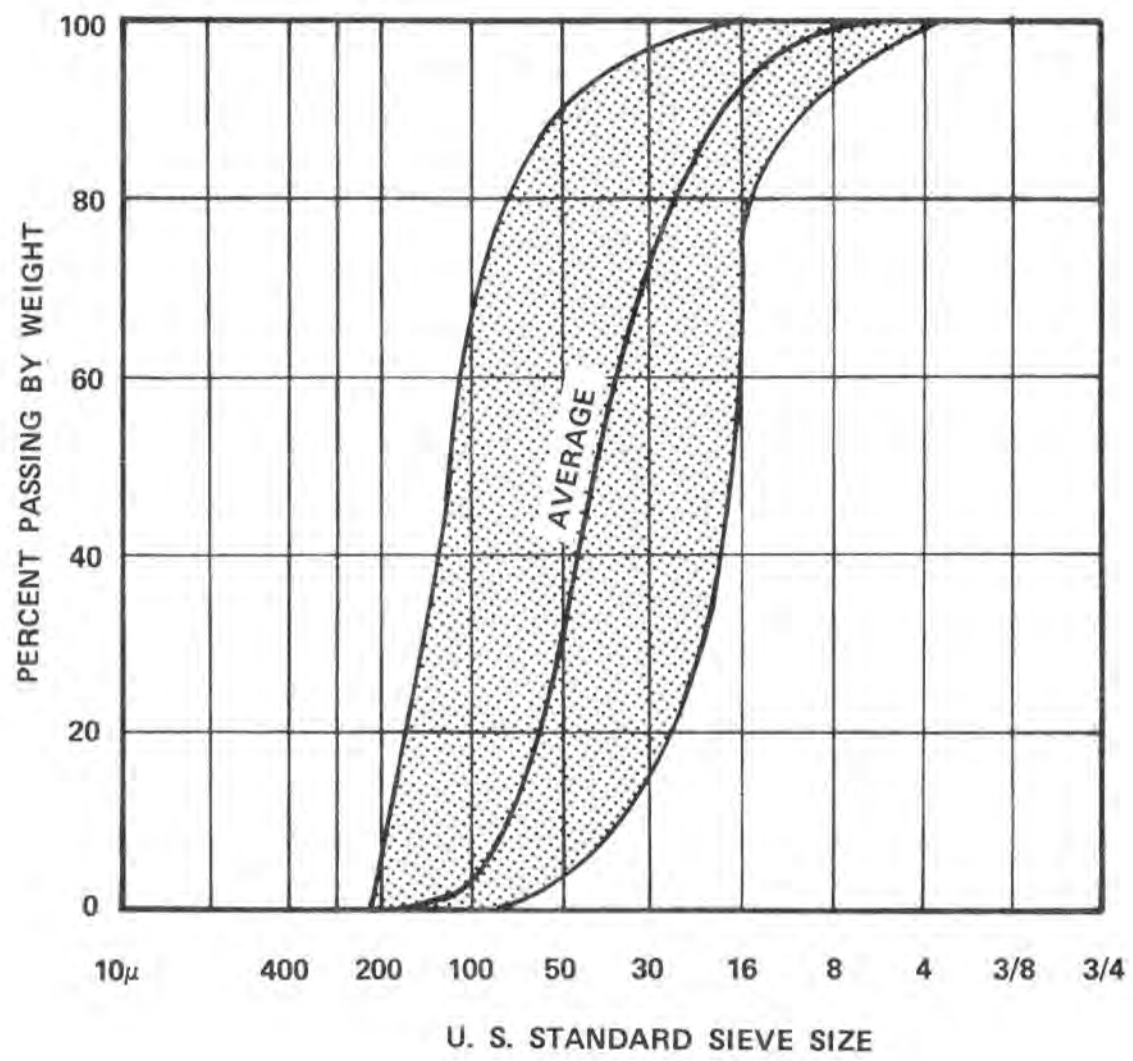


FIGURE 29. GRADATION OF ALTAMAHA RIVER SAND

Table 18. Description of Bulk Sand Samples Obtained From
the Ogeechee River

| | | | |
|------|--|------|---|
| B-1 | 10' light gray, subrounded sand with organic matter 15' subrounded, uniform-graded, fine sand | B-16 | 5' white, subrounded, uniformly graded, fine sand with organic matter and some clay lumps 10' light gray, subrounded, uniformly graded, fine sand with organic matter and some clay lumps |
| B-2 | 5' light brown subrounded sand with organic matter and some mica 10' light gray, subrounded sand with organic matter, clay lumps and some mica 15' light gray, subrounded sand with clay lumps and mica | B-17 | 5' light tan, subrounded, uniformly graded, medium sand with organic matter 10' light, subrounded sand with organic matter 15' light gray, subrounded, uniformly graded, medium sand with organic matter |
| B-3 | 5' light gray, subrounded sand with clay lumps 10' white, subrounded sand with organic matter, clay lumps and mica 20' light gray, subrounded, sand with organic matter | B-18 | 5', 10', 15' light gray, subrounded, uniformly graded sand with organic matter |
| B-3a | 5' light brown, subrounded sand with organic matter 10' light gray, subrounded sand with organic matter, clay lumps and mica | B-19 | 5' light gray, subrounded, uniformly graded, medium sand with organic matter 10' light gray, subrounded sand, with organic matter 15' light gray, subrounded, uniformly graded, fine sand with organic matter |
| B-4 | 5' light tan, subrounded sand with organic matter 10' light gray, subrounded sand with organic matter and clay lumps 15' white, subrounded sand with clay lumps and organic matter 12' light gray, subrounded sand with organic matter | B-20 | 5' light tan, subrounded, uniformly graded, medium sand with organic matter 10' light tan, subrounded sand with organic matter 15' uniformly graded, medium sand |
| B-5 | 10' light gray, subrounded sand with numerous clay lumps and organic matter | B-21 | 5', 15' uniformly graded, medium sand |
| B-6 | 5' light gray, subrounded sand with some organic matter 10' sand with some organic matter 15' white, subrounded sand with some organic matter 20' white, subrounded, uniformly graded, medium sand with some organic matter | B-22 | 10' light gray, subrounded, uniformly graded, medium sand with organic matter 15' light gray, subrounded, uniformly graded, medium sand with organic matter and clay lumps |
| B-7 | 5' light gray, subrounded, sand with organic matter 10' white, subrounded, uniformly graded, fine sand with clay lumps and some organic matter 15', 20' light gray, subrounded, uniformly graded, medium sand with organic matter and clay lumps | B-23 | 10' light gray, subrounded sand with organic matter and clay lumps 15' white, subrounded sand with clay lumps some organic matter |
| B-8 | 10', 15' white, subrounded uniformly graded, medium sand with organic matter and some clay lumps | B-24 | 5' white, subrounded sand with organic matter 10', 15' white, subrounded sand with organic matter and mica |
| B-9 | 10' white, subrounded, uniformly graded, fine sand with organic matter 15' light gray, subrounded sand with organic matter 20' light gray, subrounded, uniformly graded, fine sand with organic matter | | |
| B-10 | 10' white, subrounded, uniformly graded, fine sand with organic matter, some clay lumps and some mica 15' white, subrounded sand with organic matter 20' white, subrounded, uniformly graded, fine sand with organic matter | | |
| B-11 | 15' uniformly graded, medium, subrounded sand | | |
| B-12 | 5' white, subrounded sand with clay lumps 10' white, subrounded, uniformly graded, fine sand with clay lumps 15' light tan, subrounded, uniformly graded, medium sand with organic matter | | |
| B-13 | 5' uniformly graded, fine, subrounded sand | | |
| B-14 | 5' light tan, subrounded, uniformly graded, medium sand with organic matter 10' white, subrounded, uniformly graded, fine sand with organic matter and clay lumps | | |
| B-15 | 5' light gray, subrounded, uniformly graded, fine sand with organic matter and clay lumps 10' light gray, subrounded, sand with organic matter and clay lumps 15' light gray, subrounded, uniformly graded, fine sand with organic matter and clay lumps | | |

Table 19. Description of Bulk Sand Samples Obtained From
the Altamaha River

| | |
|------|--|
| A-1 | Light gray, subrounded, uniformly graded, fine sand, with mica, many shells, and organic matter |
| A-2 | Light gray, subangular, uniformly graded, fine sand, with mica, some shells, and some clay lumps |
| A-3 | Light gray, subrounded, uniform to well graded, fine sand, with few shells, clay lumps, and some organic matter |
| A-4 | Light brown, subangular, uniformly graded, fine sand, with few shells, mica, many clay lumps, and organic matter |
| A-5 | Light brown, subangular, uniformly graded, medium sand with mica |
| A-6 | Clear, subrounded, uniformly graded, fine sand with clay lumps and some organic matter |
| A-7 | Light brown, subrounded, uniformly graded, fine to medium sand, with mica, organic matter and some clay |
| A-8a | Clear with orange tint, subrounded, uniformly graded, fine sand with mica |
| A-8b | Light brown, subrounded, uniformly graded, fine to medium sand, with many clay lumps, some organic matter, and some mica |
| A-9 | Clear with some orange tint, subrounded, uniformly graded, medium sand with some organic matter |
| A-10 | Clear with some orange tint, subrounded, uniformly graded, medium sand with some mica and some organic matter |
| A-11 | Brown, subrounded, uniformly graded, fine to medium sand with clay lumps, organic matter, and some mica |
| A-12 | Clear with some orange tint, subrounded, uniformly graded, medium sand with some organic matter |
| A-13 | Medium sand, uniformly graded (sample not available) |
| A-14 | Clear, subrounded, uniformly graded, fine to medium sand with clay lumps and some organic matter |
| A-15 | Orange-brown, subrounded, uniformly graded, medium sand with some small clay lumps |

CHAPTER 5

SAND-ASPHALT AND SAND-CEMENT MIX DESIGNS

Introduction

Stabilized local sands and sand-stone blends can be utilized in new pavement construction as surface, base and subbase layers. One of the most promising immediate uses of local sands, however, appears to be in base construction. Therefore, the laboratory phase of this study was limited primarily to investigating the use of local sands and sand-stone blends in base course construction. Asphalt and Portland cement were selected as the stabilizing agents since these agents are most commonly used at the present time. While fatigue and rutting characteristics of sand-asphalt mixes and sand-stone asphalt blends are important, the controlling factors in the design of sand-cement bases are fatigue and shrinkage characteristics. However, before fundamental fatigue and rutting properties of these mixes can be studied, mix designs establishing the level of stabilization and density must be developed for each sand blend investigated.

Sand-Cement Mix Designs

Although Portland cement has been utilized as a soil stabilizer for many years, mix design criteria have not been standardized and vary widely with geographic location. The Portland Cement Association [51] developed a comprehensive mix design for soil cement including compressive strength, wet-dry, and freeze-thaw tests. Most state transportation agencies, however, developed more simplified design approaches related approximately to existing local environmental conditions. The Georgia Department of Transportation has used a 7-day unconfined compressive strength of 300 psi (2100 kN/m²) or greater [52] as the design criteria. Because of the relatively mild climate in the southeast and the successful use of an unconfined compressive strength design criteria by many transportation agencies, this procedure was selected as the basic mix design method for this study. All mixes prepared used Type I Portland cement.

The amount of cement required to give a cement-stabilized base the strength necessary for satisfactory performance was found to be inversely proportional to the amount of fines in the mix up to approximately 15 percent [51]. The sands encountered along the Altamaha and Ogeechee Rivers contained only about one percent fines. As a result, excessive amounts of cement would be required for the necessary strength using the existing gradation. To reduce the required quantity of cement and upgrade the material, either crushed stone fines or flyash fines was added to the sand.

Table 20. Results of Tests Run on Altamaha Sands

| Boring No. | Gradation Percent Passing | | | | Sand Equivalent | Fineness Modulus | Shape Factor | Specific Gravity |
|------------|------------------------------|-----|-----|------|-----------------|------------------|--------------|------------------|
| | #8 | #16 | #50 | #200 | | | | |
| A1 | 98 | 86 | 60 | 1 | 68 | 1.65 | | 2.68 |
| A2 | 98 | 89 | 64 | 2 | 70 | 1.54 | | 2.59 |
| A3 | 94 | 85 | 28 | 1 | 57 | 2.21 | | 2.68 |
| A4 | 98 | 93 | 46 | 1 | 68 | 1.81 | 0.6863 | 2.61 |
| A5 | 98 | 87 | 5 | 0 | 98 | 2.64 | 0.0691 | 2.62 |
| A6 | 100 | 100 | 68 | 0 | 95 | 1.32 | 0.6968 | 2.65 |
| A7 | 100 | 99 | 36 | 1 | 94 | 1.78 | 0.8091 | 2.65 |
| A8a | 100 | 99 | 33 | 1 | 93 | 1.87 | 0.7840 | 2.65 |
| A8b | 100 | 100 | 90 | 6 | 31 | .96 | 0.6438 | 2.67 |
| A9 | 96 | 74 | 9 | 0 | 98 | 2.77 | 0.9107 | 2.66 |
| A10 | 99 | 90 | 18 | 0 | 93 | 2.41 | 0.8403 | 2.67 |
| A11 | 100 | 98 | 21 | 0 | 92 | 1.99 | 0.7579 | 2.69 |
| A12 | 100 | 96 | 7 | 0 | 96 | 2.34 | 0.8695 | 2.66 |
| A13 | 99 | 93 | 3 | 0 | 96 | 2.57 | 0.8200 | 2.67 |
| A14 | 100 | 99 | 23 | 0 | 95 | 1.91 | 0.8185 | 2.67 |
| A15 | 100 | 98 | 6 | 0 | 98 | 2.25 | 0.8146 | 2.67 |
| B1 | 99 | 93 | 10 | 0 | 98 | 3.64 | | 2.65 |
| B2 | 100 | 99 | 63 | 1 | 92 | 1.32 | | 2.68 |
| B3 | 96 | 82 | 9 | 1 | 98 | 3.32 | | 2.67 |
| B4 | 97 | 88 | 17 | 0 | 98 | 3.66 | | 2.67 |
| B6 | 99 | 90 | 17 | 2 | 96 | 2.31 | 0.8284 | 2.67 |
| B7 | 100 | 91 | 12 | 0 | 97 | 2.53 | 0.7763 | 2.66 |
| B8 | 100 | 96 | 16 | 0 | 95 | 2.16 | 0.8607 | 2.68 |
| B9 | 100 | 98 | 20 | 0 | 92 | 1.90 | 0.7780 | 2.65 |
| B10 | 100 | 96 | 16 | 1 | 91 | 2.01 | 0.8421 | 2.67 |
| B11 | 100 | 100 | 20 | 2 | 91 | 1.80 | 0.8178 | 2.67 |
| B12 | 85 | 70 | 11 | 1 | 92 | 3.00 | 0.9338 | 2.68 |
| B13 | 99 | 99 | 78 | 2 | 84 | 1.13 | 0.6604 | 2.64 |
| B14 | 100 | 100 | 18 | 0 | 89 | 1.84 | 0.8583 | 2.66 |
| B15 | 100 | 99 | 18 | 1 | 92 | 1.92 | 0.7764 | 2.69 |
| B16 | 100 | 99 | 13 | 0 | 95 | 1.98 | 0.8450 | 2.62 |
| B17 | 100 | 97 | 7 | 0 | 97 | 2.17 | 0.8970 | 2.63 |
| B18 | 96 | 83 | 8 | 0 | 96 | 2.59 | 0.9107 | 2.62 |
| B19 | 100 | 96 | 23 | 0 | 97 | 2.09 | 0.8639 | 2.66 |
| B20 | 99 | 90 | 34 | 0 | 93 | 2.09 | 0.8479 | 2.67 |
| B21 | 100 | 94 | 17 | 1 | 90 | 2.19 | 0.8620 | 2.66 |
| B22 | 99 | 92 | 17 | 0 | 92 | 2.26 | 0.8773 | 2.67 |
| B23 | 100 | 100 | 10 | 1 | 97 | 2.10 | 0.8567 | 2.65 |
| B24 | 99 | 97 | 65 | 0 | 82 | 1.49 | 0.8657 | 2.66 |

Table 21. Sand-Cement Mixes Used in Beam Fatigue Tests

| MIX | BLENDS | | | Cement Cont. (%) | Comp. (1) Str. (psi) | Compaction (3) | | Gradation (% Passing) | | | | | | SOURCE |
|------|-------------|--------------|--------------|---------------------|----------------------------|----------------------|----------------------------|-----------------------|-------|--------|--------|---------|--------------------|--|
| | Sand (%) | Stone (%) | Fines (%) | | | Opt. ω (%) | γ_d^{\max} (pcf) | 318 in. | No. 8 | No. 16 | No. 50 | No. 100 | No. 200 | |
| PALT | 100 | 0 | 0 | 8.5 | 375 | 9.0 | 121.3 | 100 | 99 | 93 | 27 | 3 | 1 | Altamaha River Sand |
| APF | 95 | 0 | 5 | 8.0 | 335 | 9.0 | 123.1 | 100 | 99 | 93 | 30 | 7 | 6 | Altamaha River Sand (1% fines) with Crushed Stone Fines |
| APS | 47.5 | 47.5 | 5 | 6.0 | 630 | 7.0 | 130.1 | 97 | 73 | 62 | 22 | 7 | 6 | Combined Gradation |
| | | | | | | | | 100 | 99 | 93 | 27 | 3 | 1 | Altamaha River Sand |
| | | | | | | | | 94 | 45 | 28 | 10 | 0 | 100 ⁽²⁾ | Crushed Granite Gneiss |
| PPS | 100 | 0 | 0 | 6.0 | 315 | 9.0 | 123.1 | 100 | 100 | 97 | 42 | 16 | 10 | Finley Co. Pit Sand |
| PS | 50 | 50 | 0 | 6.0 | 695 | 7.5 | 132.0 | 97 | 73 | 62 | 26 | 8 | 5 | Combined Gradation |
| | | | | | | | | 100 | 100 | 97 | 42 | 16 | 10 | Finley Co. Pit Sand |
| | | | | | | | | 94 | 45 | 28 | 10 | 0 | 0 | Crushed Granite Gneiss |

1. Seven Day Unconfined Compressive Strength
2. Crushed Granite-Gneiss Fines
3. Standard Proctor Compaction Test (AASHTO T-99)

To obtain a wide variation of strength and material properties, mixes were prepared for both natural sands and blends having up to 50 percent stone. A single sand gradation was used in all tests to minimize effects of variation in gradation. The gradation used corresponds to the average gradation of all locations sampled on the Altamaha and Ogeechee Rivers. Altamaha River sands were sieved and recombined to give the gradation and basic properties shown in Fig. 30. By using only Altamaha River sand, the effects of variation in grain size, shape and surface texture were minimized. The properties of the standard crushed granite gneiss stone used in the cement stabilized stone-sand blends are summarized in Fig. 31. Crushed stone was obtained from the Norcross Quarry of Vulcan Materials Company. The granite gneiss stone was sieved and regraded to Georgia DOT specifications for coarse aggregate, except that aggregate larger than the 3/8 in. size was scalped and replaced with finer material to minimize the effects of large aggregates in the specimen.

A standard Proctor moisture density test (AASHTO T-99) was performed on each mix to determine the optimum density and water content, and test results are given in Appendix A. Specimens then were prepared in Proctor molds for unconfined compression testing at optimum density and moisture content. The required amounts of sand, aggregate, cement and water were weighed out separately, combined and blended thoroughly in a mixing bowl. The specimens were compacted in three layers of equal thickness. For each layer, one-third of the mixture was spooned into the mold, and the material was compacted by 25 blows of the standard Proctor hammer. The surface was scarified between each layer to prevent the formation of weak planes, and the specimen was then removed from the mold with a hydraulic extruder, sealed in a plastic bag, and stored in a moisture room. Placing the specimen in a plastic bag prevented fine material in the soil cement specimen from washing away during curing in the moisture room. After a curing period of 7 days, the cylindrical specimens were removed from the moisture room and weighed. The ends were then capped with a sulfur compound, and the specimens were tested in unconfined compression.

The cement contents used in the blends were estimated from the empirical relationships based on dry density developed by the Portland Cement Association [51]. The blends investigated for fatigue resistance consisted of the combinations of sand, crushed stone, Portland cement, crushed stone fines, and flyash and are presented in Appendix A and Table 21. Mixes varied from natural sand with 5 percent crushed stone fines to blends having 23 percent sand and 70 percent stone. Cement contents varied from 6 to 8.5 percent and 7-day unconfined compressive strengths from 315 to 695 psi (2170 to 4790 kN/m²). These sand blends had combined gradations having 1 to 10 percent fines. The corresponding moisture-density and stress-strain relationships for these and other blends studied are presented in Appendix A. To determine the effect of a lower cement content and reduced amount of fines on the unconfined strength, tests SC-11 through SC-13 were performed on selected blends (Appendix A). The maximum strengths for the pure sand blend, the 75/25 sand/stone blend, and the 50/50 sand-stone blend, were 74, 59, and 107 psi (510, 407, and 738 kN/m²), respectively. Because of the small amount of fines and low cement contents, none of these blends met the usually

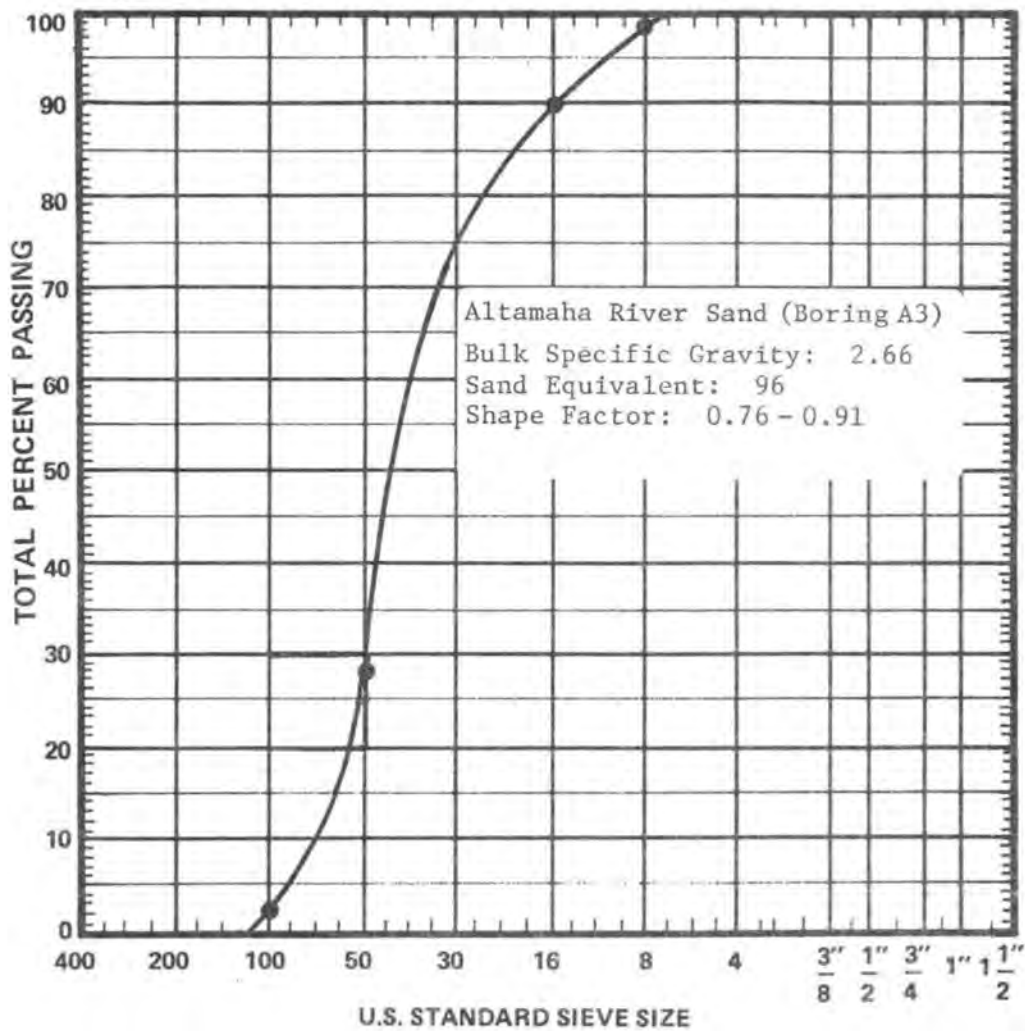


FIGURE 30. GRADATION OF THE RIVER SAND USED IN SAND-ASPHALT AVERAGE OF OGEECHEE AND ALTAMAHA

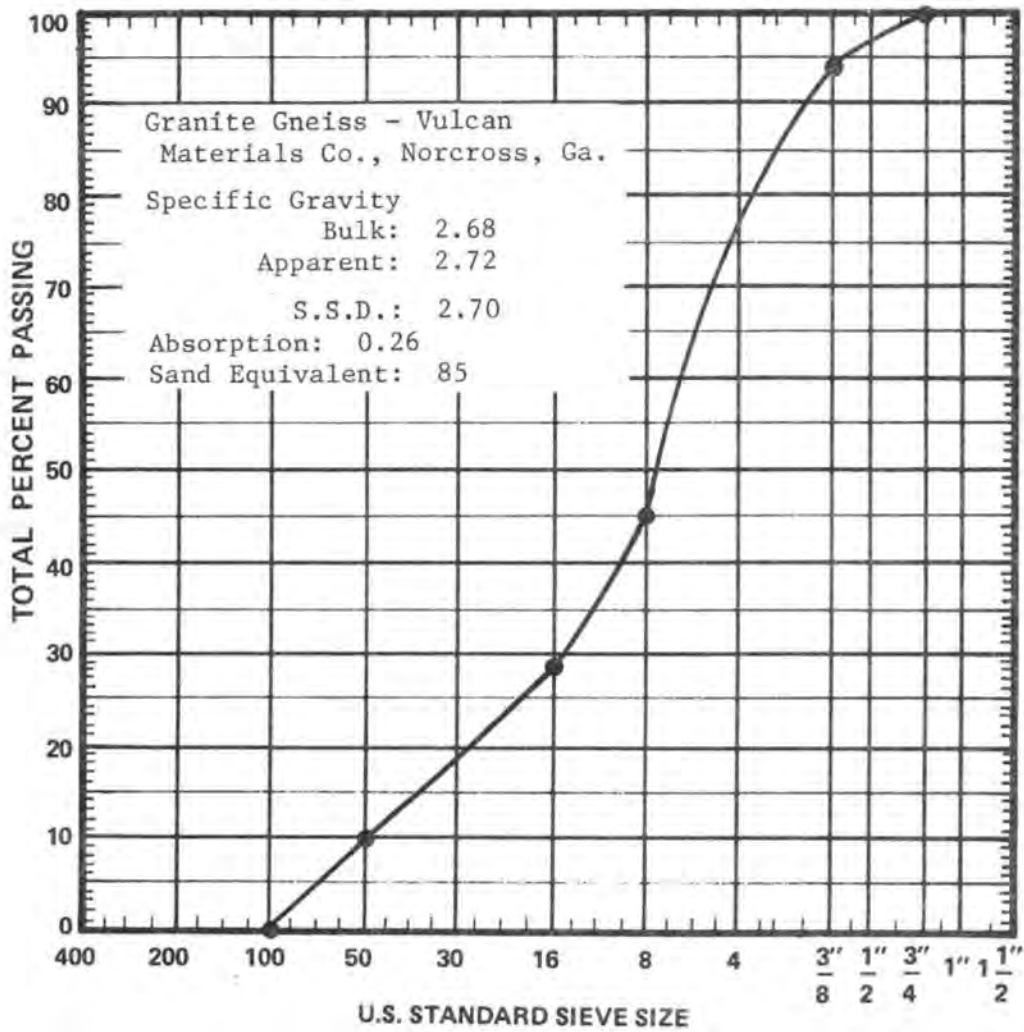


FIGURE 31. GRADATION OF THE COARSE AGGREGATE USED IN SAND BLEND ASPHALT MIXES

FIGURE 32. FINLEY PIT SAND ASPHALT MIX - 461 lb. MARSHALL STABILITY

DESIGNATION: FP

SAND: 100%

SOURCE: FINLEY CO. PIT

OPTIMUM % AC: 6.5
 STABILITY (LBS.) 461
 FLOW (1/100 IN): 9
 % VOIDS-TOTAL MIX: 15.0
 % VMA 25.0
 UNIT WGT-TOTAL MIX: 126.7

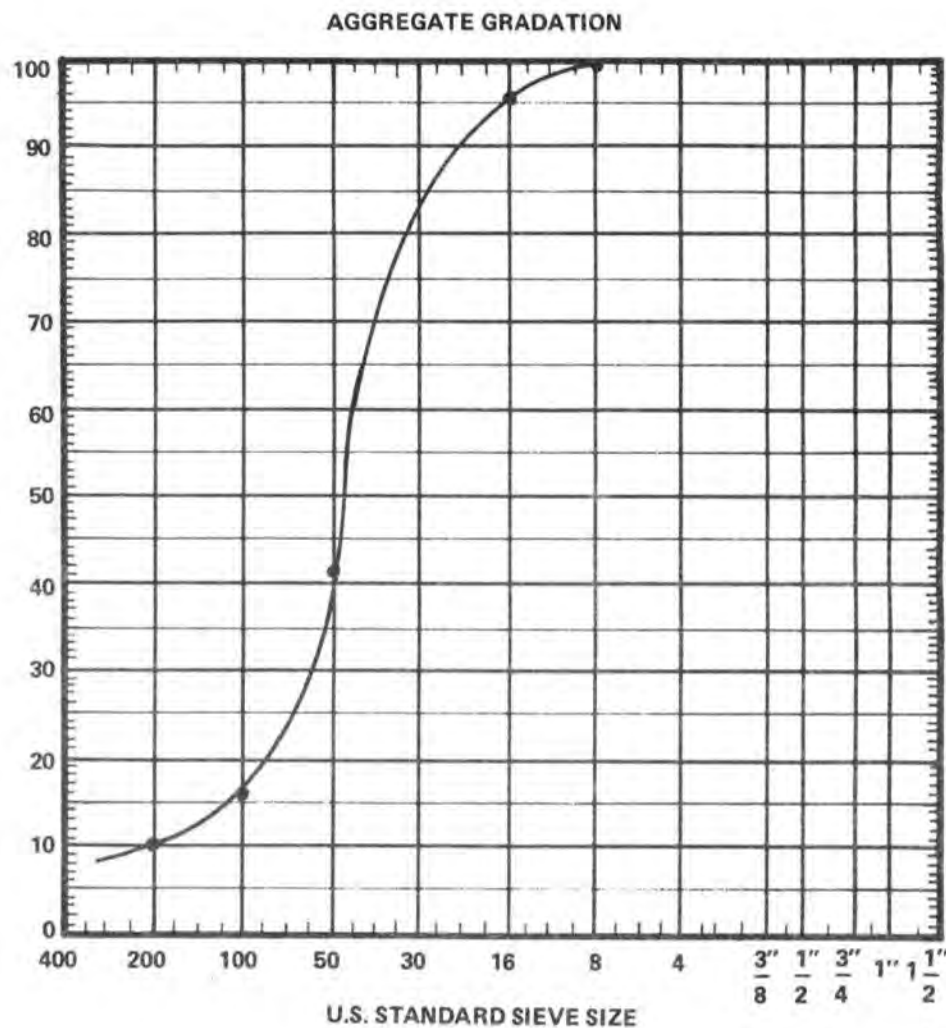
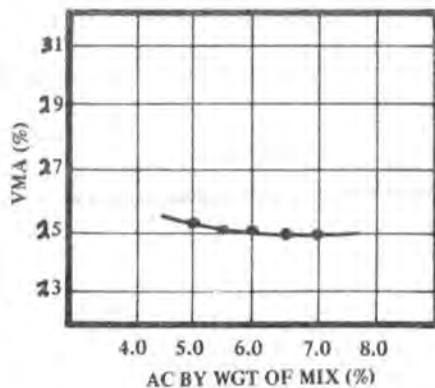
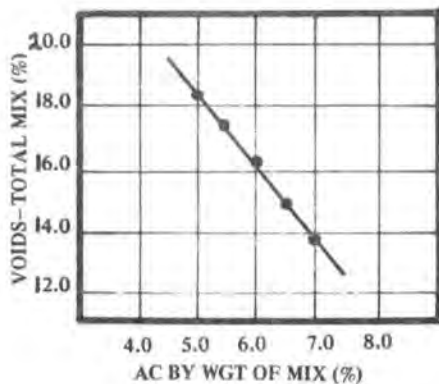
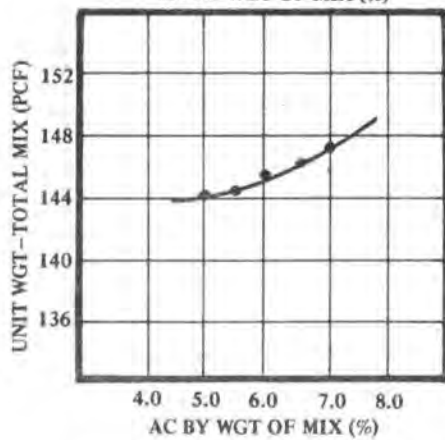
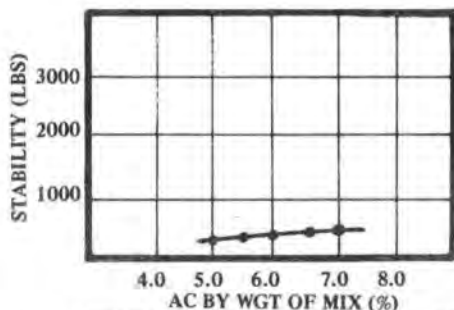
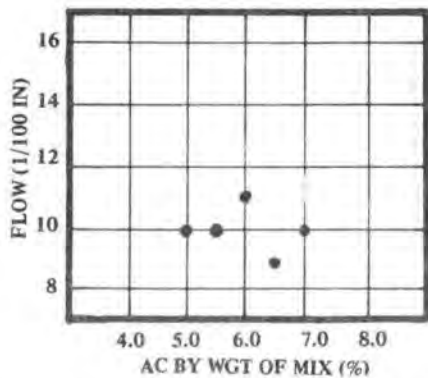


FIGURE 33. SAND-STONE BLEND MIX (16 - 84%) - FINLEY PIT SAND AND GRANITE GNEISS WITH 2097 lb. MARSHALL STABILITY

DESIGNATION: FPS I

SAND: 16%

STONE: 84%

OPTIMUM % AC: 6.5

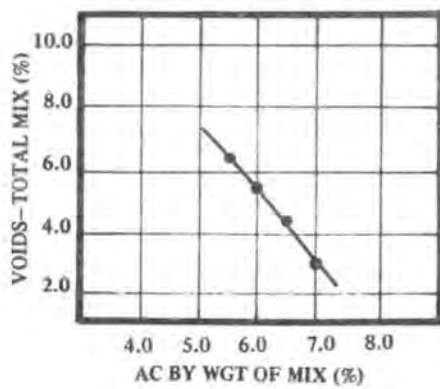
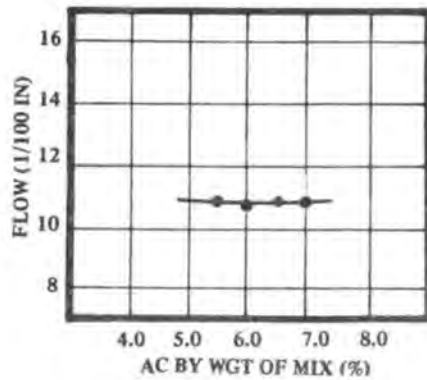
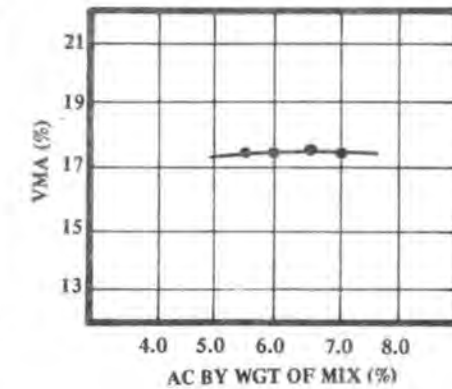
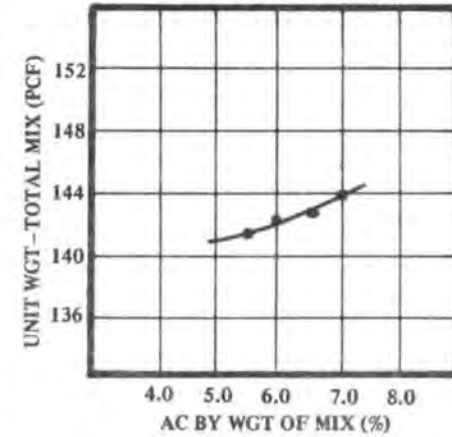
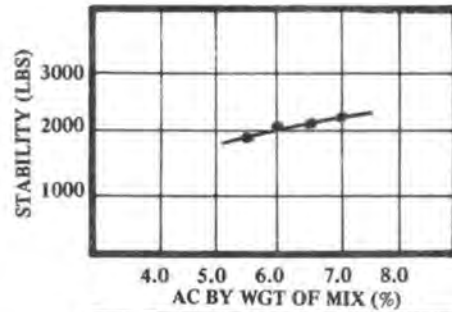
STABILITY (LBS.) 2097

FLOW (1/100 IN): 11.0

% VOIDS-TOTAL MIX: 4.5

% VMA 17.5

UNIT WGT-TOTAL MIX: 143.1



SOURCES

SAND: SAM FINLEY CO. PIT

STONE: NORCROSS GRANITIC GNEISS

AGGREGATE GRADATION

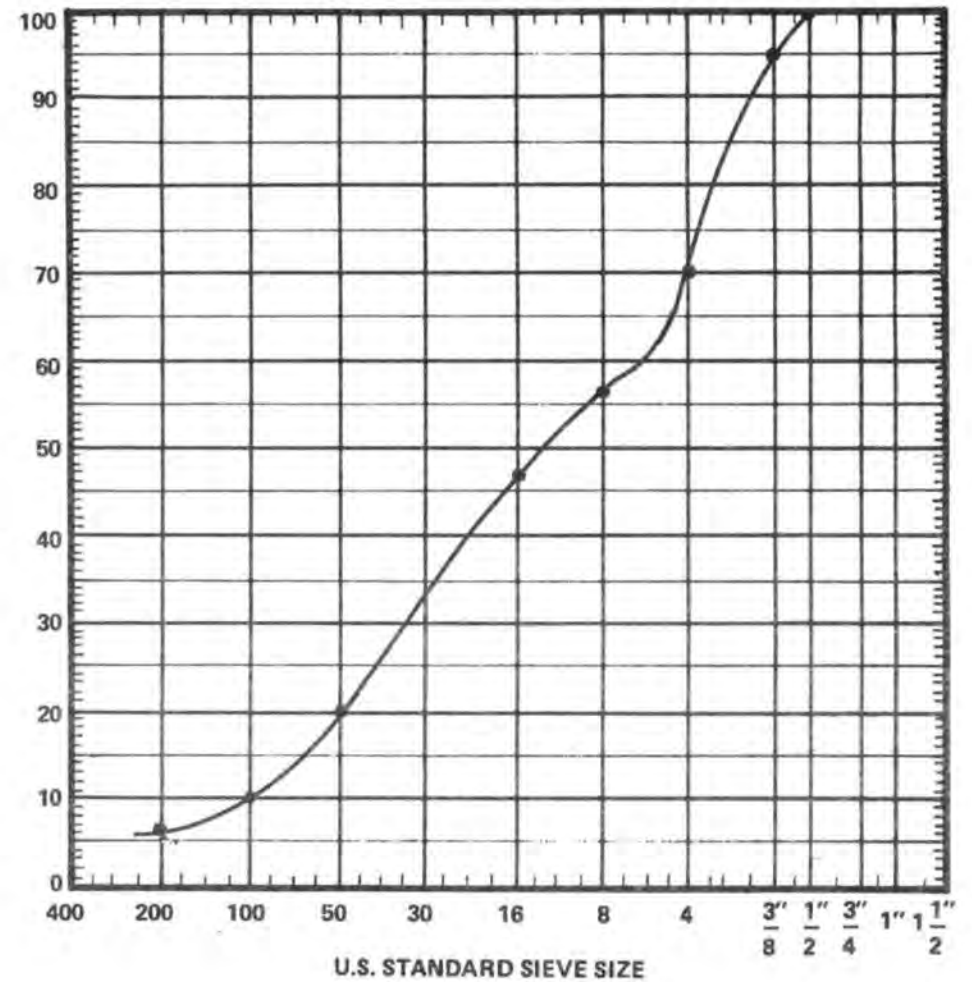


FIGURE 34. SAND-STONE BLEND MIX (70 - 30%) - FINLEY PIT SAND AND GRANITE GNEISS WITH 573 lb. MARSHALL STABILITY

DESIGNATION: FPS II

SAND: 70%

STONE: 30%

OPTIMUM % AC: 6.5

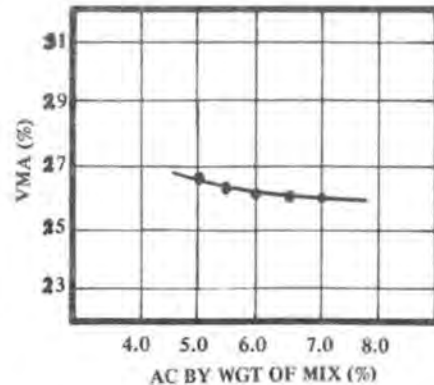
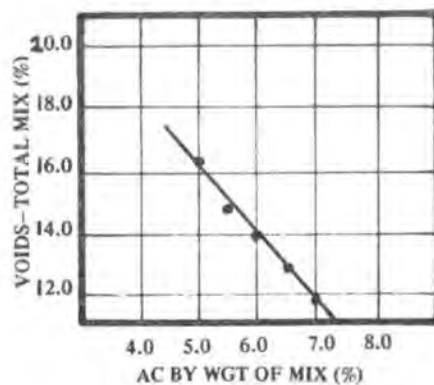
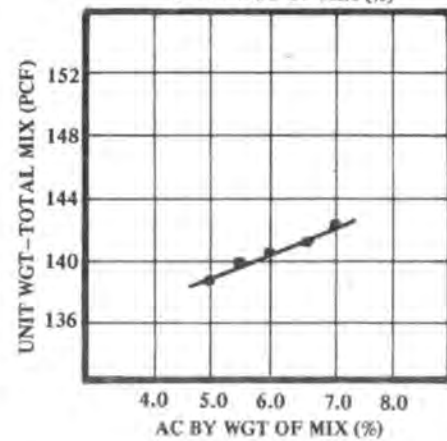
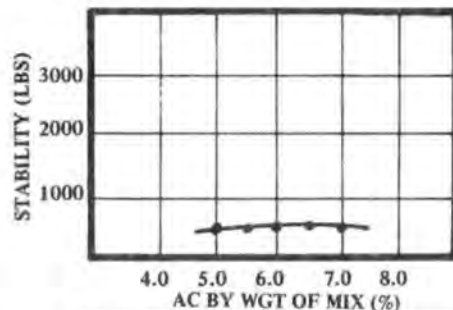
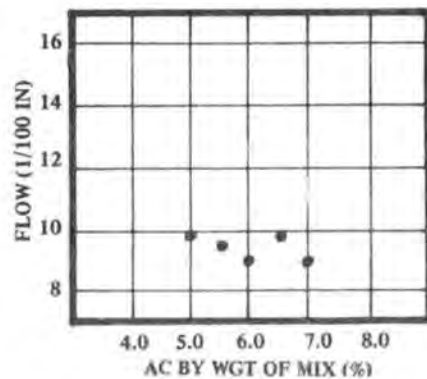
STABILITY (LBS.) : 573

FLOW (1/100 IN): 9.9

% VOIDS-TOTAL MIX: 13.0

% VMA : 26.3

UNIT WGT-TOTAL MIX: 131.2



SOURCES

SAND: SAM FINLEY CO. PIT

STONE: NORCROSS GRANITIC GNEISS

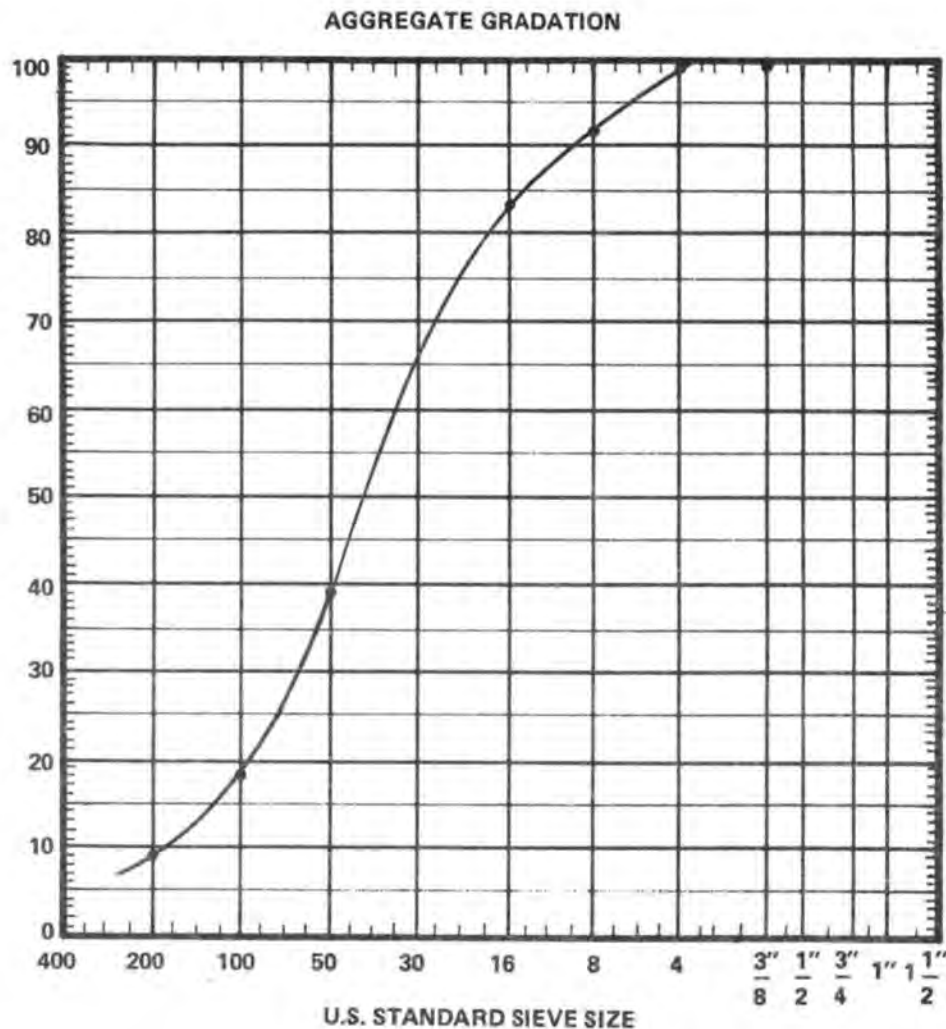


FIGURE 35. SAND-STONE BLEND MIX (40 - 60%) - FINLEY PIT SAND AND GRANITE GNEISS WITH 1527 lb. MARSHALL STABILITY

DESIGNATION: FA

SAND: 40%

STONE: 60%

SOURCES

SAND: SAM FINLEY CO. PIT

STONE: NORCROSS GRANITIC GNEISS

OPTIMUM % AC: 6.0

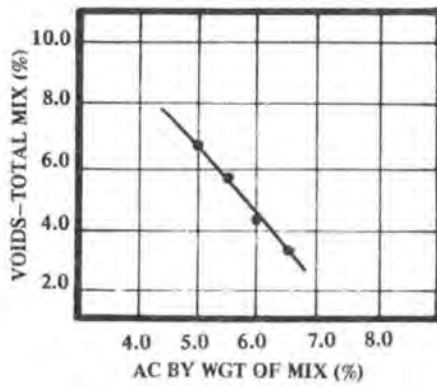
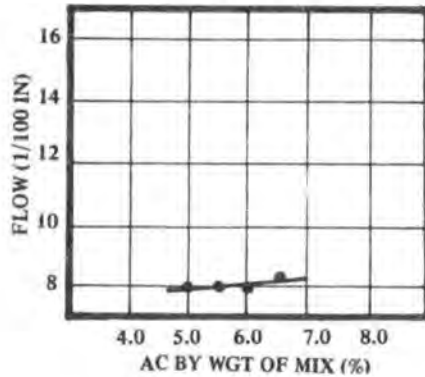
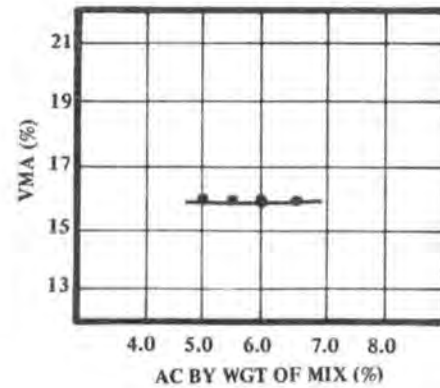
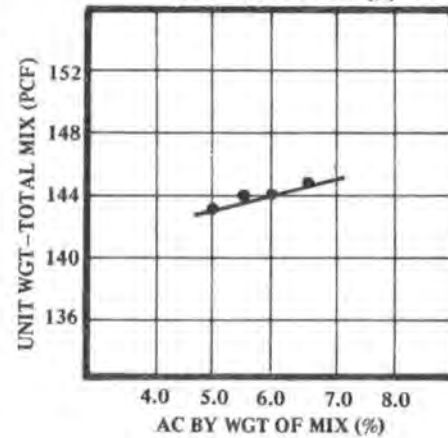
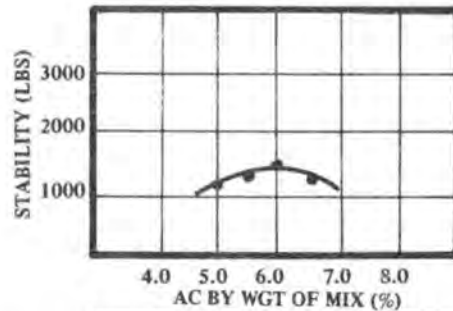
STABILITY (LBS.): 1527

FLOW (1/100 IN): 8.0

% VOIDS-TOTAL MIX: 4.4

% VMA: 16.1

UNIT WGT-TOTAL MIX: 144.3



AGGREGATE GRADATION

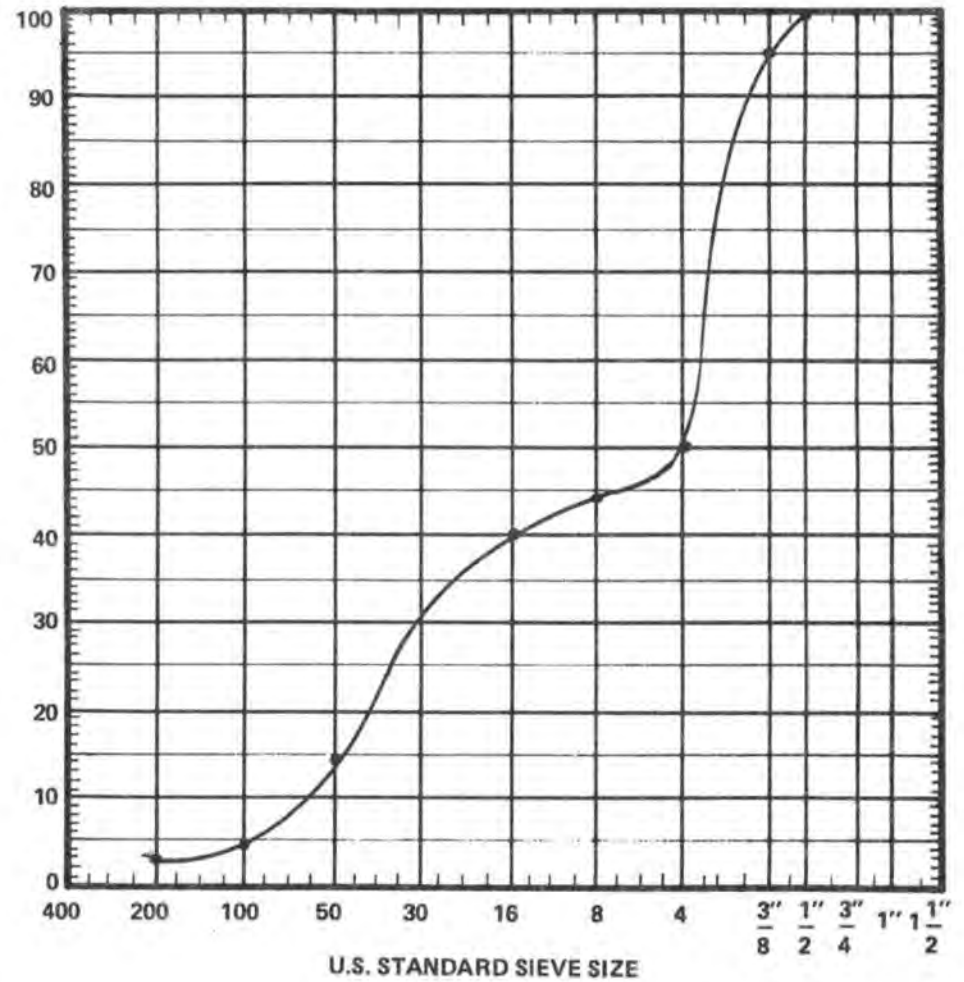


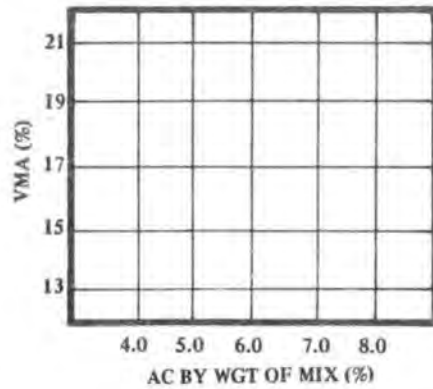
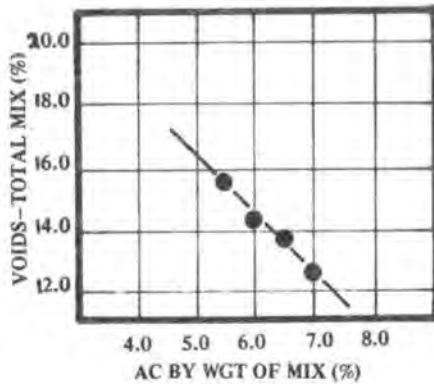
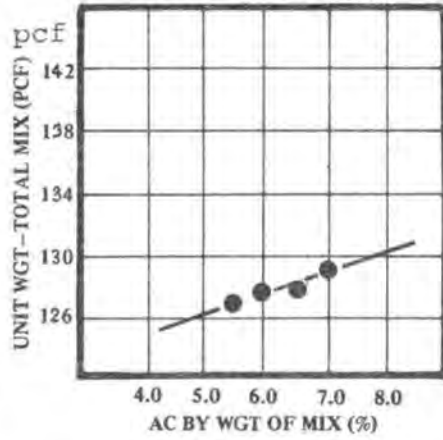
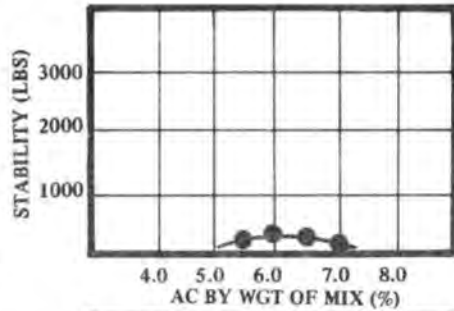
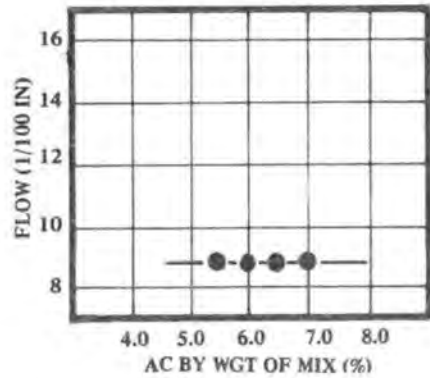
FIGURE 36. ALTAMAHA RIVER SAND MIX WITH 5% GRANITE GNEISS FINES - 308 lb. MARSHALL STABILITY

DESIGNATION: AF

SAND: 95%
 STONE: 0%
 FINES: 5%

OPTIMUM % AC: 6.0%
 STABILITY (LBS.): 308 lbs
 FLOW (1/100 IN): 8.7
 % VOIDS-TOTAL MIX: 14.2%
 % VMA

UNIT WGT-TOTAL MIX: 127.9



SOURCES

SAND: ALTAMAHA RIVER
 FINES: NORCROSS STONE

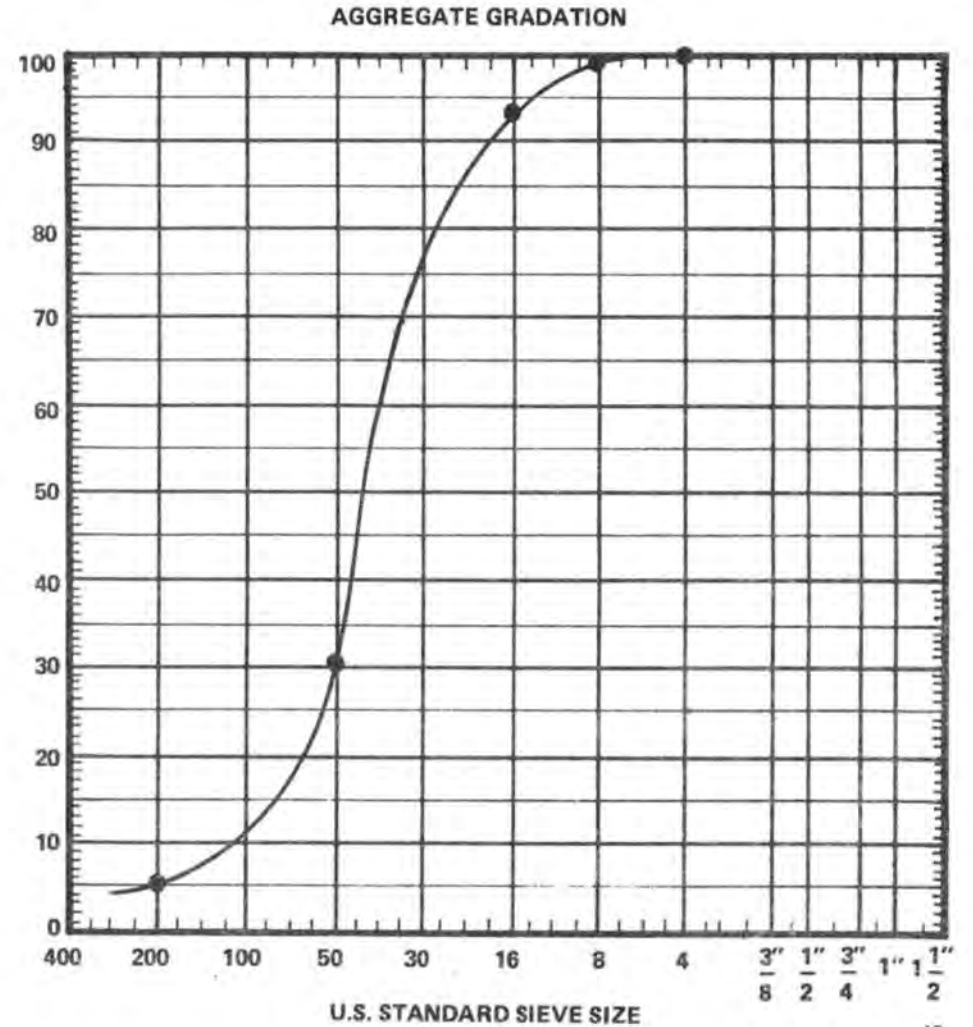


FIGURE 37. SAND-STONE BLEND (46.25 - 53.75%) - ALTAMAHA RIVER SAND AND GRANITE GNEISS WITH 1383 lb. MARSHALL STABILITY

DESIGNATION: AFS

SAND: 46.25%

STONE: 46.25%

FINES: 7.5%

OPTIMUM % AC: 7.0%

STABILITY (LBS.): 1383 lbs

FLOW (1/100 IN): 12.0

% VOIDS-TOTAL MIX: 3.3%

% VMA

UNIT WGT-TOTAL MIX: 145 pcf

SOURCES

SAND: ALTAMAHA RIVER

STONE: NORCROSS GRANITIC-GNEISS

FINES: NORCROSS STONE

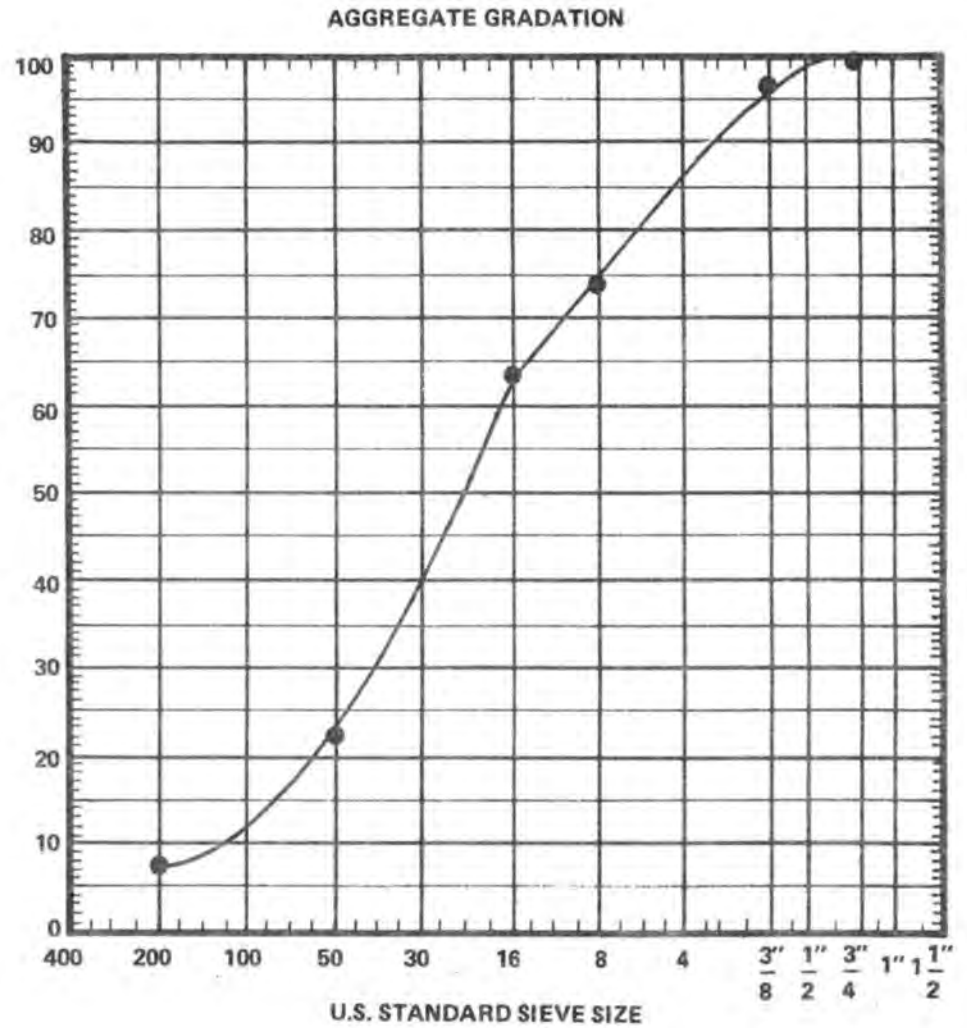
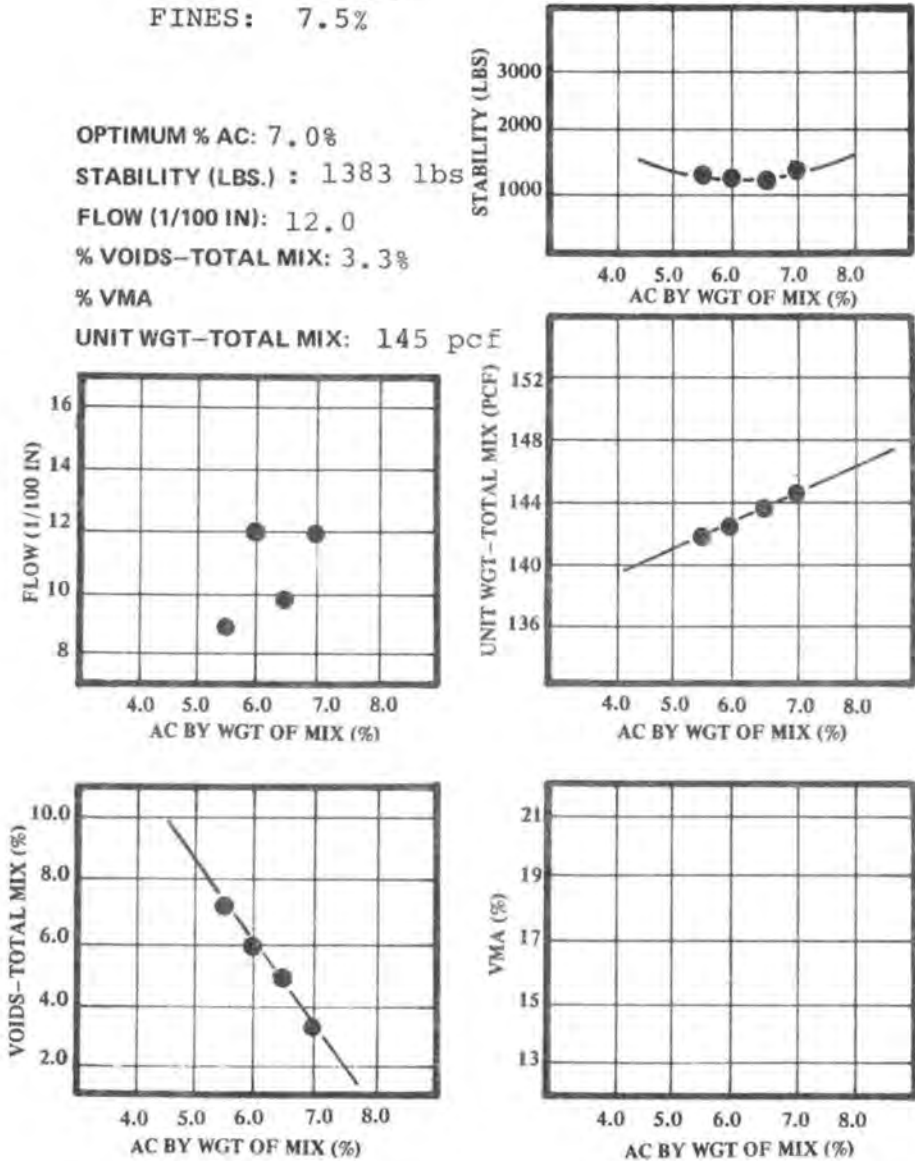


FIGURE 38. MARIANNA HIGH STABILITY SAND-ASPHALT MIX - 674 lb. MARSHALL STABILITY

DESIGNATION: HS
SAND: 100%

SOURCE: MARIANNA, FLORIDA

OPTIMUM % AC: 6.5%
STABILITY (LBS.) 674 lbs.
FLOW (1/100 IN):
% VOIDS-TOTAL MIX: 11.4%
% VMA
UNIT WGT-TOTAL MIX: 125.0 pcf

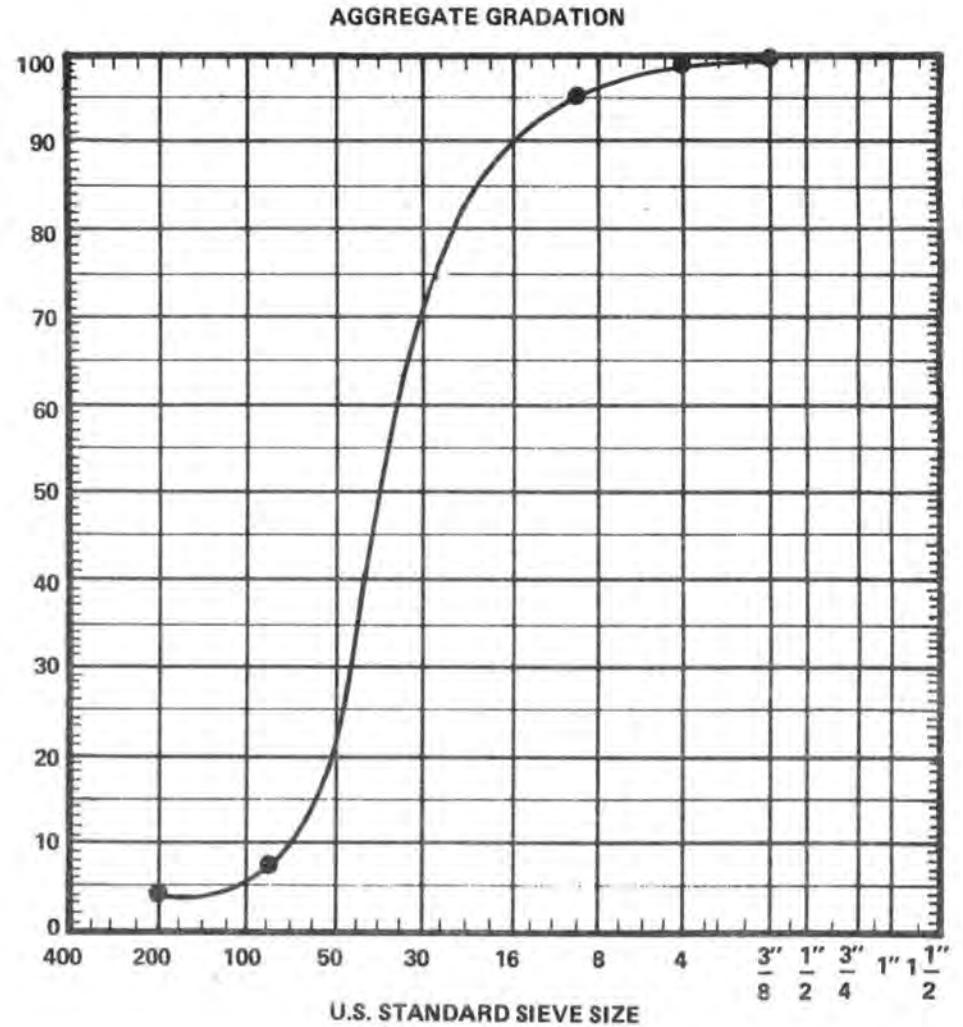
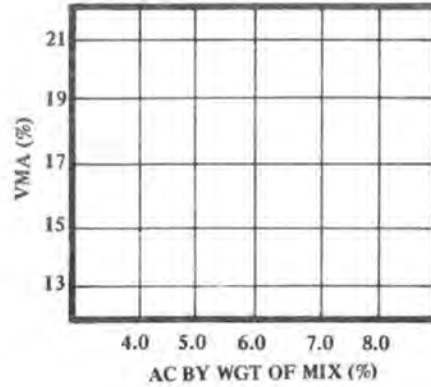
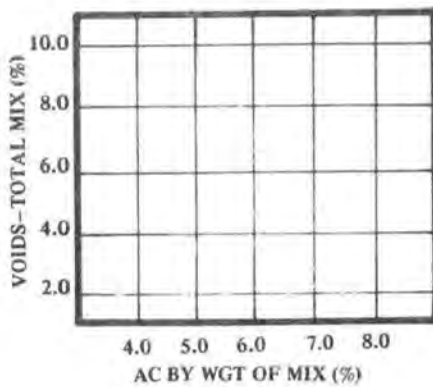
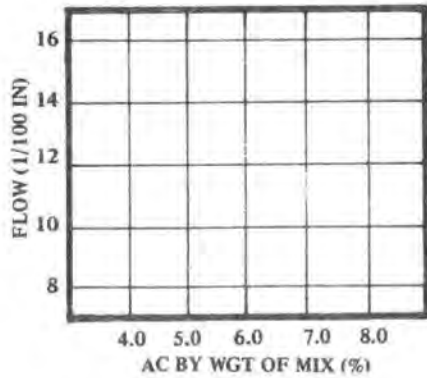
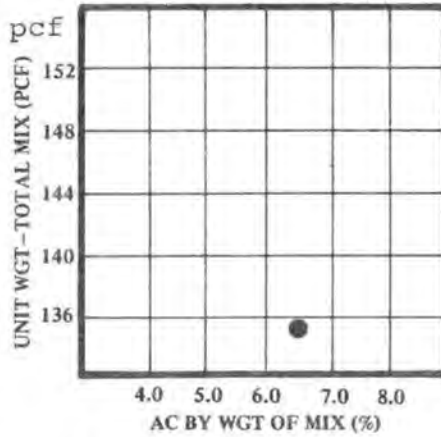
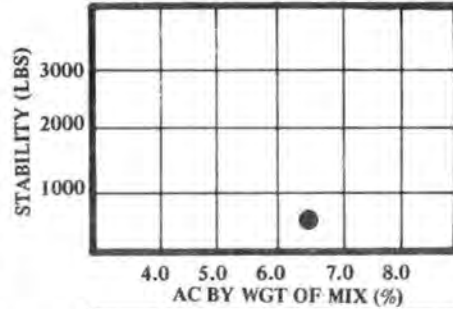


FIGURE 39. MARIANNA LOW STABILITY SAND-ASPHALT MIX - 566 lb. MARSHALL STABILITY

DESIGNATION: LS

SAND: 100%

SOURCE: MARIANNA, FLORIDA

OPTIMUM % AC: 7.0

STABILITY (LBS.): 566 lbs

FLOW (1/100 IN):

% VOIDS-TOTAL MIX: 16%

% VMA

UNIT WGT-TOTAL MIX: 124.4 pcf

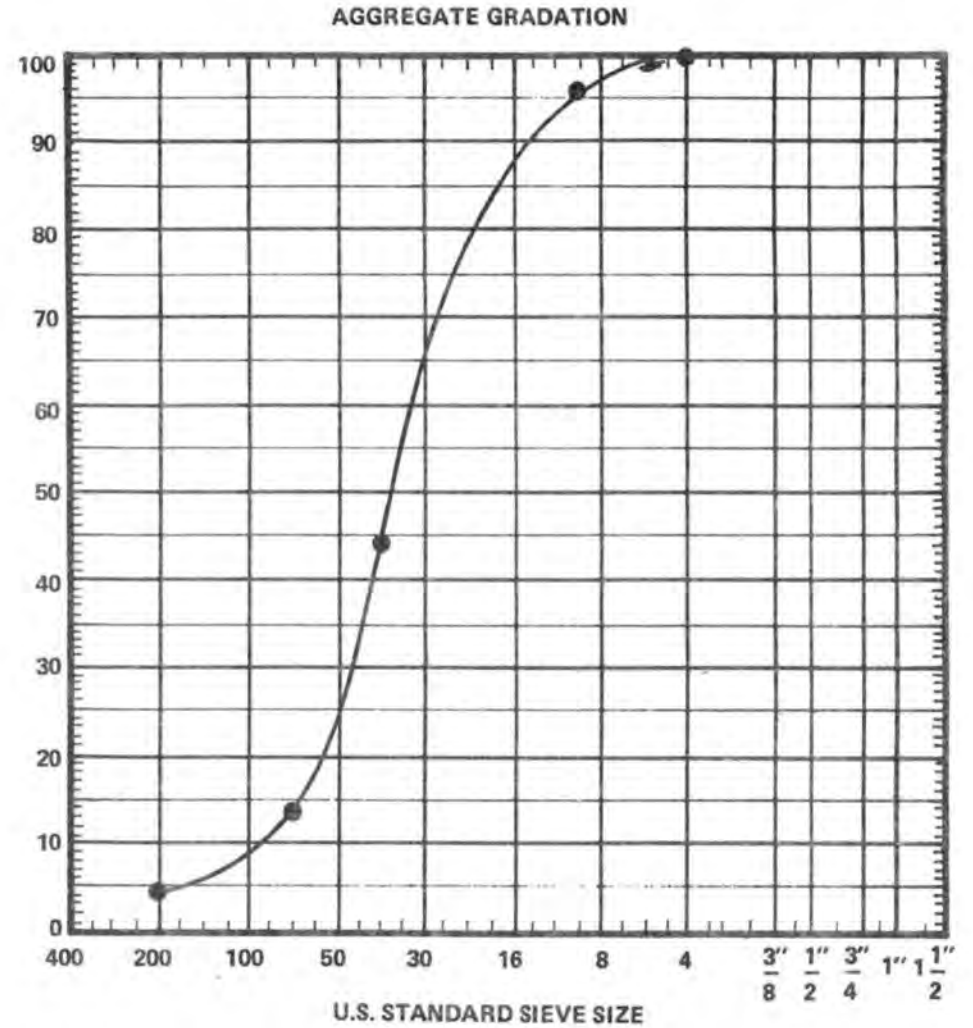
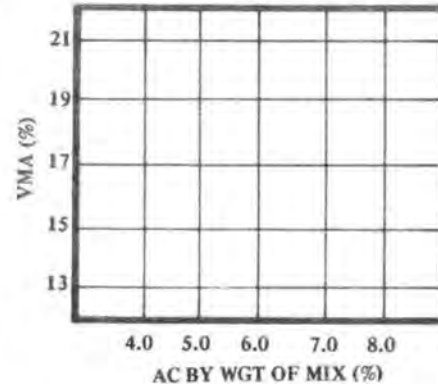
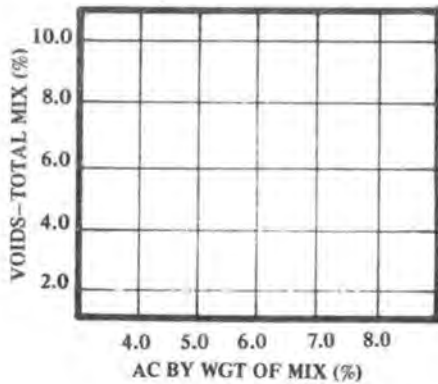
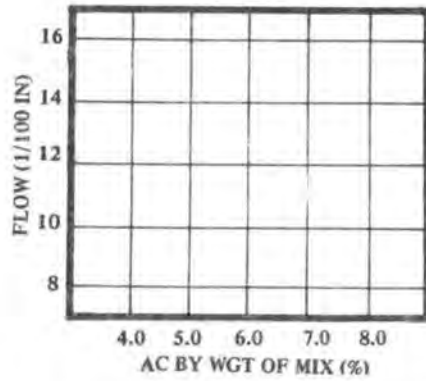
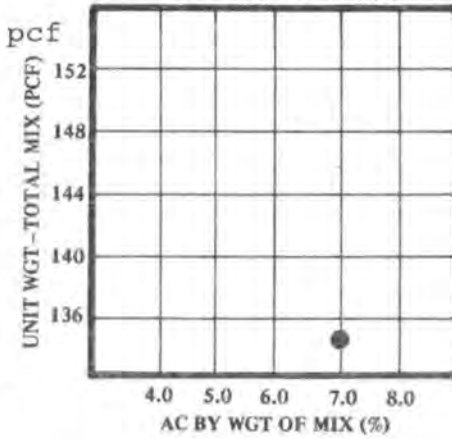
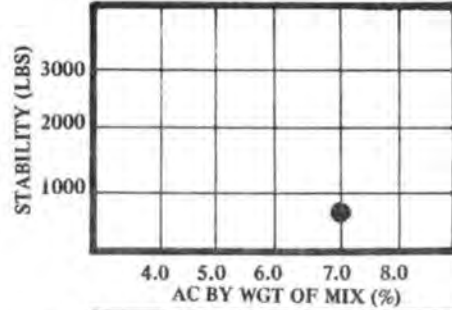
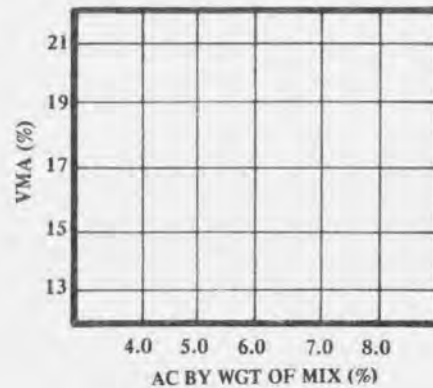
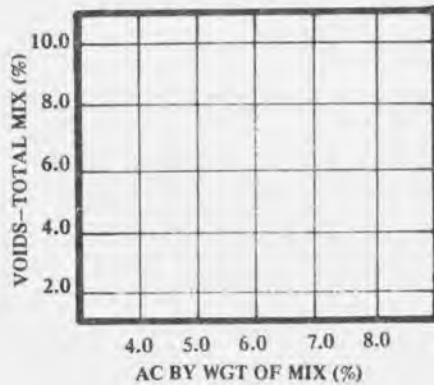
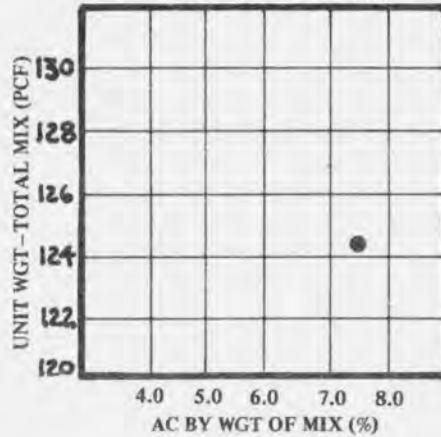
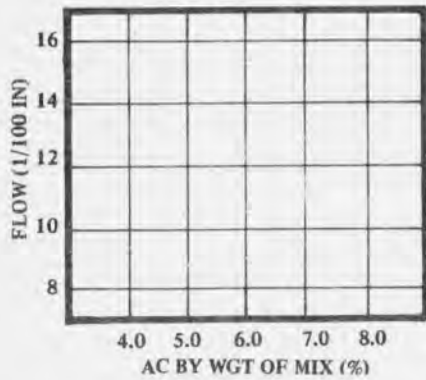
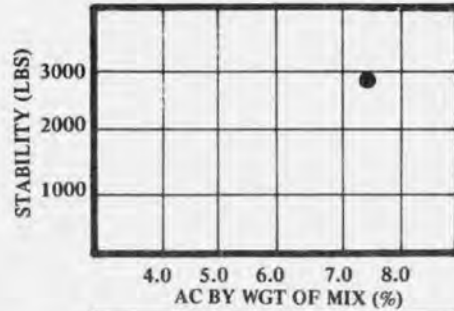


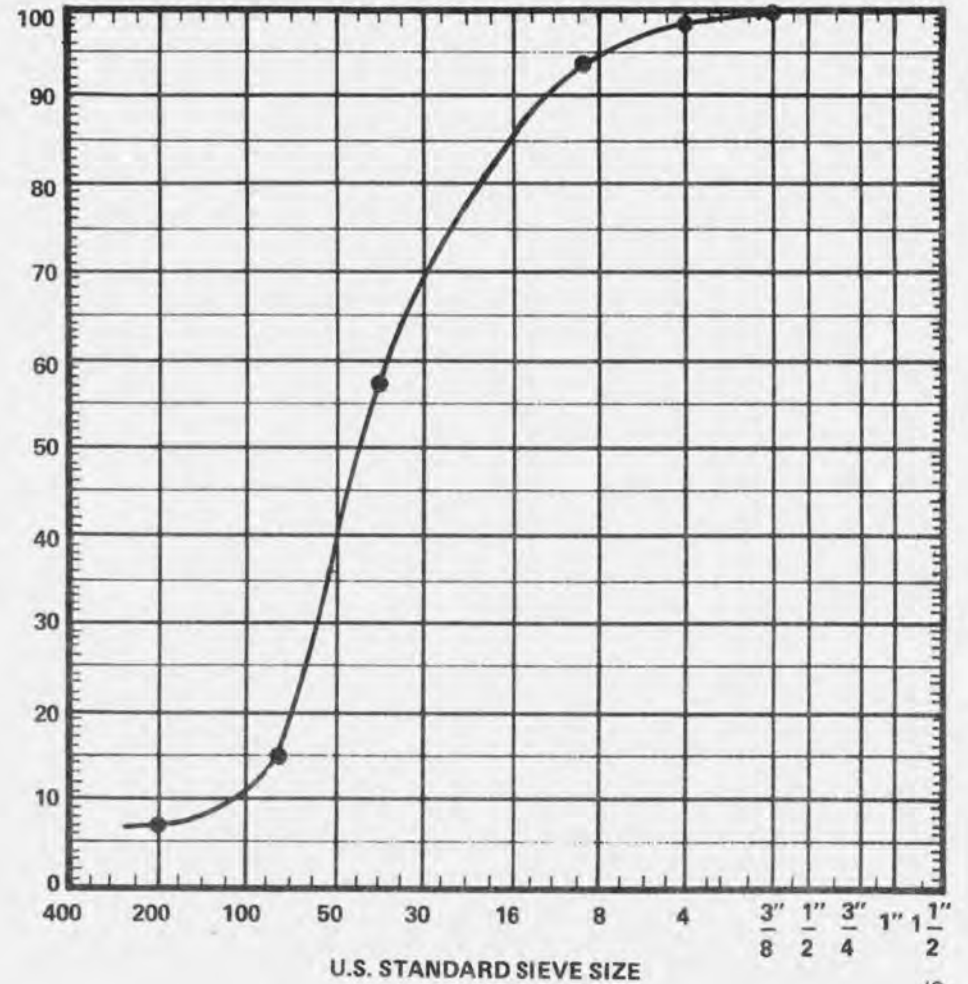
FIGURE 40. LAKE WALES SAND-ASPHALT MIX-BLOW SAND WITH A 1283 lb. HUBBARD FIELD STABILITY

STONE/SOURCE: LAKE WALES, FLORIDA

OPTIMUM % AC: 7.5%
 STABILITY (LBS.): 1283 lbs.



AGGREGATE GRADATION



used minimum unconfined compressive strength criteria of 250 to 300 psi (1,700 to 2,000 kN/m²). Fatigue tests were not performed on these blends.

Based on these test results, the following blends were chosen for fatigue testing: (1) pure sand with 8.5 percent cement, (2) 50 percent sand/50 percent stone with 5 percent stone fines and 6 percent cement, and (3) pure sand with 5 percent stone fines and 6 percent cement. To compare the performance of these blends with currently used blends, large quantities of sand and typical mix designs were obtained from the Sam Finely Co. sand pit at Bloomingdale, Ga.

Sand-Asphalt Mix Designs

Sand and sand-stone asphalt mix designs were prepared using the 50 blow Marshall Procedure by the Georgia Department of Transportation for a range of sand and sand-stone blends. Blends tested included natural Altamaha River sand with 10 percent fines, Altamaha River sand with 5 percent crushed stone fines, and sand-stone blends with 15, 40, 46 and 70 percent sand. Complete Marshall mix designs for each blend are presented in Figs. 32 through 39. The mix designs given in Figs. 32 through 35 were performed using sand from the Finley Co. pit. In most of the fatigue and rutting tests described subsequently, river sand was re-graded and substituted for Finley pit sand. Properties of AC-20 asphalt cement used in all fatigue and rutting tests are given in Table 22.

To relate the results of the laboratory tests with field experience, fatigue and rutting tests were performed on sands used in the sand-asphalt bases at the previously described Marianna and Lake Wales Test Roads in Florida. The Marshall Mix Design for these materials appears in Figs. 38, 39, and 40. The asphalt cement used in the original Marianna test road was refined from a Mexican crude oil but was not available at the time of this study. Therefore, an asphalt cement was chosen with properties similar to those used in the original test road. The asphalt cement used to prepare Lake Wales sand-asphalt base specimens was from the same South American source and had characteristics similar to those of the asphalt cement used in the test road. The specific properties of the asphalt cements used in the Marianna and Lake Wales tests were not determined.

Table 22. Physical Properties of the Asphalt Cement

| Asphalt Grade | AC 20 Trumbull (Atlanta) |
|-----------------------------------|--------------------------|
| Viscosity at 140°F at 275°F | 1761 358 |
| Penetration at 77°F | 68 |
| Specific Gravity at 60°F | 1.017 |
| Ring and Ball Softening Point | 124°F |

CHAPTER 6

FATIGUE AND RUTTING TESTS

Introduction

This study used repeated load testing to evaluate rutting and fatigue characteristics of nine sand and sand-stone asphalt base course mixes with varying asphalt contents. Fatigue characteristics of five different cement-stabilized sand and sand-stone blends were also evaluated. Rutting characteristics of the sand-asphalt mixes were determined using the repeated load triaxial test. In the repeated load triaxial test to evaluate rutting, a cylindrical specimen of sand-asphalt was subjected to a constant confining pressure, σ_3 , while 100,000 repetitions of axial deviator stress $\sigma_1 - \sigma_3$ was applied to the specimen. No rutting tests were performed on the cement-stabilized materials.

The fatigue test was performed by placing rectangular beam specimens of cement- or asphalt-stabilized sands and sand-stone blends on a rubber subgrade and applying a repeated load at the center. Fatigue test results were used to determine relative fatigue life of the mixes studied. Sand-asphalt specimens were tested between 7 and 21 days after preparation. Sand-cement specimens were tested after approximately 28 days of curing in a moisture room. In the fatigue test, a cyclic load was applied at the center of the beam. Fatigue test specimens were not subjected to stress reversals during testing.

A pneumatic loading system was used to apply the cyclic loading in both the rutting and fatigue tests. The cyclic load was applied by means of a Bellofram cylinder into which air was cycled by a 5-way spool valve. The movement of the spool valve was controlled by two solenoid pilot valves actuated by electrical signals from an electronic cyclic timer. The duration of load pulse and the rest time between pulses was controlled by the electronic timer. The load pulse used in the repeated load triaxial and fatigue tests had a duration of 0.06 sec., and the pulse had an approximately symmetrical haversine shape. The load pulse was applied 45 times per minute.

All electronic instrumentation was carefully calibrated to ensure accurate test results. The linear variable differential transducers (LVDT's) used to measure deflections were calibrated with a micrometer calibration device accurate to 0.0001 in. (0.0025 mm). The operation and calibration of the outside LVDT's were checked before each test using a steel calibration block 0.030 in. (0.76 mm) thick. The load cells were calibrated statically by applying a load of known weight and recording the output from the load cell. The SR-4 strain gauges were calibrated statically by axially deforming a cylindrical asphalt

concrete specimen that had a strain gauge glued along its longitudinal axis. The specimen deformation was measured with a dial indicator and average specimen strain was calculated and correlated with the strain measured using the SR-4 strain gauge.

Sample Preparation

The fatigue characteristics of the sand-cement and sand-asphalt mixes were evaluated using rectangular beam specimens. Rutting tests were performed on cylindrical specimens of the asphalt stabilized materials. All aggregates used in this investigation were sieved and the resulting sand or stone sizes stored separately. Each specimen was prepared by weighing the required material for each sieve size and carefully blending the sizes together.

All asphalt stabilized specimens were moulded using a kneading-type compactor. The kneading compactor produced laboratory specimens with structures (orientation of aggregate) similar to those developed in the field during the rolling operation. The load foot of the compactor was held at the proper temperature by an internal heating coil. The anti-stripping agent, ADDELIN, was added to the mix at a rate of 0.5 percent of the liquid asphalt by weight. The heated mould and mould base were placed on the kneading compactor. The moulds, aggregate and asphalt were placed in an oven and heated for approximately 2 1/2 hours at the prescribed temperatures. The aggregate, asphalt and anti-stripping agent were then weighed in the proper proportions and thoroughly mixed in a bowl to coat all the aggregate particles with asphalt cement. The asphalt concrete mixture then was placed immediately in the mould and compacted.

The aggregate, asphalt, and mould were heated in an oven to the temperature recommended by the Asphalt Institute [53] based on the viscosity of the asphalt cement. The mixing temperature for the specimens prepared using AC-20 asphalt cement was between 285°F and 295°F (140° and 146°C), and the compacting temperature was between 262°F and 278°F (128° and 137°C).

Cylindrical Asphalt Stabilized Specimens

The cylindrical specimens used in the repeated load triaxial and creep tests were 4 in. (100 mm) in diameter by 8 in. (200 mm) high. These specimens were compacted in a cylindrical steel mould using the kneading compactor. With the mould in place, the hot asphalt concrete mixture was spooned into the mould; as the mould was filled, the load foot was actuated so as to press down on the material in the mould one time between adding each spoonful of mixture. Filling the mould and compacting the specimen required approximately five minutes and took about 60 spoonfuls of material. This kneading action compacted the specimen to within 1/8 in. (3.2 mm) of the finished specimen height.

A circular piece of filter paper cut to fit the inside diameter of the mould was placed on top of the compacted specimen, and a loading head was positioned on top of the filter paper. The entire mould

assembly was immediately positioned in a testing machine. A static load was placed on the specimen to level the top and to finish compacting it to the specified height of 8 in. (200 mm). The specimen was loaded on the top and bottom by two floating pistons to minimize end effects. After cooling under the static load, the specimen was extruded from the mould and then measured and weighed. A china marker indicated the sample number and circumferential lines 2 in. (50 mm) from the ends (100 mm apart) later used for positioning the LVDT clamps on the specimen.

Asphalt Stabilized Beam Specimens

All beams were compacted and mixed at the same temperatures as those used in preparing the cylindrical specimens. After heating, the beam mould was placed in the kneading compactor on a sliding rack. Since the loading foot of the compactor did not move laterally, the beam mould was moved manually in the sliding rack during the compaction operation. The aggregate, asphalt cement and anti-stripping agent were mixed together in preweighed amounts to completely coat all aggregate particles with asphalt cement. The hot asphalt concrete mixture then was placed in the mould in four layers. Each layer was compacted by 3 to 4 passes of the compactor along the length of the beam. After all asphalt concrete was placed in the mould, a loading plate was positioned on top of the beam and loaded until a height of 3.0 in. (75 mm) was reached. This procedure also served to level the surface of the specimen.

The beam and mould were allowed to cool for approximately 2 1/2 hours, and the mould was removed. After cooling, each specimen was measured, weighed and stored on a surface ground steel plate. The specimens were kept on the perfectly flat surface so they would lie flat on the rubber pad used in the fatigue test. Machined steel plates were used for storage of the beams to avoid inducing tensile strains in the beam before testing and to give uniform subgrade support to the beam during the fatigue test. The specimen number, asphalt content, date of compaction and the future location of the loading foot were marked on each specimen.

A reference point for measuring the deflection of the centerline of the beam was then established by epoxy gluing a small aluminum tab on one side along the neutral axis at the midpoint of the beam. An SR-4 wire resistance strain gauge (BLH A9-4) was glued below the reference tab 0.1 in. (2.5 mm) above the bottom of the beam. The strain gauge was oriented parallel to the neutral axis to measure the maximum bending tensile strain in the beam. A strain gauge having a relatively long gauge length of 2.0 in. (50 mm) was used to minimize the effects of the large aggregate present in some of the base course mixes.

A fast-setting epoxy glue was used to bond both the aluminum tabs and strain gauges to the beams. The glue dried in approximately 5 minutes and later permitted relatively easy removal of the reference tab and strain gauge from the beam.

Soil-Cement Beam Specimens

The soil-cement beam specimens were prepared at the previously determined standard Proctor maximum dry density and optimum moisture content of the mix. The required aggregate, cement and water were weighed out and mixed by hand in a large bowl until a uniform blend was obtained. One-third of this mixture then was spooned into the steel mould, leveled and the mould cover placed over the surface. The specimen then was compacted by applying 25 blows of the standard Proctor hammer to the top of the cover. The cover was removed and the process repeated with the remaining two-thirds of the mixture. After approximately one-half hour, the specimen was removed from the mould and quickly transported to the moisture room where it was placed on a flat glass plate and allowed to cure for a period of 28 days. The specimens were removed from the moisture room and allowed to dry, weighed, and measured. The location of the loading foot, LVDT block and strain gauge were marked on the beam.

Repeated Load Triaxial Test

The repeated load triaxial test was used to compare rutting characteristics of the different sand and sand-stone asphalt mixes. Rutting tests were not performed on the sand-cement blends since rutting caused no problems in this material. A multi-layered elastic analysis of typical pavement structures in conjunction with the Z-function approach [54] was used to determine appropriate axial and confining stress states to be used in the rutting test. The rutting test specimens were subjected to 100,000 load repetitions, except in a few instances when the specimens failed. Only compressive stress states were used in the triaxial tests and most specimens were subjected to a confining pressure, σ_3 of 5 psi (34 kN/m²) and a deviator stress, $\sigma_1 - \sigma_3$ of 25 psi (170 kN/m²). A recent study indicated that approximately one-half of the pavement rutting in middle and southern Georgia occurred at pavement temperatures above 96°F (36°C), and one-half occurred below this temperature [54]. At two locations in Florida, the mean rutting temperature of the pavement was found to be approximately 93°F (34°C) [64]. Therefore, repeated load triaxial tests were performed at a standard test temperature of 95°F (35°C), which is approximately the mean pavement temperature for the rutting which occurred in southern Georgia and central and northwestern Florida.

Specimens 4 in. (100 mm) in diameter were tested in a 6 in. (150 mm) diameter triaxial cell enclosed in a controlled environmental chamber (Fig. 41). Axial strain was measured by placing two clamps on the specimens as illustrated in Fig. 42b and measuring the movement between these clamps using two small AC LVDT's. The LVDT's used to measure the axial deformation were wired together to give the average specimen movement between clamps. This instrumentation arrangement minimizes the effects of possible tilting of the specimen. Radial strain was measured by a single LVDT oriented horizontally in the plane of the diameter of the specimen at the open end of the lower clamp (Fig. 41). This LVDT measured a deformation directly related to the change in diameter of the specimen. The two clamps were placed one-quarter of the distance from each end of the specimen to reduce end effects. To minimize the size and weight of the measurement devices attached to the clamps, 0.375 in. (9.5 mm)

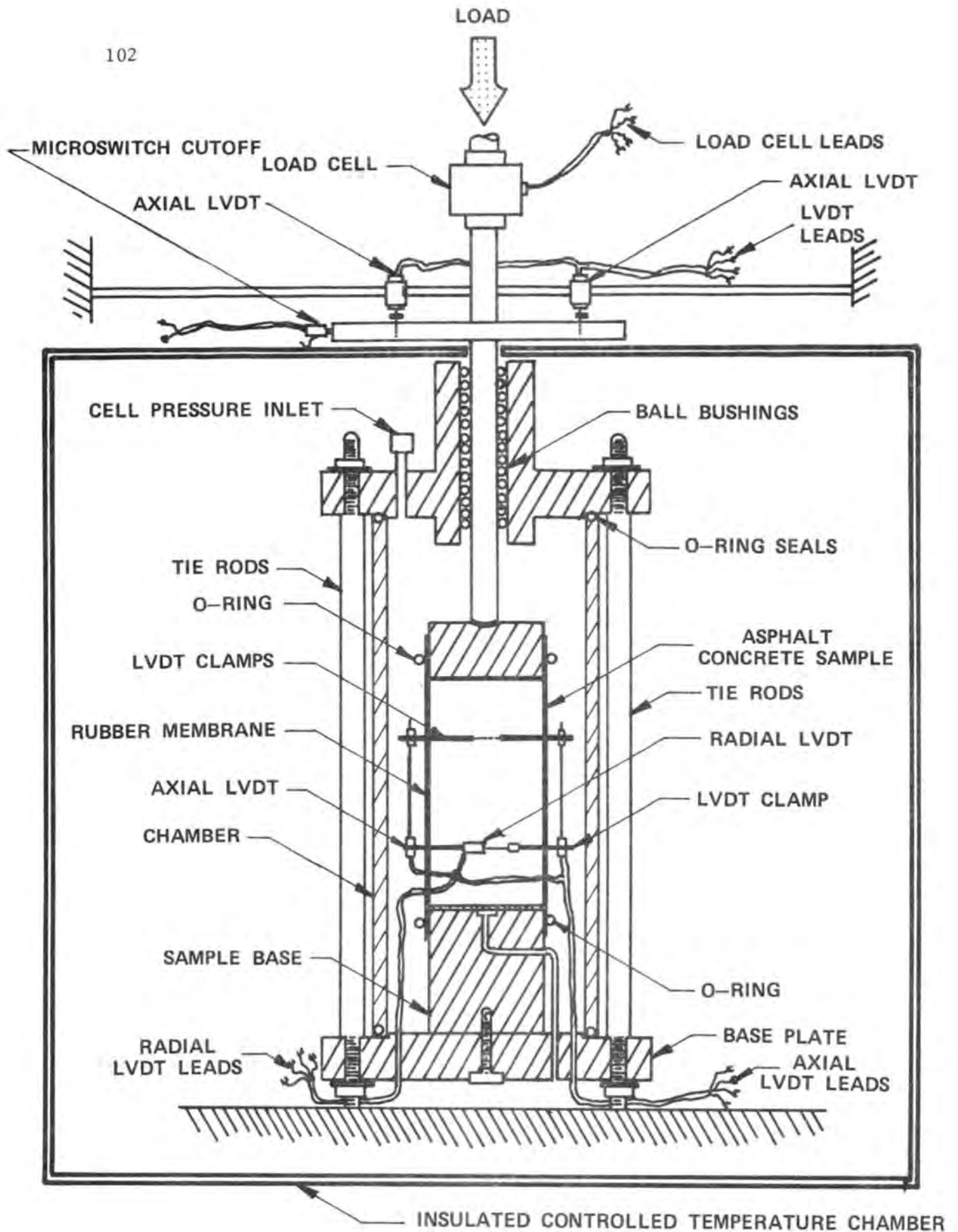


FIGURE 41. CONTROLLED TEMPERATURE REPEATED LOAD TRIAXIAL SYSTEM FOR DYNAMIC TESTING OF ASPHALT CONCRETE

diameter AC type LVDT's were used. The outputs from the inside axial and radial transducer measuring systems were recorded on a Hewlett-Packard two channel, strip chart recorder.

The total axial specimen deformation was measured by a pair of DC LVDT's which reacted against a Lucite clamp attached to the loading piston outside the environmental chamber (Fig. 41). The output from these transducers was recorded on a Hewlett-Packard X-Y recorder. Load was measured by a 2,500 lb. (11 kN) capacity load cell and recorded on a two-channel strip chart recorder.

The test procedure used for the repeated load triaxial rutting test is summarized as follows: Each sand-asphalt specimen was first carefully examined to assure that it was free from defects such as excessive voids and that both ends were flat and parallel. A rubber membrane was then placed around the sides of the specimen. The specimen was positioned on top of a bronze porous stone resting on the bottom loading platen of the triaxial cell. Top end friction on the specimen was minimized by rubbing a silicone lubricant over the top of the specimen. A thin Teflon pad was placed between the end of the specimen and the top platen. The rubber membrane was then pulled up over the top platen, and rubber O-rings were used to seal the membrane to the top and bottom platens.

The inside LVDT clamps were placed around the rubber membrane, and the LVDT probes were set at approximately the null voltage output position. The clamps were placed on the specimen tightly enough to prevent slippage during the test, but not so tight as to exert excessive additional confining stress on the specimen. Once the LVDT's were in place and adjusted, the triaxial chamber was assembled. The environmental chamber was then placed around the cell and the loading piston inspected to assure alignment with the top platen on the specimen. The top cross-arm of the loading system was lowered so that a small seating load was applied to the specimen. The triaxial chamber and enclosed specimen were maintained at the desired 95°F (35°C) testing temperature overnight in order for the specimen to reach the desired temperature.

The pens on the recorder monitoring the LVDT deflections were centered and the test started. Specimen deformation was measured continuously for the first ten repetitions, and then for a short time at approximately 100, 1,000, 10,000, 50,000, and 100,000 load repetitions. After approximately 100,000 repetitions, the test was terminated. During the test, chamber temperature, cell pressure, pilot valve pressure, deviator stress and load pulse time were observed periodically to insure proper adjustment.

The resilient modulus, plastic (permanent) strain and Poisson's ratio as a function of the number of load repetitions were obtained from this test.

Fatigue Test

A rectangular beam fatigue test was used to evaluate the fatigue characteristics of the asphalt and portland cement stabilized base mixes.

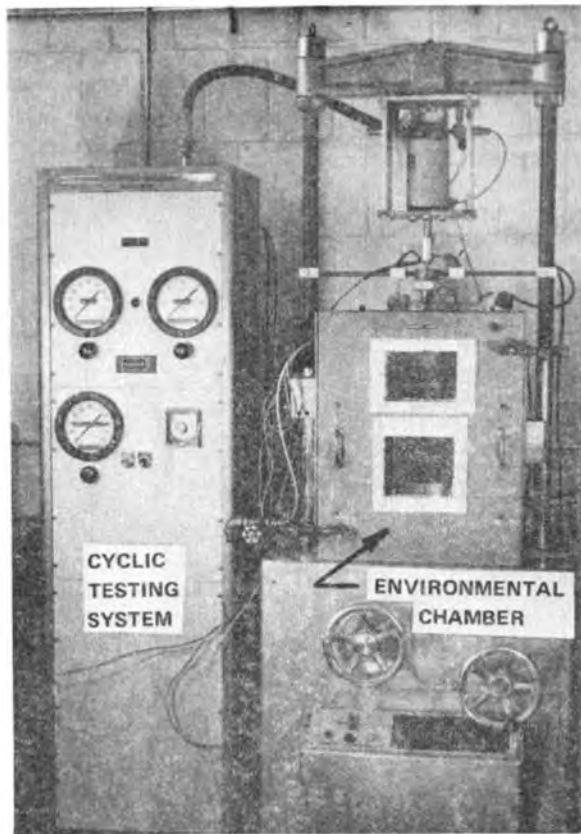
In this study, an asphalt concrete beam placed on a rubber subgrade was used to simulate field support conditions. This test also eliminated the problem of beam weight which has been noted to affect the results of unsupported beam fatigue tests.

The fatigue test equipment (Fig. 43) consisted of a load frame, a 4 in. (100 mm) thick rubber pad supporting the beam, and a pneumatic loading system. The fatigue specimen and rubber support were enclosed within an environmental chamber maintained at $80^{\circ}\text{F} \pm 1^{\circ}\text{F}$ (27°C). The rubber pad used in testing the sand-cement specimens had a Durometer reading of 40 and a modulus of subgrade reaction of 284 pci (7,860 gm/cc). Sand-asphalt specimens tested using the same 284 pci (7,860 gm/cc) rubber subgrade were found to partially fail due to the loading plate punching into the sand-asphalt beam. To prevent this type failure, a softer rubber subgrade (215 pci) was used on the surface ground base of the load frame.

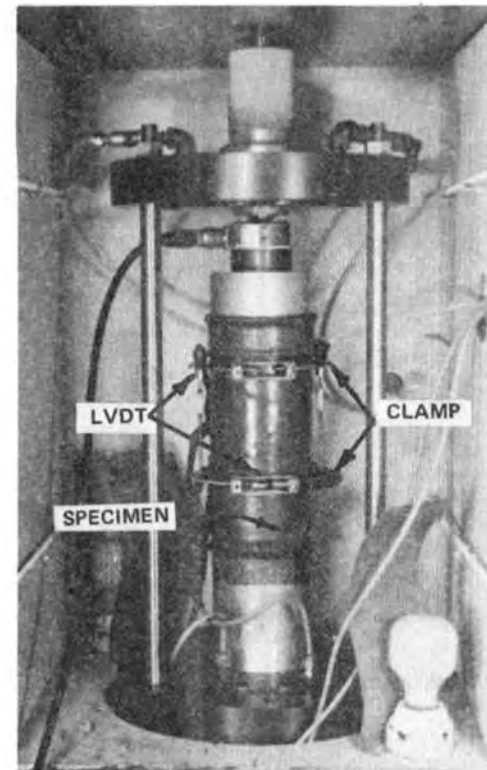
The load was applied to the center of the beam using a rigid steel plate which was 1.25 in. (32 mm) wide for the sand-cement specimens and 1.5 in. (38 mm) wide for the sand-asphalt specimens. The steel loading plate was 3 in. (76 mm) long and 1 in. (25 mm) thick. An SR-4 strain gauge was glued with EPOXY-5 cement to the side of each beam in a direction parallel to its axis 0.10 in. (2.5 mm) above the bottom. The strain gauge was located symmetrically below the center of the load. Another SR-4 strain gauge was glued to a dummy block of similar material placed inside the environmental chamber. This temperature-compensating strain gauge was used to eliminate any errors in measuring the strain in the beam that could be caused by small changes in temperature occurring in the chamber.

The deflection of the center of the beam was measured with a single DC LVDT and recorded on a Sanborn, two channel strip chart recorder. The constant load applied to the top of the beam was measured using a 2,500 lb. (11 kN) capacity load cell and recorded on a Sanborn, two channel strip chart recorder. To determine the number of load repetitions to failure, an automatic timing system was developed to measure at one-hour time intervals the centerline deflection of the beam. This system consisted of a mechanical cyclic timer wired into the paper drive motor on the Sanborn recorder and also to the 110-volt AC power cord leading to the 24-volt DC power supply for the LVDT.

Before each fatigue test was begun, the rubber pad was removed and the machined surface of the load frame which supported the pad was thoroughly cleaned to ensure continuous contact at the interface. After cleaning, the pad was replaced and a beam specimen was carefully centered on the rubber pad. The strain gauge leads were then soldered to the connections on the strain gauge and the system checked for continuity. The load foot was placed inside the previously made reference marks on the top of the beam. An LVDT with the probe in place was positioned against the aluminum tab glued to the side of the beam and was vertically aligned. The deflection under the load at the center of the beam and the radial tensile strain were measured at 100, 200, 300, 500, 750, and 1,000 load repetitions. The tensile strain measured at 1,000 load repetitions and the number of repetitions to failure was used in interpreting the

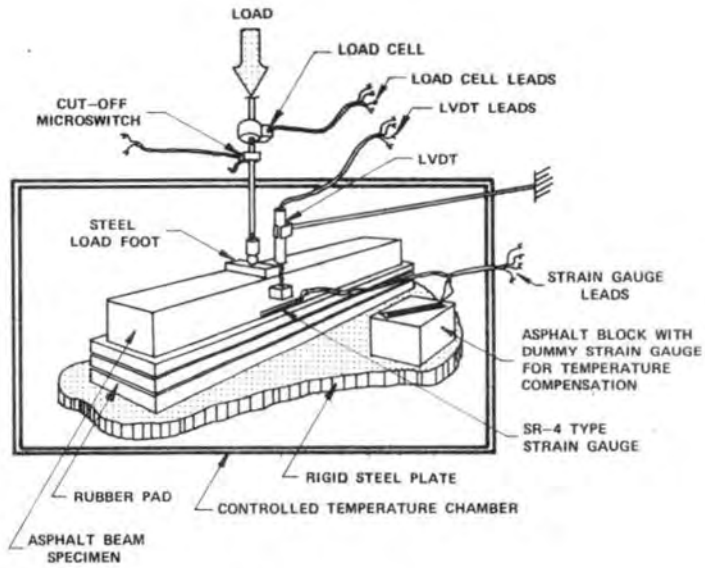


(a) General Testing System

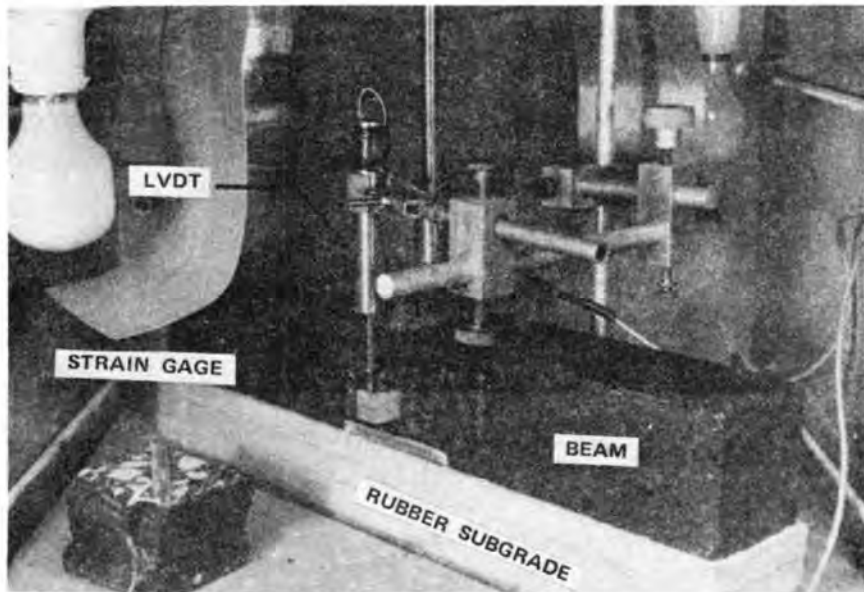


(b) Specimen and LVDT Clamps

FIGURE 42. REPEATED LOAD TRIAXIAL TESTING APPARATUS



(a) Schematic



(b) Close-Up Photograph

FIGURE 43. BEAM FATIGUE TEST APPARATUS

test results. The strain at 1,000 load repetitions was used to allow stabilization of the response of the beam to occur. After 1,000 repetitions, the strain gauge and epoxy glue were removed from the side of the beam to eliminate any strengthening effect. After this, the automatic deflection measuring system was used to obtain the centerline beam deflection at one-hour intervals until failure. A cutoff switch located on the loading piston was installed to automatically stop the test upon failure of the beam. Cracking of the beam was generally initiated in the bottom directly beneath the load. This crack rapidly propagated upward, and upon failure the beam would usually separate into two parts. Failure of the beam was defined by a relatively sudden large increase in centerline deflection of the beam as indicated by the automatically recorded beam deflections. Actual observations of the failure of several beams indicated the approach used was sufficiently accurate for establishing the number of repetitions to failure. Information from the strip chart recorders and calibration constants for the various electronic measuring devices were used to calculate radial tensile strain in the bottom of the beam, applied load, and tensile bending modulus of the asphalt concrete. A complete description of the procedure used to calculate the bending modulus has been given elsewhere [54].

CHAPTER 7

FATIGUE AND RUTTING TEST RESULTS

Sand-Asphalt

Introduction

Fatigue and rutting tests were performed on a wide range of sand-asphalt and sand-stone base course mixes. The materials used in these blends were described in Chapters 4 and 5 and the test procedures and equipment in Chapter 6. Bituminous base materials tested included both pure sand-asphalt mixes and also sand-stone blends having stone contents varying from 30 to 84 percent. The asphalt cement content varied from 5 to 7 percent and an AC-20 viscosity grade asphalt was used in all the tests.

The fatigue test consisted of applying a cyclic load at the center of a supported rectangular beam specimen until failure occurred. A constant temperature of 80°F (27°C) was maintained throughout the test. A 0.06 sec. load pulse time was used in both the fatigue and rutting tests. The repeated load triaxial test used to evaluate the plastic (rutting) properties of the mix consisted of subjecting a cylindrical specimen to 100,000 repetitions of axial load. The specimen was subjected throughout the test to a constant confining pressure. The mean pavement temperature for rutting of typical pavement sections in the coastal plains of Georgia and central and northwest Florida varied from 93°F to 96°F (34° to 36°C). Therefore, a standard testing temperature of 95°F (35°C) for this study was selected. Standard stress conditions for the repeated load triaxial test adopted to evaluate the rutting properties of the sand-asphalt and sand-stone base mixes consisted of an axial repeated deviator stress of 25 psi (170 kN/m²) and a confining pressure of 5 psi (34 kN/m²).

Fatigue Test Results

Recent research has shown that, generally, in terms of strain, fatigue curves cannot be directly compared [6,54]. The constant load method of interpreting the fatigue test results, therefore, was used in this study [6]. The load method of interpretation consisted of comparing for a constant applied load (of equal magnitude for each test), the number of repetitions required to cause failure of different stabilized mixes. The load method gave a reliable comparison when the fatigue test simulated support and loading field conditions with reasonable accuracy [6]. The load method required neither determination of the bending modulus of the stabilized layer under consideration nor the theoretical calculation of tensile strain in the layer. As a result, the load method of interpretation of fatigue test results gave a straightforward, direct comparison of relative fatigue performance.

On the other hand, interpretation of fatigue test results that used the elastic theory approach [6] required laboratory measurements of the relationship between tensile strain and number of repetitions to failure and also the relationship between bending modulus and repetitions to failure. For a given pavement structure, an appropriate layered theory then was used to calculate the theoretical tensile strain that should have existed in the stabilized layer under consideration. Measured bending modulus of the stabilized material was essential for use in the layered analysis. The number of repetitions to failure then was estimated from the laboratory fatigue curve that used the calculated tensile strain from layered theory. The layered theory approach, therefore, required use of layered theory, which involved a number of idealized assumptions, and measured bending modulus, which tended to be relatively difficult to accurately evaluate. For these reasons, the load method of interpretation was favored for evaluation of relative fatigue life of mixes and was used in this investigation. Further studies might, however, show the elastic theory approach to give better comparisons of in-situ fatigue performance. The more flexible layered theory approach can be used to compare the performance of different structural sections.

The fatigue tests were performed using a rectangular beam specimen supported on a rubber subgrade having a modulus of subgrade reaction of 215 pci (5950 gm/cc). Since the rubber subgrade used in these tests was softer than that used in a previous investigation of asphalt concrete base mixes [54], the test results cannot be directly compared using the constant load method. A softer subgrade was used for the fatigue tests on the sand-asphalt and sand-stone mixes to prevent the centrally applied load from punching into the surface of the beam specimens which were softer than the asphalt concrete base course mixes previously studied.

Test Results. Larger asphalt contents and lower air voids in the sand and sand-stone blend mixes were found to increase significantly fatigue life as illustrated in Fig. 44. The sand-stone percentages together with the material designation for each mix is shown by the corresponding data points on the figure; the 50 blow Marshall stability of the mix in pounds is given in parentheses by the data points. The data shown in Figs. 44 and 45 are the average of all tests performed on a given mix at the same air void and asphalt contents.

Close examination of the data given in Fig. 44 shows results for all sand-stone base mixes tested and indicates a general trend between an increase in fatigue life and an increase in Marshall stability. Figure 45 better illustrates the general trend and shows the relationship between 50 blow Marshall stability and fatigue life, but without considering other important variables such as asphalt content. This figure shows all individual data points from each test. A better relationship between fatigue life and stability is obtained if the influence of asphalt content variations and Marshall stability are considered for all sand-asphalt and sand-stone blends as illustrated in Fig. 46. Increased fatigue life with increasing Marshall stability and asphalt content occurred for Marshall stabilities between 200 and 2,000 lbs. (900 to 9,000 kN) which was the range of stabilities tested. A similar increase in fatigue life with both stability and asphalt content was found for just the sand-asphalt

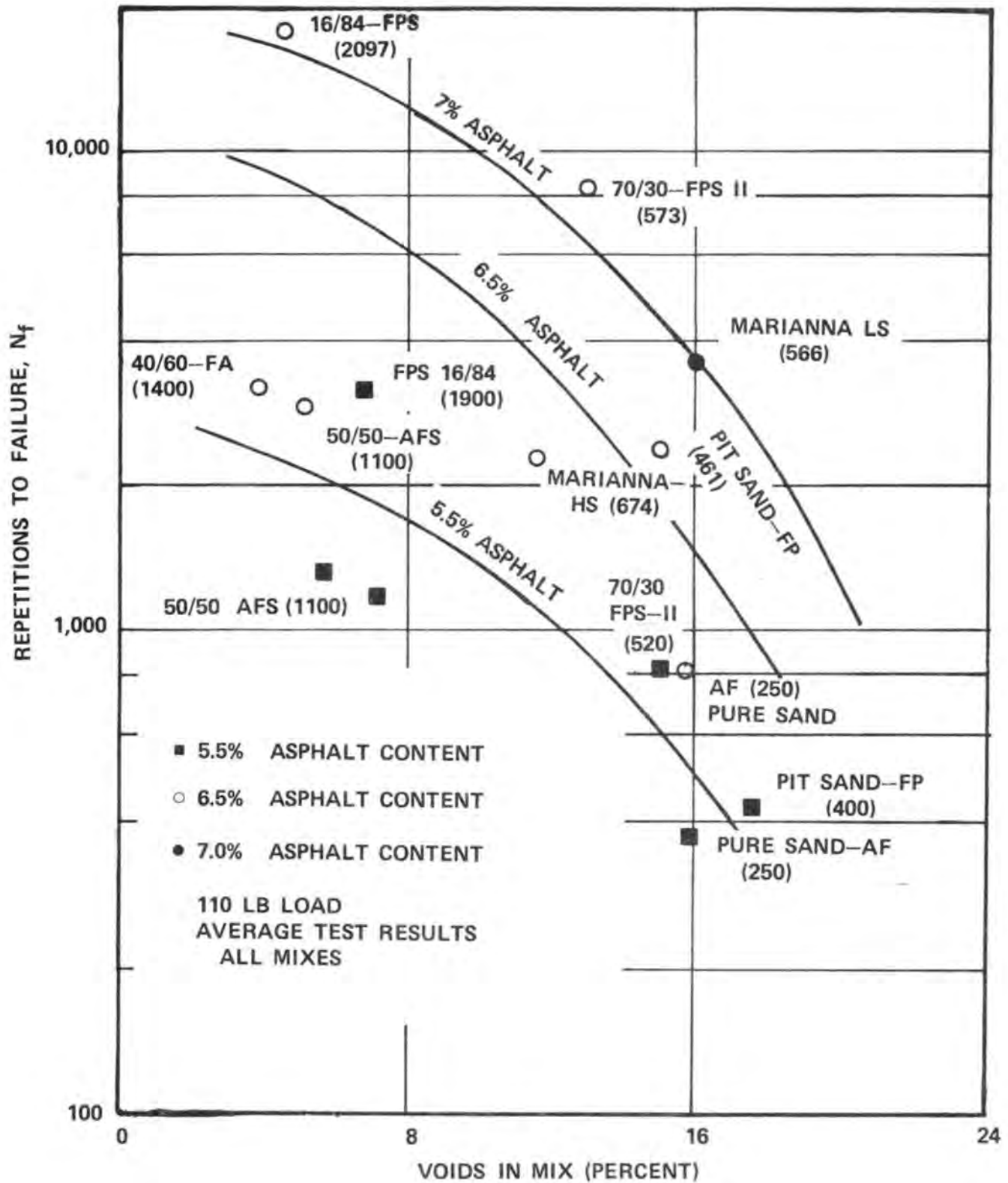


FIGURE 44. EFFECT OF AIR VOIDS AND ASPHALT CONTENT ON FATIGUE LIFE OF SAND-ASPHALT AND SAND-STONE BLEND MIXES

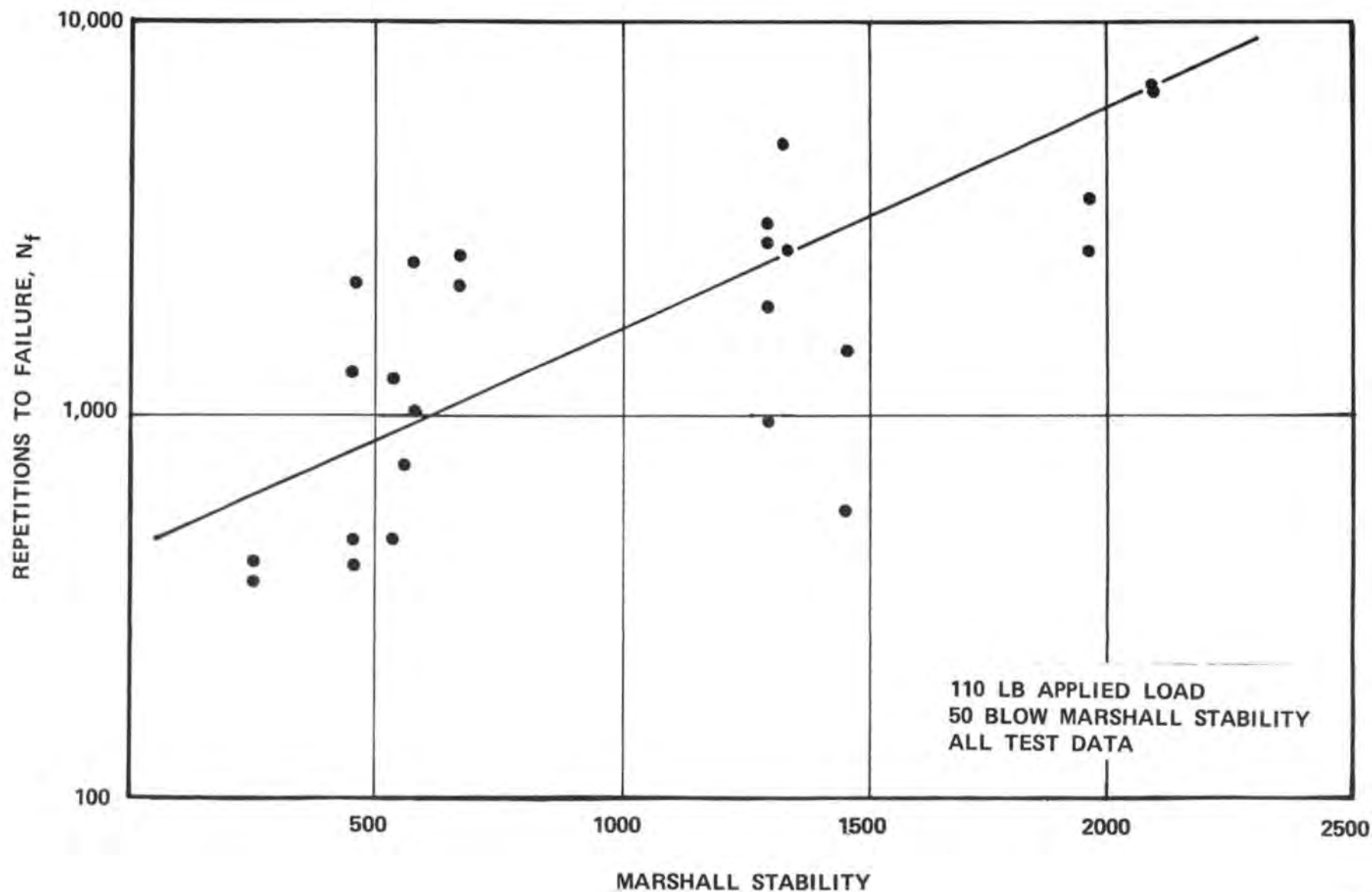


FIGURE 45. EFFECT OF MARSHALL STABILITY ON FATIGUE LIFE OF SAND-ASPHALT AND SAND-STONE BLEND MIXES

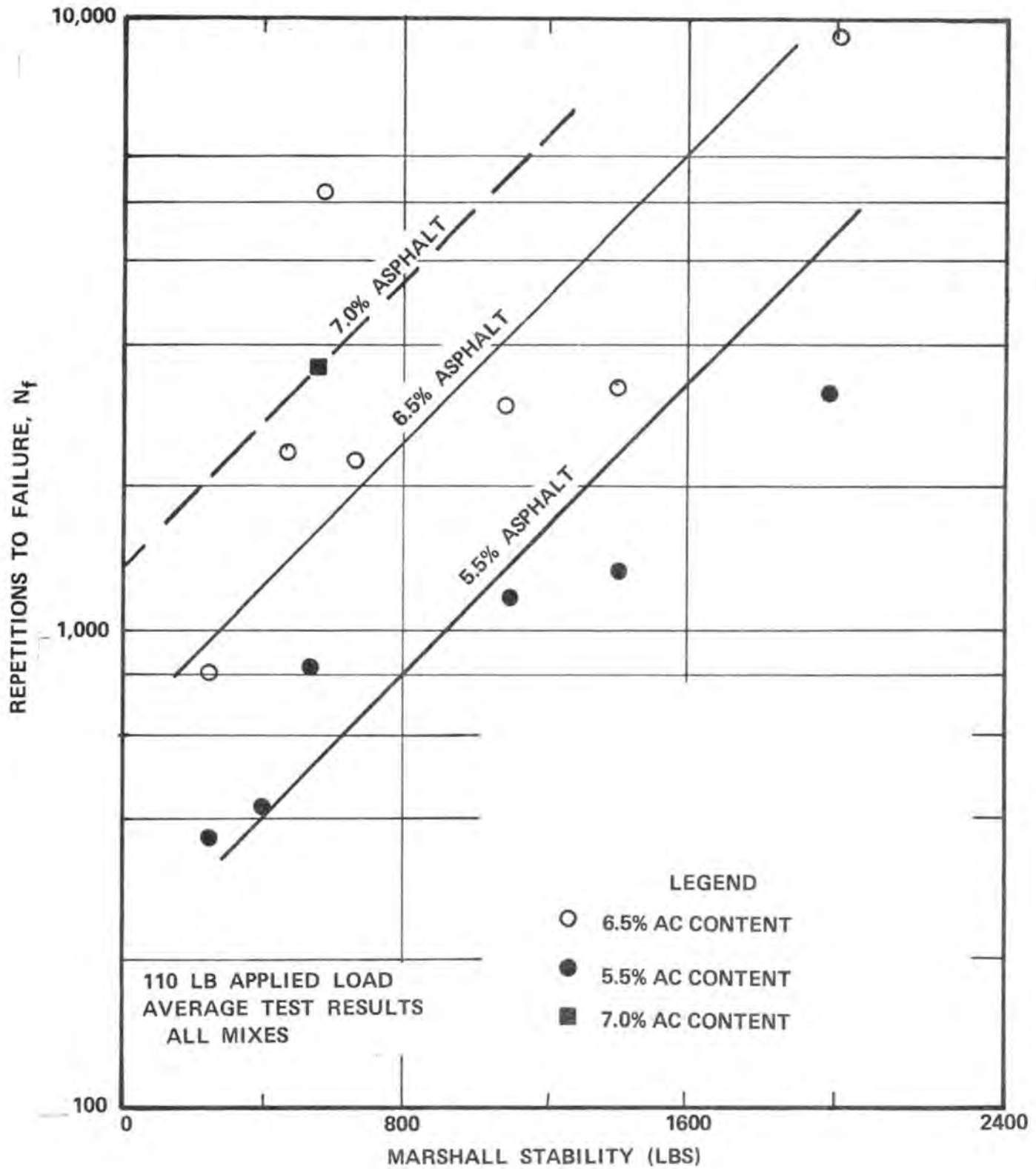


FIGURE 46. GENERAL EFFECT OF MARSHALL STABILITY AND ASPHALT CONTENT ON FATIGUE LIFE - ALL SAND-ASPHALT AND SAND-STONE MIXES

mixes as shown in Fig. 47. In the present study of sand-asphalt and sand-stone blend base mixes, a general trend was found between air voids in the mix and Marshall stability (Fig. 48). Therefore, it came as no surprise that mixes with high Marshall stabilities generally exhibited long fatigue lives since these mixes also had low air void contents. The air void content of the mix was probably a more fundamental parameter with respect to fatigue life than Marshall stability. Generally, however, Marshall stability appeared to be directly related to the fatigue resistance in the mixes investigated.

The relationship between the dynamic bending modulus of the beam specimens and fatigue life is shown in Fig. 49. The dynamic modulus was calculated from the measured tensile strain in the beam using the procedure developed by Barksdale and Miller [54]. Fatigue life became larger at an increasing rate as the bending modulus increased and also increased with asphalt content, as can be seen from this figure.

Fatigue test results indicated that the fatigue life of sand-asphalt and sand-stone blends was directly related to asphalt content and air voids in the mix. Previous research [cf. 7,17,54] indicated that asphalt content and air voids in conventional stone mixes were the primary independent variables related to fatigue life. Due to the wide range of materials and gradations of the sand-asphalt and sand-stone base mixes tested in this study, however, other variable(s) undoubtedly affected fatigue life in addition to asphalt content and air voids. Based on fatigue test results for the limited number of mixes studied in this investigation, the Marshall stability of the mix, in part, appeared to account for this observed variability. Although Marshall stability was probably not a fundamental independent variable, for the present time, Marshall stability of the mix together with asphalt content and air voids can be used in design for estimating the fatigue resistance of sand-asphalt and sand-stone blends.

Rutting

Rutting test results for the sand and sand-stone asphalt mixes are summarized in Table 23 and Figs. 50-52. In these figures, laboratory test results are presented in terms of the theoretical rut depth that would occur in the base of a typical pavement section having a sand-asphalt or sand-stone blend base. Comparisons of rut depths were made for full-depth bituminous pavements that had a 3.5 in. (90 mm) asphalt concrete surfacing and a 7 in. (180 mm) sand-asphalt or sand-stone blend base. This pavement section was assumed to be constructed over a reasonably good subgrade that had a modulus of elasticity of approximately 4,000 psi (28,000 kN/m²). The pavement section was assumed to be located in the coastal plain area of the southeast where the mean pavement temperature for rutting was close to 95°F (35°C). The rut depths given were for 1,200,000 equivalent 18 kip (80 kN) single axle loads. From the correlation studies of rutting described in Chapter 3, λ_1 was taken to be 3.0 (Table 16) and Z to be 0.42 (Fig. 21) for the given structural and environmental conditions.

Table 23. Summary of Repeated Load Triaxial Test Results Performed on Sand and Sand-Stone Blend Asphalt Specimens - 100,000 Load Repetitions at 95°F

| Sand Source | Blend | | Test No. | A.C. (%) | Voids (%) | Marshall Stability (lbs.) | Stress State | | Plastic Strain ϵ_p (in./in.) |
|-------------|-------------------------|-----------|------------|-------------|-----------|---------------------------|------------------|-----------------------------|---------------------------------------|
| | Sand (%) | Stone (%) | | | | | σ_3 (psi) | $\sigma_1 - \sigma_3$ (psi) | |
| Finley Pit | 100 | 0 | FP-I-1 | 6.5 | 15.0 | 461 | 5 | 40 | (Failed) |
| Finley Pit | 100 | 0 | FP-I-2 | 6.5 | 15.0 | 461 | 5 | 25 | 0.0175 |
| Altamaha | 100 | 0 | FP-I-3 | 6.5 | 15.0 | 461 | 5 | 25 | 0.0086 |
| Altamaha | 100 | 0 | FP-I-4 | 6.5 | 15.0 | 461 | 5 | 25 | (Failed) |
| Altamaha | 100 | 0 | FP-I-5 | 6.5 | 15.0 | 461 | 5 | 25 | 0.0111 |
| Altamaha | 100 | 0 | FP-II-1 | 5.5 | 17.6 | 461 | 5 | 25 | 0.0106 |
| Altamaha | 100 | 0 | FP-II-2 | 5.5 | 17.6 | 461 | 3 | 33 | (Failed) |
| Altamaha | 100 | 0 | FP-II-3 | 5.5 | 17.6 | 461 | 5 | 25 | 0.0140 |
| Altamaha | 16 | 84 | 2-FPS-I-1 | 5.5 | 6.7 | 1963 | 5 | 25 | 0.004 |
| Altamaha | 16 | 84 | 2-FPS-I-2 | 5.5 | 6.7 | 1963 | 5 | 25 | 0.0046 |
| Altamaha | 16 | 84 | FPS-I-1 | 6.5 | 4.5 | 2097 | 5 | 25 | 0.0121 |
| Altamaha | 16 | 84 | FPS-I-2 | 6.5 | 4.5 | 2097 | 5 | 25 | (Void) |
| Altamaha | 16 | 84 | FPS-I-3 | 6.5 | 4.5 | 2097 | 5 | 25 | 0.013 |
| Altamaha | 16 | 84 | FPS-I-4 | 6.5 | 4.5 | 2097 | 5 | 25 | 0.0072 |
| Altamaha | 40 | 60 | FA-I-1 | 6.5 | 3.4 | 1359 | 5 | 25 | 0.005 |
| Altamaha | 40 | 60 | FA-I-2 | (Void Test) | | | | | |
| Altamaha | 40 | 60 | FA-I-3 | 6.5 | 3.4 | 1350 | 5 | 25 | 0.018 |
| Altamaha | 40 | 60 | FA-I-4 | 6.5 | 3.4 | 1350 | 5 | 25 | 0.0173 |
| Altamaha | 40 | 60 | FA-I-5 | 6.5 | 3.4 | 1350 | 5 | 25 | 0.012 |
| Altamaha | 40 | 60 | FA-II-1 | 5.5 | 5.7 | 1350 | 5 | 25 | 0.0045 |
| Altamaha | 40 | 60 | FA-II-2 | 5.5 | 5.7 | 1350 | 5 | 25 | 0.013 |
| Altamaha | 40 | 60 | FA-II-3 | 5.5 | 5.7 | 1350 | 5 | 25 | (Failed) |
| Altamaha | 40 | 60 | EA-II-4 | 5.5 | 5.7 | 1350 | 5 | 25 | 0.019 |
| Altamaha | 40 | 60 | FA-II-5 | 5.5 | 5.7 | 1350 | 5 | 25 | 0.0052 |
| Altamaha | 70 | 30 | FPS-II-1 | 6.5 | 13.0 | 573 | 5 | 25 | 0.025 |
| Altamaha | 70 | 30 | FPS-II-2 | 6.5 | 13.0 | 573 | 5 | 25 | 0.017 |
| Altamaha | 70 | 30 | FPS-II-3 | 6.5 | 13.0 | 573 | 5 | 25 | (Failed) |
| Altamaha | 70 | 30 | 2-FPS-II-1 | 5.5 | 15.0 | 533 | 5 | 25 | 0.0089 |
| Altamaha | 70 | 30 | 2-FPS-II-2 | 5.5 | 15.0 | 533 | 5 | 25 | 0.0080 |
| I-95 | | | I95-1-C | | | | 5 | 28 | 0.0134 |
| I-95 | Sand, Shell Stone Blend | | I95-2-C | - | - | - | 5 | 25 | 0.010 |
| I-95 | | | I95-3-C | | | | 5 | 25 | 0.012 |
| I-95 | | | I95-4-C | | | | 5 | 25 | 0.0115 |
| Marianna | Low Stability | | LS-I-1 | 7.0 | - | - | 5 | 25 | 0.0036 |
| Marianna | | | LS-I-2 | 7.0 | - | - | 5 | 25 | 0.0045 |
| Lake Wales | 50 | 50 | LW-I-1 | 7.5 | - | - | 5 | 25 | (Failed) |
| Lake Wales | 50 | 50 | LW-I-2 | 7.5 | - | - | 5 | 20 | 0.0091 |
| Marianna | High Stability | | HS-I-1 | 6.5 | - | - | 5 | 25 | 0.0064 |
| Marianna | | | HS-I-2 | 6.5 | - | - | 5 | 25 | 0.0043 |
| Marianna | | | HS-I-3 | 6.5 | - | - | 5 | 25 | .001 |
| Altamaha | 100 | 0 | FP-I-6 | 6.5 | 15.0 | | 4 | 30 | (Failed) |
| Altamaha | 100 | 0 | FP-I-7 | 6.5 | 15.0 | | 5 | 29 | (Failed) |
| Altamaha | 100 | 0 | FP-I-8 | 6.5 | 15.0 | | 5 | 12 | 0.0045 |
| Altamaha | 100 | 0 | FP-I-9 | 6.5 | 15.0 | | 5 | 25 | 0.0088 |
| Altamaha | 100 | 0 | FP-I-10 | 5.0 | 18.4 | | 5 | 25 | 0.0107 |
| Altamaha | 100 | 0 | AF-I-1 | 5.5 | - | - | 5 | 25 | (Failed) |
| Altamaha | 100 | 0 | AF-II-1 | 6.5 | - | - | 5 | 25 | 0.0048 |
| Altamaha | 50 | 50 | AFS-I-1 | 5.5 | 7.3 | | 5 | 25 | 0.007 |
| Altamaha | 50 | 50 | AFS-II-1 | 6.5 | 5.0 | | 5 | 25 | 0.0041 |

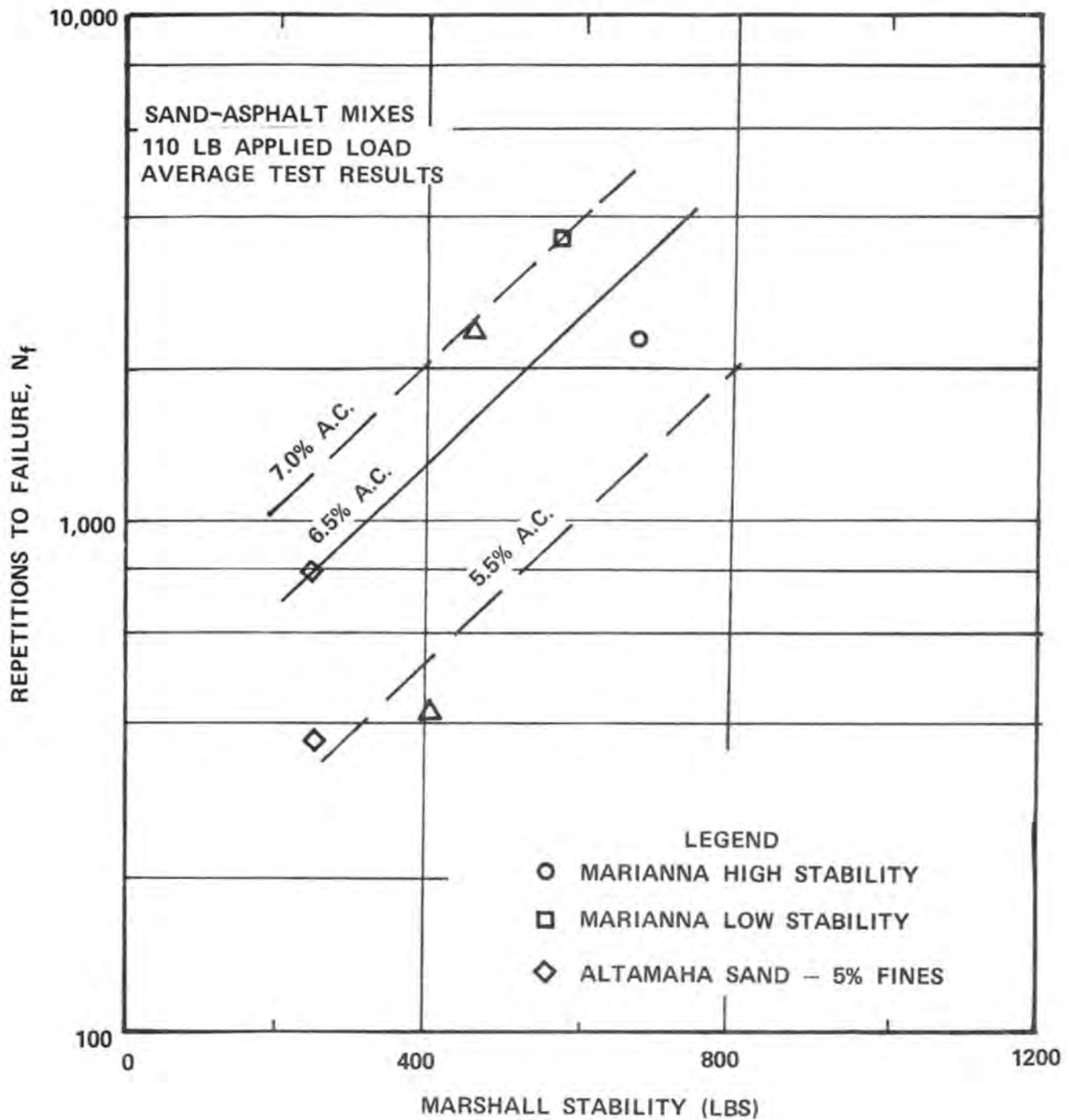


FIGURE 47. GENERAL EFFECT OF MARSHALL STABILITY AND ASPHALT CONTENT ON FATIGUE LIFE-SAND-ASPHALT MIXES

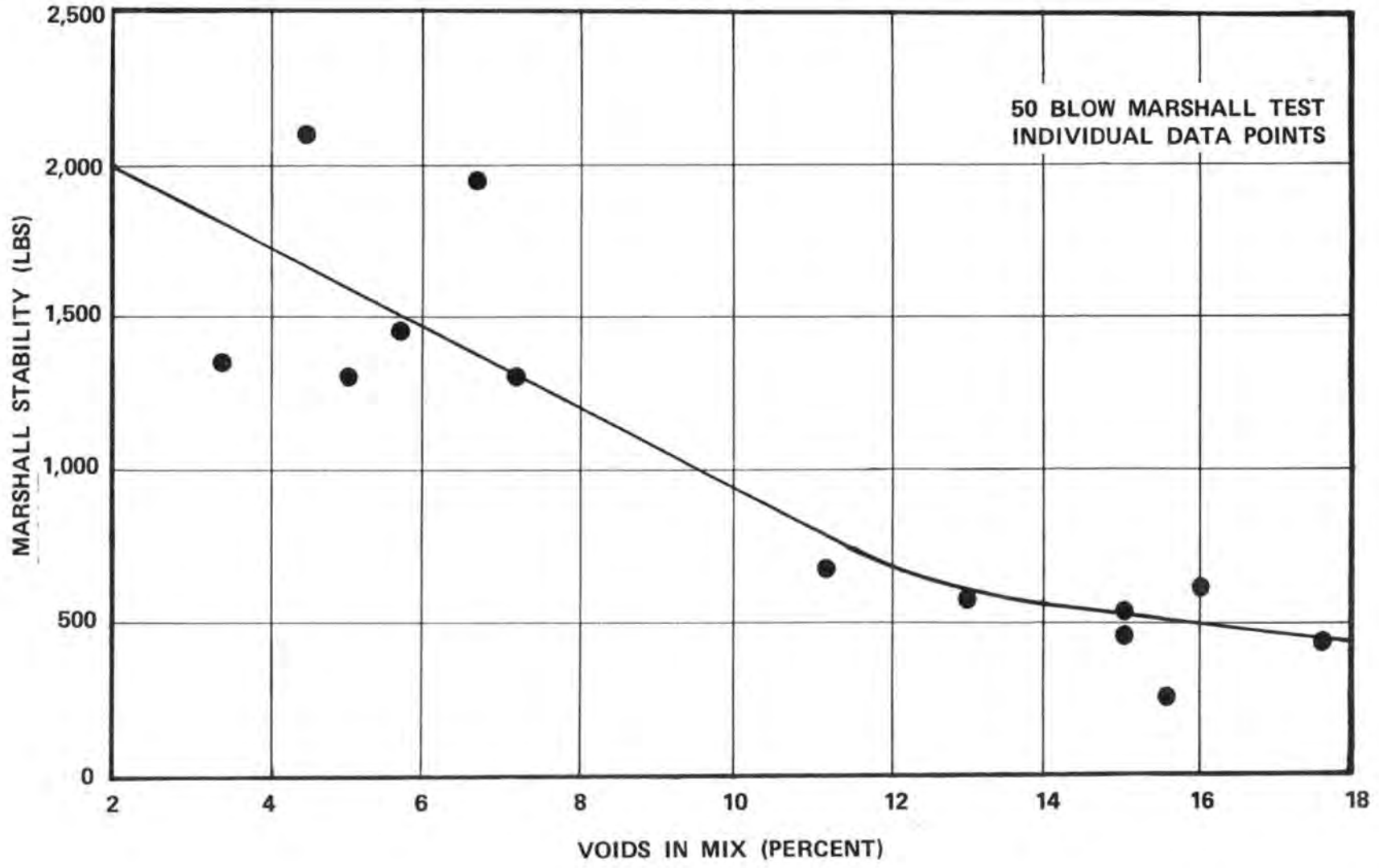


FIGURE 48. RELATIONSHIP BETWEEN MARSHALL STABILITY AND VOIDS IN MIX FOR SAND-ASPHALT AND SAND-STONE BLENDS TESTED

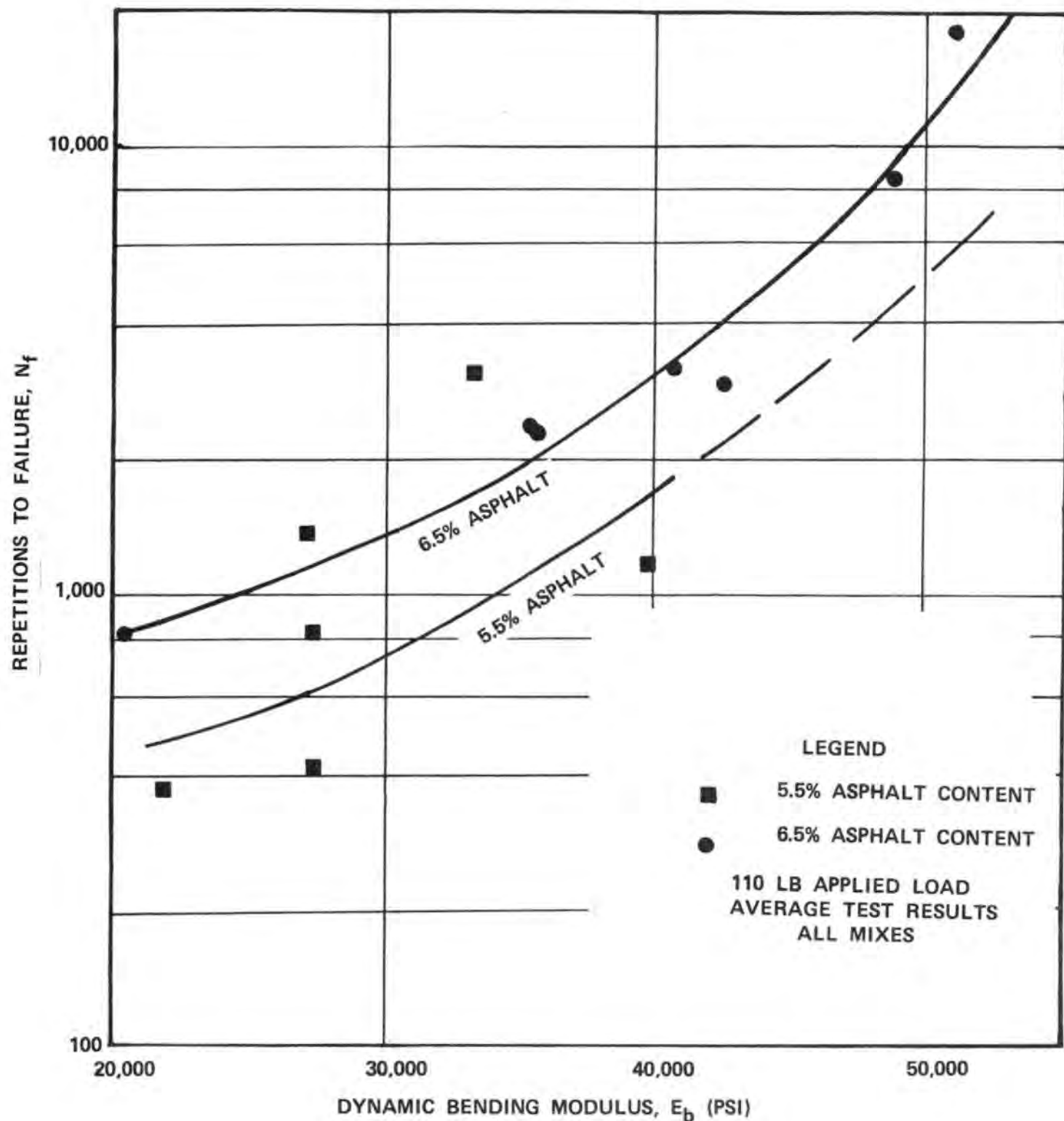


FIGURE 49. EFFECT OF ASPHALT CONTENT AND REPETITIONS TO FAILURE ON BENDING MODULUS—SAND—ASPHALT AND SAND—STONE BLEND MIXES

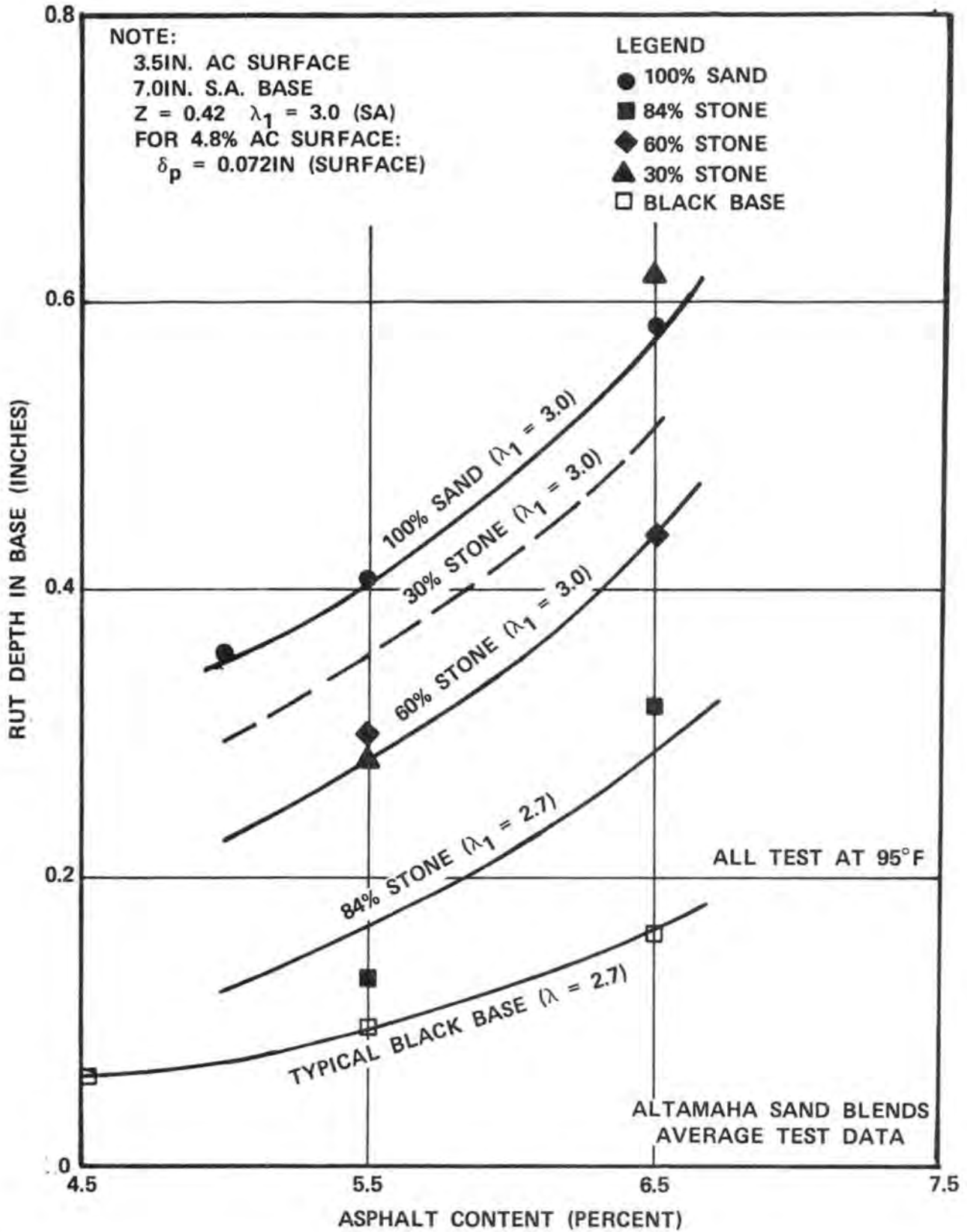


FIGURE 50. RELATIONSHIP BETWEEN STONE CONTENT AND RUT DEPTH FOR VARYING ASPHALT CONTENTS—ALTAMAHA SAND—STONE BLEND MIXES

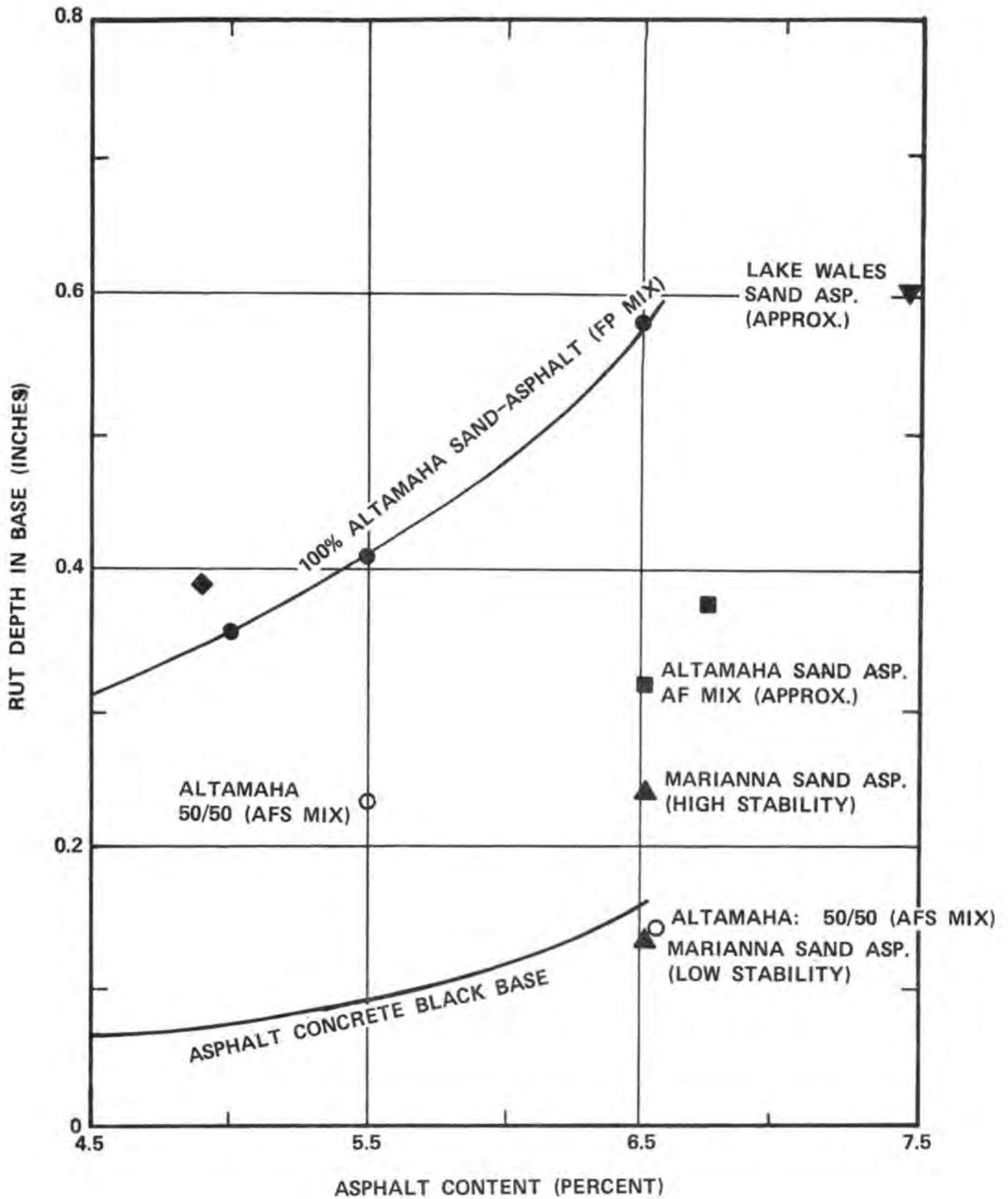


FIGURE 51. EFFECT OF MIX TYPE ON RUT DEPTH—SELECTED SAND-ASPHALT AND SAND-STONE BLEND MIXES TESTED

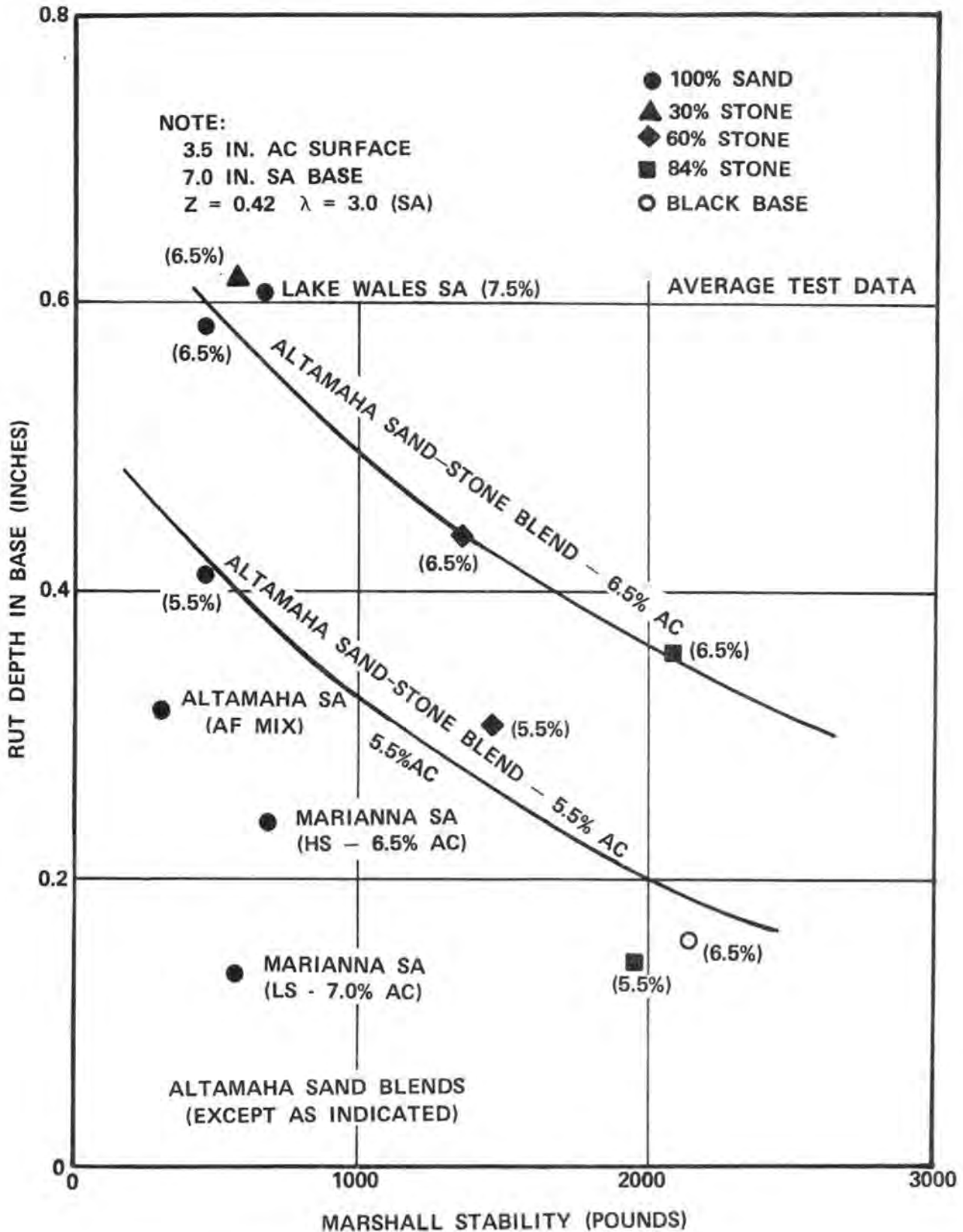


FIGURE 52. EFFECT OF MARSHALL STABILITY AND ASPHALT CONTENT ON RUT DEPTH - ALTAMAHA SAND-STONE BLEND MIXES AND SELECTED SAND-ASPHALT MIXES

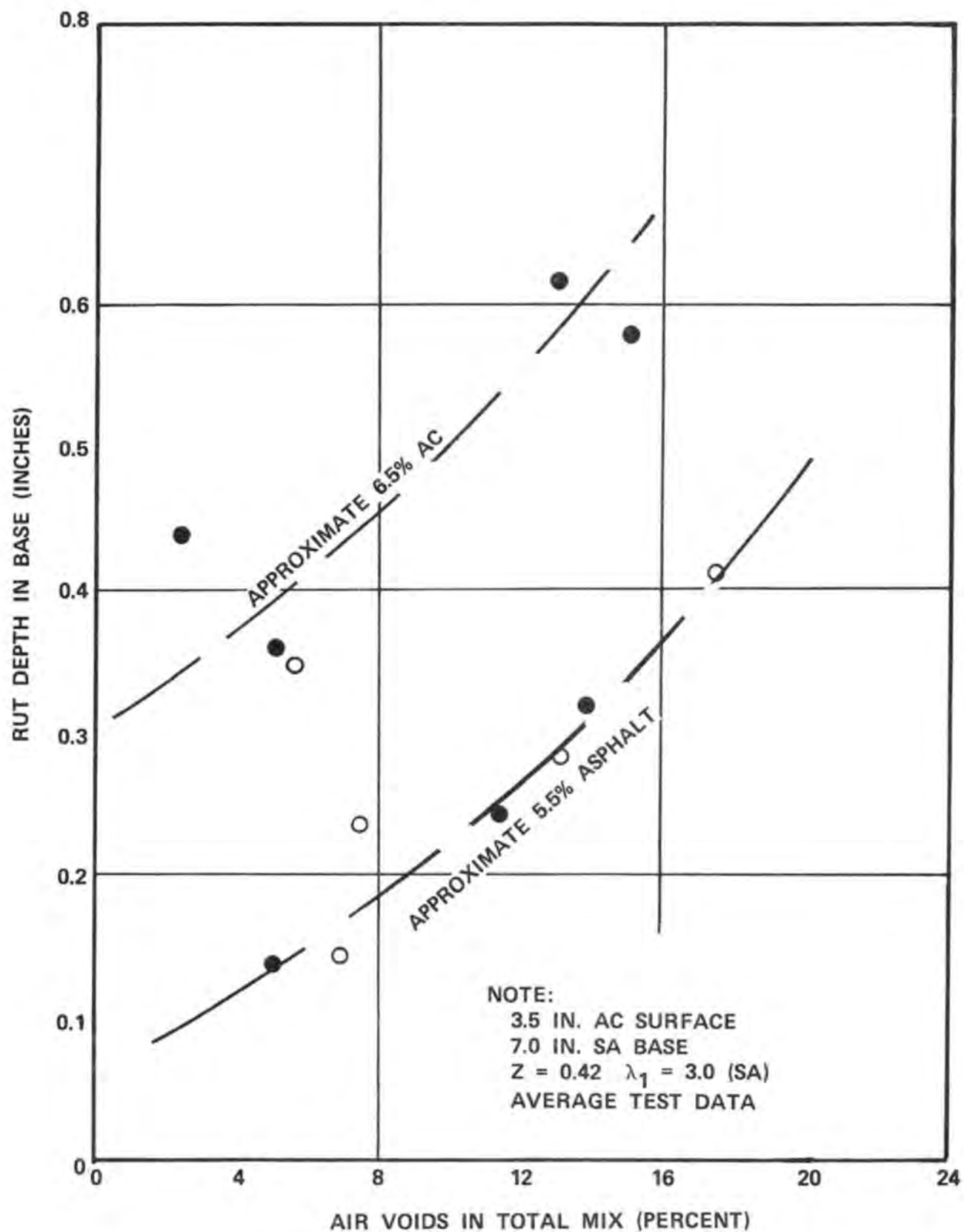


FIGURE 53. GENERAL EFFECT OF AIR VOIDS AND ASPHALT CONTENT ON RUT DEPTH

For an asphalt concrete surface course mix that had 6.5 percent asphalt, the repeated load triaxial test results given by Barksdale and Miller [54] indicated that the typical plastic strain, ϵ_p , of the specimen when subjected to 100,000 repetitions was $\epsilon_p \approx 0.0073$ in./in. and for an asphalt content of 4.8 percent approximately 0.0043 in./in. These plastic strains were for the standard testing conditions corresponding to a confining pressure of 5 psi (35 kN/m²), a deviator stress of 25 psi (170 kN/m²), and a testing temperature of 95°F (35°C). The plastic strain properties obtained from the repeated load triaxial test for the sand-asphalt and sand-stone blends tested appear in Table 23 and were also for the same standard test conditions.

By comparing predicted rut depths rather than the measured plastic strains, a better feeling was developed for the effect of the mix variables on the actual relative magnitude of rutting that was likely to develop in a typical pavement section. The theoretical approach used to calculate the rut depth was described in Chapter 3 and is summarized by equation (6) for asphalt concrete sections and equation (18) for pavements with sand-asphalt and sand-stone blend bases. The theoretical relationships for rutting given in Figs. 50-52 were for the specific mixes tested. Since the magnitude of rutting was found to vary greatly for different mixes, great caution should be exercised in generalizing these results to other mixes.

Test Results: With an increase in asphalt content, rut depth in the sand-asphalt and sand-stone blends tested rose at an increasing rate (Fig. 50). For an increase in asphalt content from 5.5 to 6.5 percent, the rut depth increased by 40 to 70 percent for the sand-asphalt and sand-stone blends tested. This compared favorably with an increase in rutting of 70 percent for a typical asphalt concrete black base mix tested previously [54]. The pure sand-asphalt base mix was most affected in terms of increase in rut depth by an increase in asphalt content, with the effect becoming less with increasing stone content. These test results indicated that addition of stone to a sand-asphalt mix was quite effective in reducing rutting of the mix. For example, for an asphalt content of 6 percent, going from a pure sand-asphalt to a 40 percent sand/60 percent stone base mix was found to reduce rutting in the base by approximately 25 percent. Similar blends using other materials may show even greater reductions in rut depth with increasing stone content. The reduction in rut depth that accompanied increased stone content was probably due to increased internal friction of the mineral skeleton that resulted from the presence of large-size stone aggregate in the sand-stone blends. The large-size aggregate tends to decrease the number of grain-to-grain point contacts and increase aggregate interlock. Since rutting was found to be directly related to asphalt content, an optimum asphalt content within normally used limits was not found that would minimize rutting. Rutting, of course, can be minimized by lowering asphalt content, blending sands, or by adding crushed stone or mineral filler to the mix.

The large variation in rutting observed for the different materials tested is illustrated in Fig. 51 as a function of asphalt content. For comparison, the relationship between rut depth in the base and asphalt

content shown on the figure was for the typical crushed granite gneiss black base mix studied previously [54]. The I-95 sand-stone blend (tests were performed on both compacted specimens and cores taken from the roadway), the Lake Wales sand-asphalt, and the Altamaha sand-asphalt (AF) mix all indicated relatively large rutting in the base (greater than 0.3 in.). In contrast, the Marianna sand-asphalt mixes and the Altamaha 50 percent sand-50 percent stone (AFS) mix both indicated rut depths less than 0.24 in. (6 mm).

The influence on rut depth of Marshall stability and asphalt content for the Altamaha sand-stone blends is shown in Fig. 52. Each Altamaha sand-stone mix was blended with the same type sand and crushed stone, and only the relative amounts were varied. For these mixes, which were composed of similar materials, rut depth for a given asphalt content was found to be almost inversely proportional to the 50 blow Marshall stability of the mix. This finding indicated that the Marshall stability can probably be used as a general guide for evaluating the relative beneficial effect on rutting of blending crushed stone with sand or perhaps due to blending two sands together. The test results shown in Fig. 52 indicate that both Marshall stability and asphalt content were related to rutting. However, when mixes composed of different materials were included, the scatter in data shown on the figure indicated that other less well-defined characteristics of the mix (e.g., grain size and angularity of the sand) also had important effects on rutting. Figure 53 shows the theoretical relationship based on laboratory test results between rut depth in the base, air voids in the mix, and asphalt content. Once again when all mixes were considered, the scatter was relatively large. An apparent general trend was observed of increasing rut depth at an increasing rate with increased air voids and with increased asphalt content.

The test results shown in Figs. 50 through 53 indicate that rutting in sand-asphalt and sand-stone blend asphalt base mixes was influenced in a reasonably well-defined way by asphalt content and in a less well-defined manner by both Marshall stability and air voids. Marshall stability and air voids in the mix, however, appeared to be interrelated variables since a reasonably well-defined relationship was found to exist between these variables (Fig. 48). Because of the scatter in the data shown in Figs. 51 and 52, apparently other factors influenced the overall rutting characteristics of the mix when subjected to heavy traffic. Acott [22] concluded that fineness modulus and shape factor are both important variables and that a sand suitable for use in a gap-graded mix must have a fineness modulus greater than 1.0 when the shape factor is less than 0.7. Analogous relationships need to be developed for sand-asphalt base mixes.

Sand-Cement

The results of the beam fatigue tests performed on the sand-cement specimens are summarized in Table 25. A comparison of these fatigue results with those obtained for a typical black base asphalt concrete mix [54] are shown in Fig. 54. All of the sand-cement laboratory specimens except those with low densities performed better with respect to

Table 24. Summary of Fatigue Test Results on Sand and Sand-Stone Blend Asphalt Mixes

| Blend | Sample | A.C. (%) | Voids (%) | ϵ_r (1) | N_f (2) | Load (lbs.) | E (3) (psi) | Marshall Stability (lbs.) | Avg. N_f | E Avg. (psi) |
|--|--------------|-------------|--------------|-----------------------|-----------|----------------|----------------|---------------------------------|---------------------|--------------------|
| Marianna High Stability | HS-1-1 | 6.5 | 11.4 | 1.4×10^{-3} | 2522 | 110 | 37,724 | 674 | 2270 | 35,700 |
| | HS-1-2 | 6.5 | 11.4 | 8.8×10^{-4} | 2105 | 110 | 10,777,712 | 674 | | |
| | HS-1-3 | 6.5 | 11.4 | 6.94×10^{-4} | 2182 | 110 | 91,059 | 674 | | |
| | HS-1-4 | 6.5 | 11.4 | 8.24×10^{-4} | 4032 | 72.8 lbs.* | 41,806 | 674 | | |
| Marianna Low Stability | LS-1-1 | 7.0 | 16 | 1.89×10^{-3} | 700 | 110 | 23,900 | 566 | 3626 | 58,000 |
| | LS-1-2 | 7.0 | 16 | 1.14×10^{-3} | ~2000 | 110 | 47,000 | 566 | | |
| | LS-1-3 | 7.0 | 16 | 1.18×10^{-4} | 8180 | 72.8 | 103,248 | 566 | | |
| Altamaha Sand 95% Sand 5% Fines | AF-I-1 | 5.5 | 15.7 | 2.52×10^{-3} | 400 | 110 | 16,323 | ~ 250 | 375 | 21,800 |
| | AF-I-2 | 5.5 | 15.7 | 1.79×10^{-3} | 350 | 115 | 27,311 | ~ 250 | | |
| | AF-1-3 | 6.5 | 13.9 | 2.14×10^{-3} | 827 | 110 | 20,300 | ~ 250 | | |
| Altamaha River Sand Stone 46.25% Fines 7.5% | AFS-I-1 | 5.5 | 7.0 | 1.44×10^{-3} | 1000 | 110 | 34,400 | 1100 | 1167 | 39,760 |
| | AFS-I-2 | 5.5 | 7.0 | 9.54×10^{-4} | 1900 | 110 | 59,600 | 1100 | | |
| | AFS-I-3 | 5.5 | 7.0 | 1.89×10^{-3} | 600 | 115 | 25,300 | 1100 | | |
| | AFS-II-1 | 6.5 | 5.0 | 1.31×10^{-3} | 2800 | 116 | 41,900 | 1100 | | |
| | AFS-II-2 | 6.5 | 5.0 | 1.2×10^{-3} | 3100 | 110 | 43,870 | 1100 | | |
| Pure Pit Sand | FP-I-4 | 6.5 | 15.0 | 1.63×10^{-3} | 1233 | 110 | 29,167 | 461 | 2375 @ 6.5% | 35,430 |
| | FP-1 (4) | 6.5 | 15.0 | 1.23×10^{-3} | 25,000 | 110 | 37,099 | 461 | | |
| | FP-II-2 | 5.5 | 17.6 | 1.57×10^{-3} | 450 | 110 | 30,600 | 400 | | |
| | FP-2 | 6.5 | 15.0 | 1.13×10^{-3} | 5710 | 110 | 43,325 | 461 | | |
| | FP-I-5 | 6.5 | 15.0 | 1.6×10^{-3} | 400 | 100 | 29,900 | 461 | | |
| | FP-I-3 | 6.5 | 15.0 | 1.37×10^{-3} | 2150 | 110 | 37,660 | 461 | | |
| | FP-II-3 | 5.5 | 17.6 | 6.3×10^{-4} | 450 | 110 | 21,600 | 400 | | |
| | FP-II-1 | 5.5 | 17.6 | 1.6×10^{-3} | 375 | 110 | 29,900 | 400 | | |
| 16% Sand 84% Stone Screenings | FPS-1 (4) | 6.5 | 4.5 | 1.11×10^{-3} | 28,658 | 110 | 45,400 | 2097 | 18,170 @ 6.5% | 51,000 |
| | FPS-2 (4) | 6.5 | 4.5 | 6.82×10^{-4} | 30,000 | 110 | 84,940 | 2097 | | |
| | 2FPS-I-2 | 5.5 | 6.7 | 1.7×10^{-3} | 2,716 | 110 | 27,600 | 1900 | | |
| | 2FPS-I-1 | 5.5 | 6.7 | 1.31×10^{-3} | 3,600 | 110 | 39,000 | 1900 | | |
| | FPS-I-4 | 6.5 | 4.5 | 1.31×10^{-3} | 7,200 | 110 | 39,000 | 2097 | | |
| | FPS-I-3 | 6.5 | 4.5 | 1.42×10^{-3} | 6,828 | 110 | 35,055 | 2097 | | |
| 70% Sand 30% Stone Screenings | FPS-II-1 (4) | 6.5 | 13.0 | 9.74×10^{-4} | 24,300 | 110 | 52,800 | 573 | 10,552 @ 6.5% | 48,875 |
| | FPS-II-4 | 6.5 | 13.0 | 4.06×10^{-4} | 996 | 110 | 38,300 | 573 | | |
| | FPS-II-2 (4) | 6.5 | 13.0 | 7.4×10^{-4} | 14,500 | 110 | 73,700 | 573 | | |
| | FPS-II-3 | 6.5 | 13.0 | 1.57×10^{-3} | 2,413 | 110 | 30,700 | 573 | | |
| | 2FPS-II-2 | 5.5 | 15.0 | 2.14×10^{-3} | 454 | 110 | 20,300 | ~ 520 | | |
| | 2FPS-II-1 | 5.5 | 15.0 | 1.44×10^{-3} | 1,209 | 110 | 34,400 | ~ 520 | | |
| 40% Sand 60% Stone | FA-1 | 6.5 | 3.8 | 9.84×10^{-4} | 5,000 | 110 | 57,200 | 1400 | 3,210 @ 6.5% | 40,730 |
| | FA-I-2 | 6.5 | 3.4 | 1.31×10^{-3} | 2,500 | 110 | 39,000 | 1400 | | |
| | FA-II-1 | 5.5 | 5.7 | 1.32×10^{-3} | 2,100 | 110 | 38,640 | ~ 1400 | | |
| | FA-II-2 | 5.5 | 5.7 | 2.6×10^{-3} | 534 | 110 | 15,649 | ~ 1400 | | |

1. Resilient strain measured after 1,000 load repetitions using an SR-4 strain gauge.
2. Repetitions to failure, N_f
3. Resilient modulus of beam in bending calculated using the resilient strain.
4. Fatigue tests performed on rubber subgrade having a modulus of Subgrade Reaction of 284 pci; all other tests were performed on a rubber subgrade having a modulus of Subgrade Reaction of 215 pci.

Table 25. Summary of Fatigue Test Results For Sand and Sand-Stone Blend Cement Stabilized Specimens

| MLX | Description | Comp. Str. (psi) | Density (pcf) | Applied Load (lbs.) | Rep. to Failure | Tensile ⁽²⁾ Strain ($\times 10^{-4}$ in/in) | Bending Modulus (psi) ⁽²⁾ | | Comments |
|------|---|--------------------|---------------|---------------------|------------------|---|--------------------------------------|------------------------------|--|
| | | | | | | | Strain ($\times 10^3$) | Deflection ($\times 10^3$) | |
| PPS | Fine Pit Sand 6% cement | 315 | 122.2 | 198 | 179,433 | 1.59 | 692 | 126 | |
| | | | 122.1 | 172 336 | 857,010 50 | 1.14 2.44 | 765 - | - - | No failure; load to 336 lbs. |
| PALT | Altamaha River Sand - 8.5% cement | 375 | 112.3 | 230 | 4 | 3.44 | 468 | 42 | Low Density |
| | | | 117.5 | 148 245 | 816,657 1,757 | 0.69 - | 1,195 - | 915 - | No failure; load to 245 lbs. |
| PS | 50% Fine Pit Sand 50% Crushed Stone - 5% cement | 395 ⁽³⁾ | 129.8 | 200 | 25,800 | 1.72 | 646 | 54 | |
| | | | 131.1 | 226 | 440,367 | 1.05 | 1,195 | 85 | No failure |
| | | | 130.5 | 272 | 162,414 | 1.58 | 956 | 153 | |
| APF | Altamaha River Sand with 5% Crushed Stone Fines - 8% cement | 335 | 122.7 | 119 240 | 1,900,000 20 | 0.97 | 656 | 160 | No failure; load to 240 lbs. |
| | | | 122.2 | 131 300 | | 0.69 | 1,060 | 236 | No failure; load to 300 lbs. |
| | | | 117.5 | 175 | 435 | 1.5 | 648 | - | Low Density |
| SPF | Altamaha River Sand with 5% Crushed Stone Fines - 6% cement | 235 | 115.8 | 144 | 166 | 1.65 | 486 | - | Low Density |
| | | | 116.1 | 121 | 151 | 1.25 | 537 | - | Low Density |
| PPS | Fine Pit Sand - 8% cement | | 137.2 | 139 | 187,000 | 0.75 | 817 | 99 | High Density |
| | | | 135.4 | 245 360 | 1,100 1 | 1.58 - | 861 - | 44 - | No Failure Failure at 360 lbs. static |
| APS | 50/50 Altamaha sand-stone blend 5% fines; 6% cement | 630 | 124.6 | 178 515 | 582,182 2 | 0.92 2.65 | 1,080 - | - - | No failure; load to 515 lbs. |
| | | | 127.6 | 230 | 2,720 | 0.96 | 1,335 | - | |

1. All tests were performed on rubber subgrade having a modulus of subgrade reaction of 284 pci.

2. Bending moduli values determined at 1,000 load repetitions.

3. The seven day unconfined compressive strength for 6 percent cement was 695 psi; the compressive strength for a 5 percent cement content was not determined.

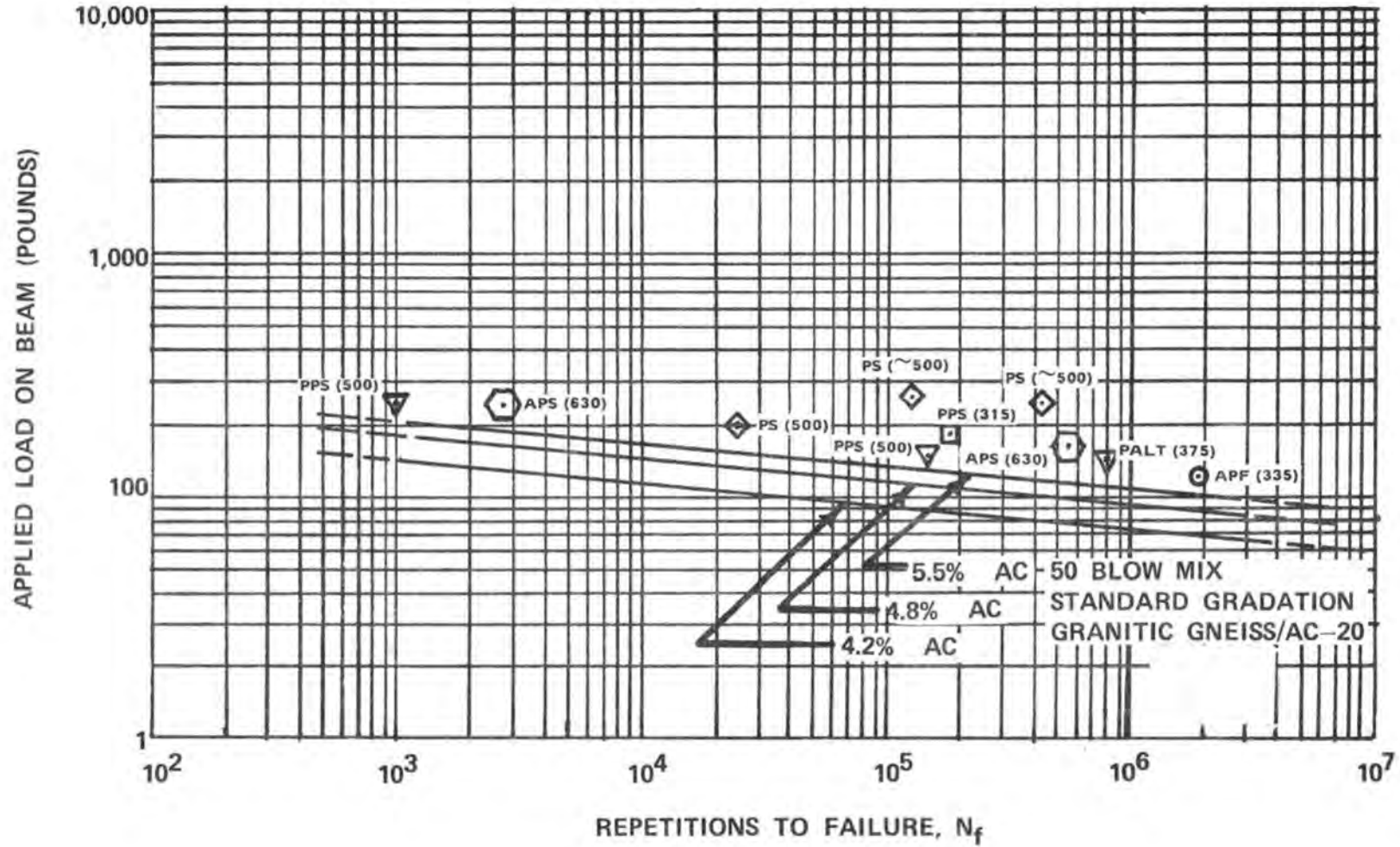


FIGURE 54. COMPARISON OF FATIGUE PERFORMANCE OF SAND-CEMENT AND SAND/STONE CEMENT SPECIMENS WITH A BLACK BASE ASPHALT CONCRETE MIX

fatigue than the asphalt concrete base mixes prepared using all crushed stone. The 50/50 blend of Finley Pit sand and crushed stone exhibited the best fatigue performance. This mix had a cement content of 5 percent and an estimated 7-day unconfined compressive strength of approximately 500 psi (3,450 kN/m²). The 50/50 blend of crushed stone and Altamaha sand having a cement content of 6 percent had an unconfined compressive strength of 630 psi (4,300 kN/m²). Although this material exhibited reasonably good fatigue properties, it did not perform as well as the 50/50 blend using Finley Pit sand and a cement content of 5 percent. The trend of better performance for Finley Pit sand than for Altamaha River sand was generally true for fatigue tests on the sand-cement specimens as well as the fatigue and rutting tests on the sand-asphalt specimens. Since the unconfined compressive strengths of these two blends were reasonably similar, fatigue resistance was apparently not a function of just the unconfined compressive strength of the mix.

The APF, PPS, APF and PALT mixes had unconfined compressive strengths in the range of 315 to 375 psi (2170 to 2585 kN/m²) and did surprisingly well when compared with the standard asphalt concrete base mixes. In all cases, the fatigue life of these sand-cement mixes exceeded that of the asphalt concrete base mixes. The Altamaha River sand mix, which had 5 percent crushed stone fines and 6 percent cement content, had the worst fatigue performance of all sand-cement mixes tested, and the fatigue resistance was less than that of the sand-asphalt mixes. This blend had a 7-day unconfined compressive strength of 235 psi (1620 kN/m²) which was considered low. Low unconfined compressive strength resulted partially because the specimens were compacted to an average of only 94 percent of the standard Proctor maximum dry density. An analysis of other test data indicated that fatigue life of the sand-cement mixes tested was greatly reduced when density was significantly less than 100 percent of the standard Proctor value. The water content of low compressive strength mixes was found to have an important influence on compressive strength as shown in Fig. 55. Seven-day unconfined compressive strength was approximately doubled for two of the low-compressive strength mixes investigated when water content was reduced from 9 to 5 percent.

Laboratory test results indicated that sand-cement mixes having field 7-day unconfined compressive strengths of 300 psi (2070 kN/m²) or greater should have sufficient fatigue resistance. Of great practical significance was the finding that both density and water content had a significant effect on fatigue resistance. In selecting an appropriate theoretical design cement content, variations in cement content that occur during mixing and placing and detrimental effects of weathering must also be considered. The laboratory fatigue test results presented in this section, of course, did not consider the important effects of shrinkage cracking upon overall pavement performance.

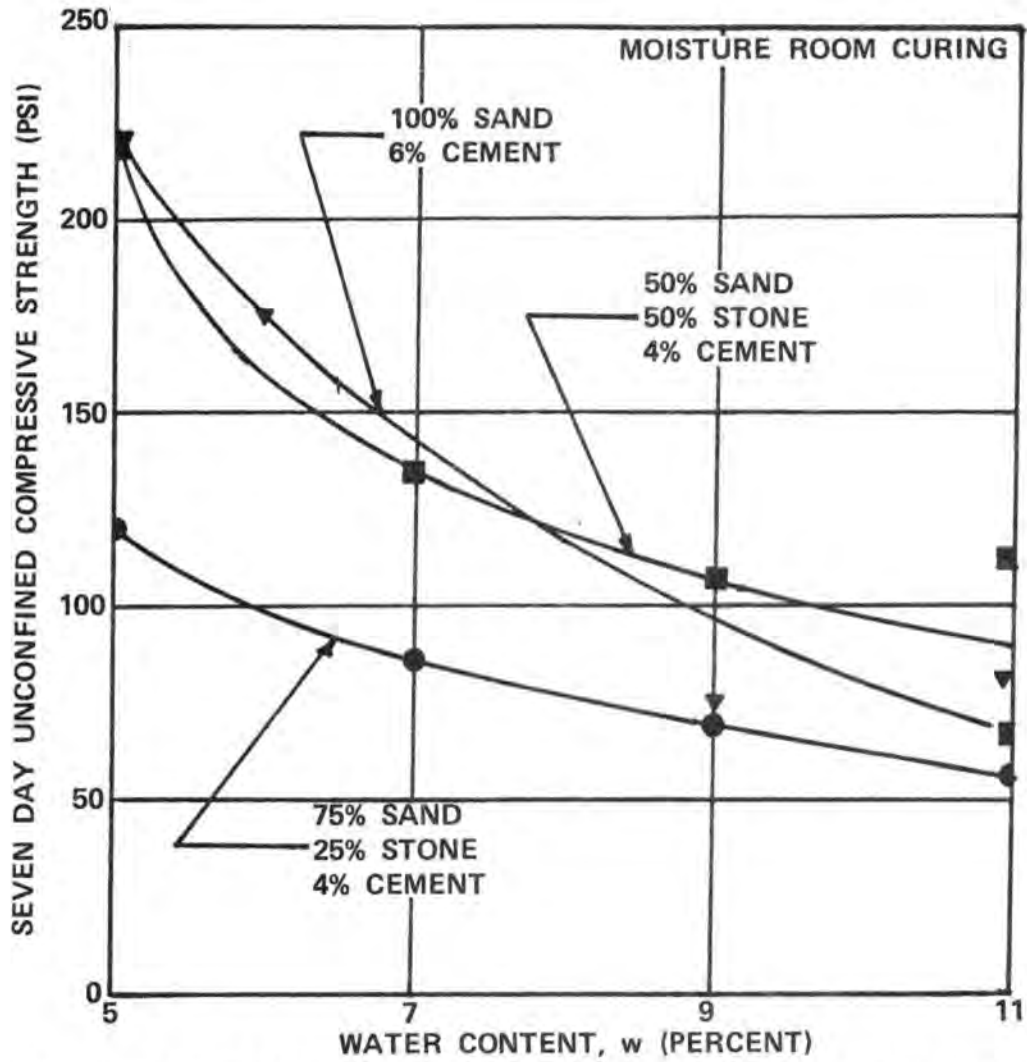


FIGURE 55. EFFECT OF MOULDING WATER CONTENT ON SEVEN DAY UNCONFINED COMPRESSIVE STRENGTH — SAND/STONE BLEND CEMENT SPECIMENS

CHAPTER 8

DISCUSSION AND DESIGN RECOMMENDATIONS

Introduction

The cost of transporting large quantities of crushed stone to the coastal plain areas of Georgia is presently quite expensive. In the future, substitution of local sands for all or part of the presently used crushed stone designs will probably become an economic necessity. Also, as the cost of petroleum products continues to rise, structural sections will have to be employed that minimize the use of expensive stabilizing agents. Finally, construction practices will have to be modified to meet strict environmental requirements. The time for innovation in pavement design now has come. The development of new structural pavement designs must be done on a sound basis by integrating the results of formal test sections, laboratory tests, and carefully performed inspections of existing pavements. The recommendations for design of sand-asphalt, sand-stone blend asphalt, and sand-cement bases given in this section are based on discussions with numerous engineers; field observations in Georgia, Florida, South Carolina and Maryland; and fatigue and rutting tests performed as a part of this investigation.

Present Sand-Asphalt Design and Construction Practices

Florida Department of Transportation

The Florida Department of Transportation has used sand-asphalt bases extensively throughout Florida and has used sand-stone asphalt blends to a much lesser extent. Some natural sands were also used in the surface (15 to 20 percent) and binder courses. A 500 lb. (2 kN) 50 blow Marshall stability mix was generally used for sand-asphalt bases, although in some areas, such as West Palm Beach, mixes were used with stabilities as low as 100 to 200 lbs. (0.45 to 0.9 kN). The air voids in sand-asphalt mixes were limited to 12 percent. For sand-asphalt bases, two sands and/or crushed stone screenings were frequently blended to meet stability and gradation requirements. When possible, a well-graded sand was used having angular grains. The rounded blow sands found in the Lake Wales area were typically blended with 50 percent screenings. Specifications allowed up to 12 percent fines, but experience has shown that usually 6 to 7 percent fines or more was required to meet stability requirements. Up to 7 percent clay size material can be used for a sand-asphalt base, although former standard specifications allowed only 4 percent clay.

In sand-asphalt bases, Florida typically used 6 1/2 to 7 1/2 percent of an AC-20 viscosity grade asphalt cement having a viscosity at 140°F (60°C) between 1600 and 2400 poises. As indicated by field experience, however, asphalt cements with viscosities in the range of 2000 to 2400

poises at 140°F (60°C) had fewer problems during laying than asphalt cements with lower viscosities. Silicone, which has been found effective in keeping the mix from tearing during laying and for use with adsorptive aggregates, was added at the rate of 1.5 ppm to the asphalt cement. The addition of more than 2 ppm silicone was found to cause problems with the mix. The Maryland and Georgia Departments of Transportation also added silicone to sand-asphalt mixes, although South Carolina has not followed this procedure.

Sand-asphalts, mixed at approximately 250°F (121°C), were laid at close to that temperature since the heat loss from the mix was generally relatively small during hauling and placing. The Florida Department of Transportation placed sand-asphalt mixes at about 92 or 93 percent of the 50 blow Marshall maximum density. Experience has shown that even this density was attained with relative difficulty in the field. Also, the use of a sand-asphalt lift thickness greater than 3 in. (75 mm) was found to result in rolling of the layer during compaction. During construction, if the sand-asphalt hung under the screed of the paving machine, the stability of the mix had to be reduced by adjusting the cold gate at the plant, changing the blend, or increasing the asphalt content of the mix. Louisiana used 2 percent mineral filler in all mixes to increase the stability and decrease the air voids. Florida is presently considering requiring 1 1/2 to 2 percent crushed stone screenings.

Pavements in Florida having sand-asphalt bases were found to show good performance (Fig. 56) with surfacing rutting usually less than 0.5 in. (12 mm). A cross-slope of 2 percent was used in Florida and no problems of ponding of water were reported. The surface cracking that developed was typically longitudinal. Because of the favorable climate and good subgrade conditions found throughout most of the state (usually a CBR of 15 to 25), relatively light structural sections were used in Florida. For pavements that endured high volumes of traffic, a 3 to 5 in. (75 to 130 mm) thick asphalt concrete surfacing mix was placed over approximately 10 in. (250 mm) of sand-asphalt base. A 12 in. (300 mm) prepared subgrade was placed below the base. For low volume roads, a 1.5 in. (40 mm) thick sand-asphalt surfacing was placed over 6 to 8 in. (150 to 200 mm) of unstabilized limerock base. An intermediate section consisted of 1.5 in. (40 mm) of asphalt concrete surfacing placed over a 4 in. (100 mm) sand-asphalt base. For shoulders on interstate pavements, a similar section was used with the sand-asphalt base being 5 in. (130 mm) rather than 4 in. (100 mm). In areas of concentrated traffic that required relatively high stability, such as in metropolitan areas, a sand-asphalt base that had a stability of 750 lbs. (3.3 kN) was sometimes specified. Usually, however, a sand-stone blend asphalt concrete mix was used to meet higher stability requirements. This type mix has presently been used on 6 or 8 jobs. Sand-stone blend asphalt concrete mixes can be placed in lifts up to 5 to 6 in. (130 to 150 mm) in thickness, and can have up to 25 percent sand.

The Marianna test sections inspected were in excellent condition after 1.2 million equivalent 18 kip axle loadings, and had an average PSI of 3.91. Only minor longitudinal cracking was observed locally in some sections (Fig. 56). The section used consisted of a 3 in. (75 mm)

thick asphalt concrete surface and binder overlaying a sand-asphalt base 4 to 8 in. (100 to 200 mm) in thickness. The test sections rested on an excellent sand subgrade. Rut depths (Fig. 57) in the sections were typically 0.25 to 0.3 in. (6 to 8 mm) at time of the field inspection with maximum observed rut depths of 0.5 in. (12 mm). The sand-asphalt base was constructed using an excellent local sand having a small amount of clay. The Marshall stability of the sand asphalt base mix on the average varied from 566 to 675 lbs. (2.5 to 3.0 kN).

The performance of sand-asphalt and limerock bases was compared at the Lake Wales Test Road. The sand-asphalt and limerock base sections both had a 1 1/2 in. (40 mm) and 3 in. (75 mm) asphalt concrete surfacing and a 3 to 10 in. (75 to 250 mm) thick base. After approximately 1.75 million equivalent 18 kip axle loads, the sections having limerock bases all were in good condition, although some longitudinal cracking was observed in the thinner sections (Fig. 58). The sand-asphalt base sections that had a 3 in. (75 mm) surfacing and 8 in. (200 mm) base were not cracked (Fig. 59), whereas the sections having a 1.5 in. (40 mm) surfacing were cracked. Moderate transverse cracking was observed in the sand-asphalt sections having 4 to 6 in. (100 to 150 mm) bases for both 1.5 in. (40 mm) and 3.0 in. (75 mm) asphalt concrete surfaces.

In the Lake Wales Test Road, after 1.75 million 18 kip axle loadings, the limerock base sections performed better than those constructed with sand-asphalt. Perhaps one factor that partially accounted for performance difference was the fact that the natural sand used in the sand-asphalt was a local blow sand known to be inferior to the more angular sands found in the northern part of the state. To improve the characteristics of the mix, therefore, these sands were blended with equal amounts of crushed stone screenings. The resulting mean Marshall stability of the mixes used in each section varied from 340 to 528 lbs. (1.5 to 2.3 kN).

Georgia Department of Transportation

The Georgia Department of Transportation has used sand-asphalt for surfacing, leveling, and base courses since about 1974. Therefore, extensive histories of the performance of sand-asphalt construction have not been developed. In the 4th District, sand-asphalt was used most often for leveling and thin overlay surfacing work. Sand-asphalt surfacing and leveling mixes are now generally used for the levels of traffic summarized in Table 26. Sand can be used in asphalt concrete surface, binder, and base mixes as long as the standard specifications are satisfied including gradation and stability requirements. The amount of local sand that can be used is limited in only the surface E mix to 30 percent.

Recently Georgia has been using an asphalt content of 5.5 to 7 percent in sand-asphalt mixes. In the Albany and Bainbridge areas, screenings were generally blended with the sand and an asphalt content of 7 to 7.5 percent was usually required. Type 1 Sand-Asphalt (SA-1) required a minimum 50 blow Marshall stability of 350 lb. (1.55 kN) while Type 2 Sand-Asphalt (SA-2) required a minimum stability of 700 lbs. (3.1 kN). Both sand-asphalt mixes required a maximum air voids content of 15 percent, a flow of 5 to 15, a 24-hour immersion compression retention of 70



FIGURE 56. MARIANNA SECTIONS AFTER 1.2×10^6 EQUIV. LOADINGS - 3 IN. SURF. AND 4 IN. LOW STABILITY SAND-ASPHALT BASE

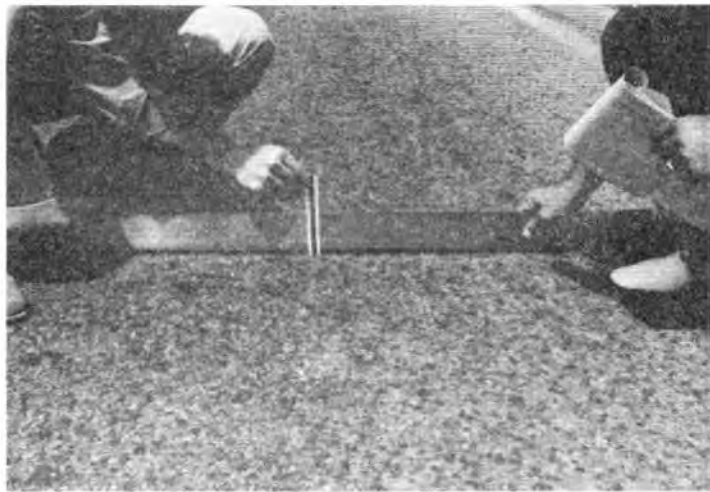


FIGURE 57. 0.25 IN. RUT IN LOW STABILITY MARIANNA SECT. - 3 IN. AC OVER 4 IN. S.A. BASE

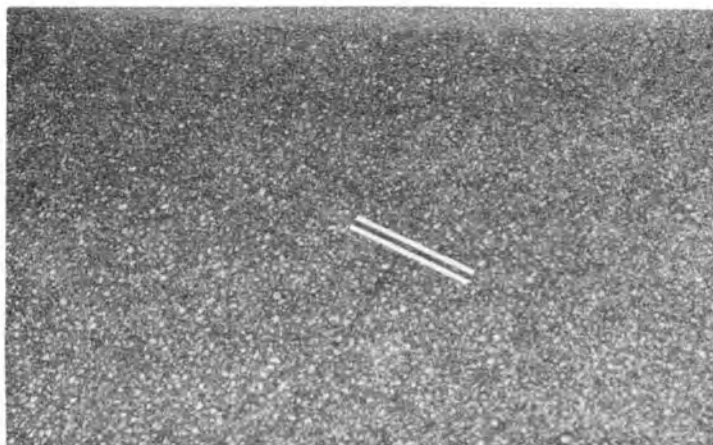


FIGURE 58. LIGHT LONG. CRACKING IN LAKE WALES LIMEROCK BASE SECTION - 1.5 IN. AC SURF. AND 4 IN. BASE AFTER 1.75×10^6 EQUIV. LOADING



FIGURE 59. LONG. CRACKING IN LAKE WALES SAND-ASPHALT BASE SECTION AFTER 1.75×10^6 EQUIV. LOADS - 3 IN. AC SURFACE AND 6 IN. BASE

Table 26 . Practice Presently Usually Followed by the Georgia Department of Transportation for the Use of Surfacing and Leveling/Patching Sand-Asphalt Mixes

| Vehicles Per Day | Truck Traffic (Percent) | Allowable Mixes |
|------------------|-------------------------|--|
| 0 - 499 | < 7 | (1) SA-1 for surface and leveling/patching |
| 0 - 499 | > 7 | (2) SA-2 for surface and patching |
| 500 - 999 | < 7 | SA-2 for surface and leveling/patching |
| 500 - 999 | > 7 | Sand asphalt surface not permissible; use G or H mix for leveling and patching |
| 1000 - 1999 | any | Sand asphalt not permissible; Use G or H mix for leveling and patching |
| 2000 | any | Use H, F, E, modified B or D mix. |

NOTES:

1. Sand-Asphalt Mix SA-1: 5.5 to 7.0% A.C., 5 to 16 flow, 50 Blow Marshall Stability of 350 lbs.
2. Sand-Asphalt Mix SA-2: 5.5 to 7.5% A.C., 5 to 16 flow, 50 Blow Marshall Stability of 700 lbs.

percent, and 95 percent unstripped aggregate. The sand equivalent required was 25, although if blending was performed, the sand equivalent of the natural sand could have been as low as 20. Some problems with clay balling were reported with sands that had sand equivalents in the vicinity of 20 to 22 when a drum mixer was used. In conventional asphalt plants, the clay balls were screened out and have not caused any problems. Gradation specifications for the sand required that 100 percent pass the No. 50 sieve and between 2 and 20 percent pass the No. 200 sieve.

Experience in Georgia has shown that a dirty sand with approximately 4 to 7 percent clay was probably best, provided the clay broke down and did not form balls during mixing. Silicone was added to the sand-asphalt mix which reduced the problem of tearing during placement.

Sand-asphalt base courses were placed in 2 in. (50 mm) maximum-lift thicknesses with a total thickness of usually 6 in. (150 mm). Leveling courses were placed in 1 in. (25 mm) lift thicknesses with a maximum total thickness of 2 in. (50 mm). The sand-asphalt mix was generally laid at 280 to 300°F (140 to 150°C), although mixes were sometimes placed at temperatures as low as 240°F (116°C). Some problems with obtaining a good bond of the sand-asphalt was experienced. As a result, specifications now require either an SS-1 or AC tack coat, although the AC tack coat is preferred by some engineers.

At the present time, the in-place costs of sand-asphalt leveling, resurfacing, and base course mixes are on the average \$0.80, \$0.78, and \$0.86 per square yard per inch of mix; average prices for corresponding asphalt concrete mixes are \$0.91, \$0.89, and \$0.83/yd²/in. of mix. These figures are for average project sizes of about 2,000 tons for the leveling and base course mixes and 4,000 tons for the surface mixes. It is surprising that bid sand-asphalt prices in Georgia are only slightly less than asphalt concrete for the resurfacing and leveling mixes and more expensive for the base course mix. These high bid prices are not consistent with those reported by other states and with the actual material costs. As more experience with sand-asphalt mixes is gained by local contractors, the bid prices of this type mix should significantly decrease.

When alternate bids for sand-asphalt and soil cement bases have been obtained for the same project, soil cement generally has been found to be less expensive which is in agreement with the findings in Maryland. A 7-day unconfined compressive strength of 300 to 350 psi (2100 to 2400 kN/m²) has been specified generally. The Moultrie By-Pass used a 6 in. (150 mm) sand-cement base that had a 5 percent cement content. In addition, a number of streets have used 6 in. (150 mm) sand-cement bases underlying typically a 1.5 in. (38 mm) E mix surface course. A single surface treatment layer has been frequently placed over the sand-cement base to obtain good bond between the base and surface course.

Maryland Department of Transportation

The Maryland Department of Transportation typically has placed 5 to 7 in. (130 to 180 mm) of asphalt concrete over a 6 in. (50 mm) sand-asphalt

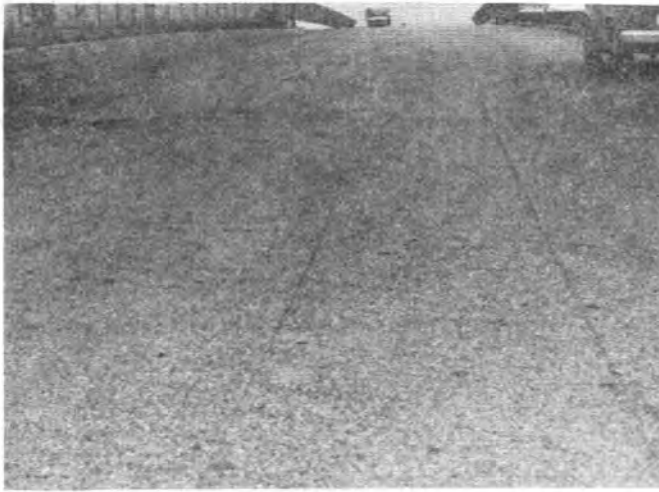
base. Sand-asphalt bases have not been used usually on either interstate or secondary road construction. Specifications allowed the use of either a natural sand, screenings, or sand-aggregate blends. The only gradation requirement was that not more than 12 percent pass the No. 200 sieve. Also, the sand-asphalt base mixes were required to have a 50 blow Marshall stability of not less than 250 lbs. (1.1 kN) with flows less than 16. The sand-asphalt base was placed at a density of 95 percent of the 50 blow Marshall value. An asphalt content of typically 4.2 to 4.8 percent was used in sand-asphalt mixes by the Maryland Department of Transportation. Good performance has been found from this type construction (Fig. 60). In some adsorptive sands, crushed stone screenings have been blended with the sand. A cross-slope of 1.6 percent was used in Maryland. Presently, the approximate in-place cost of sand-asphalt bases is about \$0.85/sq.yd./in. compared to \$0.59/sq.yd./in. for soil cement (not including a curing compound which is always used) and \$0.51/sq.yd./in. for crushed stone.¹

To overcome problems experienced with separation between 3 in. (75 mm) layers, Maryland began to use either one 6 in. (150 mm) layer or an increased asphalt content by approximately 1/2 percent [70]. The sand-asphalt was mixed at a temperature of about 300°F (149°C) and rolling began at about 260° to 270°F (77° to 132°C). The mix was compacted using a roller speed of about 1.5 to 2.0 mph (0.9 to 1.2 km/hr.) in low gear. A 6 in. (150 mm) deep lift construction was found to be superior to placing the mix in a 3 in. (75 mm) deep lift in regard to higher densities and better heat retention characteristics. Although the roughness of 3 in. and 6 in. (75 and 150 mm) bases was about the same (170 to 200 in./0.1 mile), the 6 in. (150 mm) sand-asphalt base apparently resulted in more grade control problems than the thinner sections. In the 3 in. (75 mm) lift 94 to 96 percent of the 50 blow Marshall density was obtained, while in the 6 in. (150 mm) lift, densities were about 97 to 100 percent. Of interest is the fact that the compaction energy required to obtain these densities in the 3 in. and 6 in. (75 and 150 mm) lifts was approximately the same.

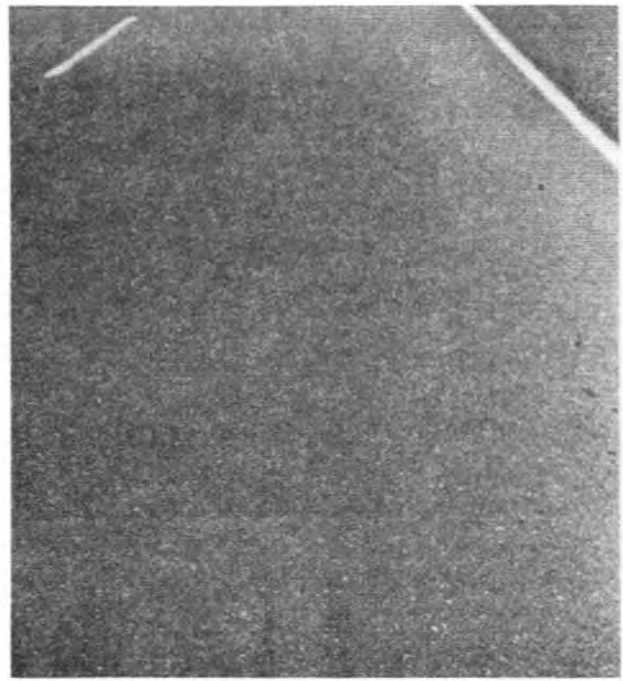
The following rolling pattern was developed to give the best compaction qualities of the sand-asphalt base mix studied [70] based on observed rutting and displacement of the surface:

1. An initial stabilizing pass consisting of one pass with a 10 ton (88 kN), unballasted Buffalo Springfield steel wheel roller. Lift densities were increased from 80 to 87 percent of the 50 blow Marshall value in the 3 in. (75 mm) lift, and from 72 to 87 percent in the 6 in. (150 mm) lift.

¹These figures are based on average 1975 bid prices which have been increased by 17 percent to reflect 1977 price levels. The price given for crushed stone is for short to moderate haul distances and would be low for projects located a long distance from a quarry.



(a) Dover Road, Baltimore - 6.5 in. AC Surf. and 5 in. S.A. Base, Heavy Truck Traffic

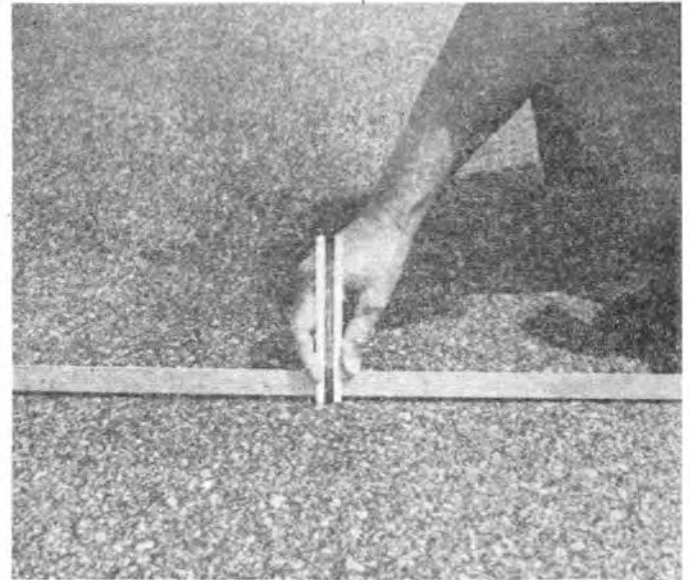


(b) Moderate Long. Cracking in M-100 After 7 yrs. - S.A. Base Under 7 in. AC

FIGURE 60. MARYLAND SAND-ASPALT BASE PAVEMENTS WITH 4 to 4.5 PERCENT ASPHALT



(a) General View: 8 in. AC Overlying 8 in. S.A. Base



(b) Rut Depth of 0.4 in.

FIGURE 61. I-20 EAST OF COLUMBIA, S.C. AFTER 1.2×10^6 EQUIV. LOADS: SAND-ASPALT BASE WITH APPROX. 4.5% ASPHALT, NO CRACKING

2. Two passes with an unballasted, pneumatic seven-wheel Bros SP 6000 pneumatic roller having a 40 psi (275 kN/m²) tire pressure (dry).
3. One smoothing pass of a 10-ton wheel roller.
4. Two passes with the unballasted pneumatic tired roller at 75 psi (500 kN/m²) immediately followed by a second 10-ton (88 kN) steel wheel roller to finish the surface.

Soil cement construction has not been used very extensively in Maryland since some problems with reflection cracking have been encountered with this type construction. In-place mixing has been used in the most recent soil cement jobs. A 7-day unconfined compressive strength of 450 psi (3,000 kN/m²) has been required. Typically, a cement content of approximately 7 percent by weight has been used. Generally, a 6 in. (150 mm) base course has been constructed and a curing surface treatment placed over it.

On the eastern shore of Maryland, an unstabilized mixture of sand and slag has been used sometimes for base course construction. Approximately 30 to 40 percent slag with 100 percent that passed the 1 in. (25.4 mm) sieve and 3 to 5 percent that passed the No. 4 sieve was blended with sand in a plant mix operation. This sand-aggregate blend was compacted to 100 percent of AASHTO T-180 density. Usually 7 lbs. (30 N) of calcium chloride was added at the pugmill to the top coarse of the sand-aggregate. The addition of calcium chloride was commonly practiced in Maryland for sand-aggregate and dense graded stone bases. A field study [16] has shown that these unstabilized sand-aggregate bases performed exceptionally well (Fig. 8).

Sand-asphalt base mixes have been frequently used for subdivision streets, parking lots, and other private work in Maryland. Usually a 1.5 in. (40 mm) asphalt concrete surfacing was placed over a 5 in. (130 mm) sand-asphalt base. Very loose gradation requirements were used with this material. A dirty sand was used to keep the required quantity of asphalt cement down. Typically, 4 percent asphalt-cement was used in the mix. The plant was run on the cold side at 250 to 275°F (120 to 135°C), and a mixing time was used of 30 to 35 sec. After mixing, the aggregate sometimes appeared brown and not well coated with asphalt. Although this mix was indeed dry, coating of the aggregate has been found to occur during the rolling operation. This low asphalt content sand-asphalt was compacted immediately after placement using a rubber-tired roller followed about 200 ft. (60 m) by a steel wheel roller. Six in. (150 mm) of this type sand-asphalt base mix was assumed locally to be equivalent to 4 in. (100 mm) of 1.5 in. (40 mm) maximum size crushed stone black base.

South Carolina Department of Highways and Public Transportation

The South Carolina Department of Highways used sand-asphalt extensively for bases in the Coastal Plain, but sand-cement is not presently used in this area. An AC-20 viscosity grade asphalt cement usually was

used in sand-asphalt mixes with asphalt contents varying from 4.2 to 4.8 percent. Substitution of local sands for the finer portions of surfacing and binder mixes has also been permitted in South Carolina. For sand-asphalt base mixes, essentially the only gradation specification required that the sand have less than 12 percent fines, as determined by washing, with up to 6 percent clay as determined by the elutriation test. Although a 300 lb. (1.3 kN) Marshall stability mix has been used for most work, a stability of 500 lbs. (2.2 kN) has been used on some primary and interstate construction. In some instances, sand-asphalt mixes were used and consisted entirely of crushed stone screenings that had a sand equivalent greater than 35. The sand-asphalt was mixed at temperatures from 250 to 325°F (121 to 163°C) with a maximum reduction in temperature of 20°F (11°C) at the time of rolling. No specification requirements were placed on either density or rolling procedures. South Carolina has experienced rutting problems in some sections with reported rut depths in the worst case of 1 to 1.5 in. (25 to 40 mm) on several roadways that used sand-asphalt bases. Undoubtedly these rutting problems were caused partially by the lack of field density control and perhaps by the use of low stability sand-asphalt bases up to 10 in. (250 mm) in thickness.

On interstate work, a structural section has been used consisting of a 2 in. (50 mm) asphalt concrete surfacing, 4 in. (100 mm) asphalt concrete binder, and 6 in. (150 mm) sand-asphalt base. Previously, thicker sections were used that typically consisted of 3 in. (75 mm) of asphalt concrete surfacing, 5 in. (130 mm) of asphalt concrete binder and 8 in. (200 mm) of sand-asphalt base. Only A-4 soils or better have been used in the top 18 in. (460 mm) of the subgrade for interstate work. On primary roadways, a section often used at the present time consists of a 1 1/2 to 2 1/2 in. (40 to 60 mm) asphalt concrete surfacing, 2 1/2 in. (60 mm) asphalt concrete binder, and a 6 in. (150 mm) sand-asphalt base. Sand-asphalt seldom has been used for new construction on lightly traveled roads in South Carolina. Sand-asphalt has been used as a thin surface overlay on existing secondary roads with the overlay thickness typically varying from 0.75 to 0.8 in. (19 to 20 mm). Many pavements constructed with sand-asphalt bases have performed satisfactorily, such as I-20 (Fig. 61). One section on I-20 was observed to be in excellent condition after 1.2 million equivalent 18 kip axle loads (one direction). Cracking was not observed in this or similar sections although rut depths measured with a 4 ft. (1.2 m) straightedge were typically 0.25 to 0.4 in. (6 to 10 mm). This section consisted of 8 in. (200 mm) of asphalt concrete overlaying an 8 in. (200 mm) sand-asphalt base. No problems with ponding of water were reported on I-20 which had a cross-slope of 1.67 percent.

Typical average costs of bituminous construction reported in the coastal plain area of South Carolina were as follows: Sand-asphalt: \$0.55/sq.yd./in.; Crushed stone base: \$0.41/sq.yd./in.; and Black Base: \$0.80/sq.yd./in. These prices were the in-place cost of the material and did not include the cost of the asphalt cement which usually varied from 4.2 to 4.8 percent.

General Considerations for Sand-Asphalt Base Construction

Fatigue Performance

The relative beneficial effect of increasing the asphalt content and decreasing the void content on the typical sand-asphalt and sand-stone mixes studied in the lab is shown in Figs. 62 and 63, respectively. Increasing the asphalt content from 5 to 6 percent can double the fatigue life, while going from 6 to 7 percent can increase the fatigue life by a factor of approximately five. Approximately the same beneficial effect of increase in asphalt content on relative fatigue life has been observed for mixes with both high and low Marshall stabilities. Increasing the asphalt content can significantly increase the fatigue resistance of a sand-asphalt base mix and can be achieved relatively easily and controlled in the field.

As shown in Fig. 63, laboratory studies that decreased the void content a small amount resulted in very important increases in fatigue life. For example, a decrease from 16 to 12 percent air voids would potentially double the life of a mix if other factors remained the same. The laboratory test results shown in Figs. 62 and 63 indicated that to maximize the fatigue life of a sand-asphalt or sand-stone blend, a mix should be developed that has a low air void content and the highest practical asphalt content that would not result in excessive rutting of the pavement. The test results also suggest that for a given asphalt and air void content, mixes with higher Marshall stabilities probably have potentially greater fatigue lives. The test results suggested this to be true in both the stability range from 1000 to 2000 lbs. (4.4 to 8.9 kN) which corresponded to low air void contents and for stabilities from 460 to 600 lbs. (2 to 2.7 kN) which corresponded to high air void contents. The observed general trend of increased fatigue life for a given air void content for mixes with higher Marshall stabilities was based on the analysis of fatigue test results for a limited number of mixes. Additional tests need to be performed to verify that the observed trend is valid for a wide range of mixes. The general relationship observed between fatigue life and Marshall stability disregarding the effect of air void content is shown in Fig. 64 for an asphalt content of 6 1/2 percent. The test results indicated a definite relationship between increased fatigue life and increased Marshall stability. Variables that had the primary effect on fatigue life, however, appeared to be asphalt content and air voids with Marshall stability playing a secondary role.

The fatigue performance of sand-asphalt mixes can be evaluated using the beam fatigue test described in Chapter 6. In general, however, the fatigue life of a mix can be adequately controlled primarily by limiting the air voids, using a reasonably high percent of asphalt in the mix, and controlling the Marshall stability. In special instances where aggregates may be weak or fracture readily (such as shells), beam fatigue tests should be used to evaluate the fatigue resistance of the mix. As a simpler alternative method for evaluating such mixes, an unsupported static flexure test could be performed on the suspect mix and the results compared with mixes having known performance records.

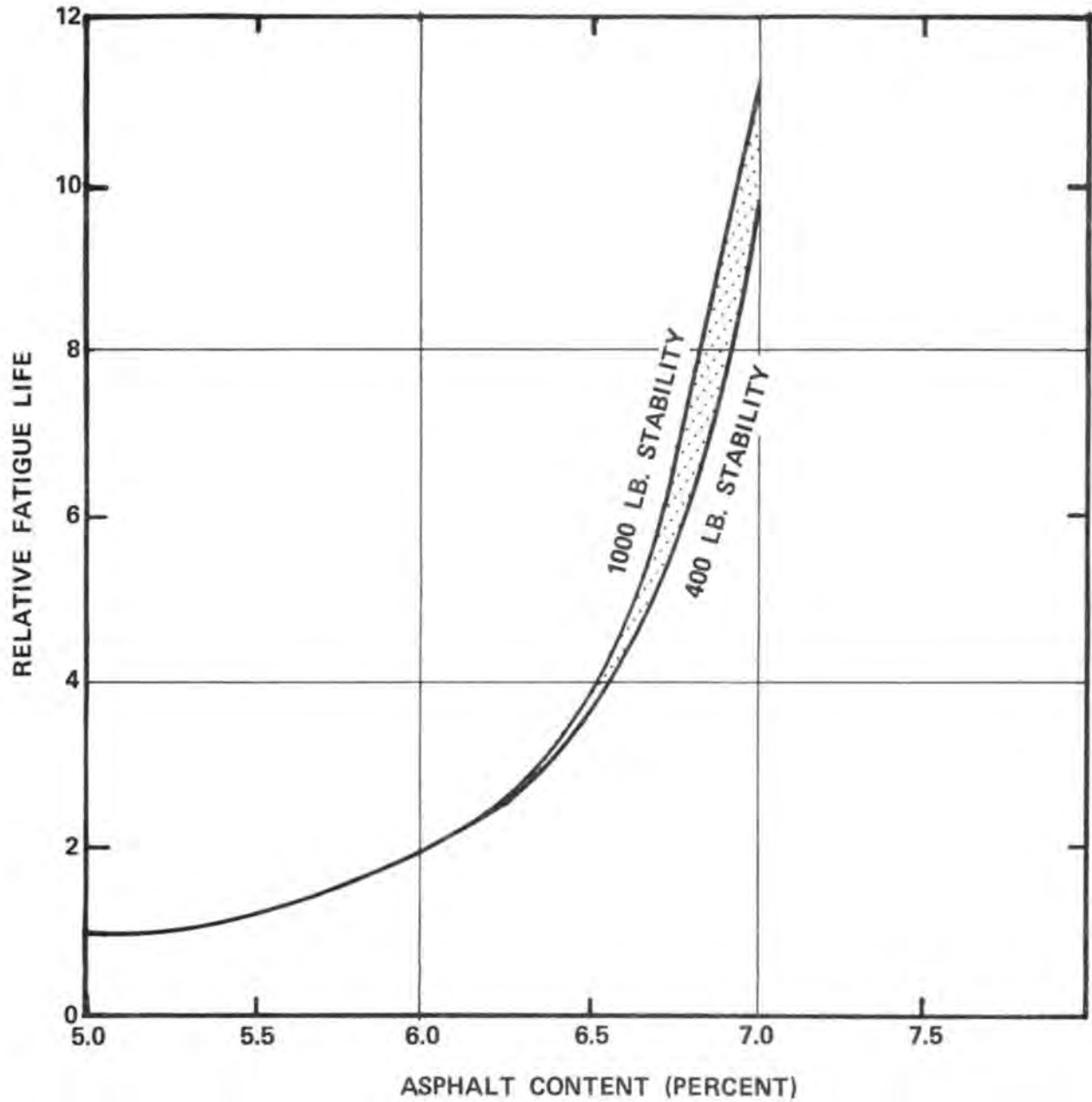


FIGURE 62. RELATIVE EFFECT OF INCREASING ASPHALT CONTENT ON FATIGUE LIFE OF SAND AND SAND/STONE ASPHALT MIXES

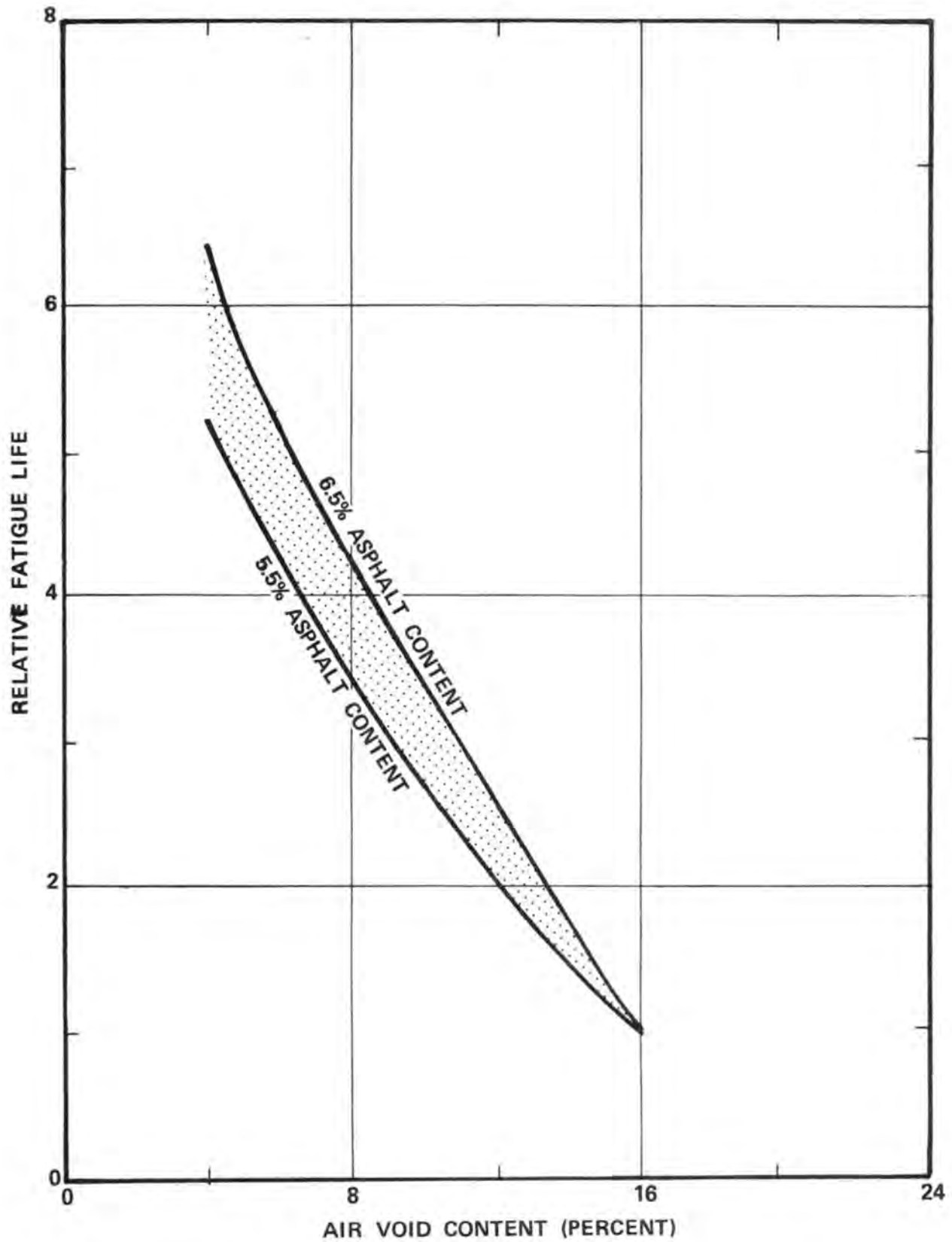


FIGURE 63. RELATIVE EFFECT OF AIR VOID CONTENT ON FATIGUE LIFE

With time, sand-asphalt mixes constructed with high initial air void contents will undergo compaction due to traffic. A method for evaluating the beneficial effect of the application of heavy truck traffic and compaction of asphalt concrete has been developed by Barksdale [6]. Following this approach, assume that the combined effects of traffic compaction and aging of a typical sand-asphalt base mix can be approximated by an effective transition period of four years [6,54]. Now consider two similar sand-asphalt mixes that originally were compacted to 14 and 16 percent voids. Due to traffic compaction, assume these mixes will eventually reach a limiting equilibrium void content of 12 percent. The corresponding fatigue life curves for the two mixes constructed using the graphical approach developed by Barksdale [6] which approximately considers the effects of traffic and weathering are shown in Fig. 65. The sand-asphalt mix with 14 percent air voids has a fatigue life ratio of 1.4. The fatigue life ratio for the mix with 14 percent voids is defined as the ratio of the laboratory fatigue life of a mix with 12 percent voids (the assumed limiting air void content) to that of a mix with 14 percent voids. The fatigue life ratio of the mix with 16 percent voids is defined similarly and is 2.4. The fatigue life lines shown in Fig. 65 shows the remaining fatigue life for each mix in terms of number of remaining wheel load repetitions as a function of time.

Considering only the laboratory fatigue test results and not the effects of traffic compaction, a sand-asphalt mix compacted to 14 percent voids would have a theoretical fatigue life of approximately 7 years compared to 4 years if the mix initially had 16 percent voids. The field fatigue life of these same two typical sand-asphalt mixes having 14 and 16 percent voids, using the graphical procedure, would be approximately 8.7 and 6.8 years, respectively. These fatigue lives should be compared to a maximum limiting fatigue life of 10 years for a mix originally compacted to the limiting 12 percent air void content that should ultimately be reached by the other mixes due to traffic compaction. The detrimental effects of an initial high air void content in the mix, therefore, are not nearly as great as indicated by the laboratory fatigue test results, which do not consider the significant beneficial effect of heavy traffic on fatigue life. Nevertheless, a 2 percent decrease in initial air voids content can increase the fatigue life of the mix by as much as 25 percent even when the traffic compaction effects are considered.

The important beneficial effect of traffic compaction on mixes having initially high air void contents partially explains the reasonably good performance of sand-asphalt bases constructed with very little field control on sand-asphalt density which is practiced by the South Carolina Department of Highways. South Carolina typically has used 4 to 8 in. (100 to 200 mm) of asphalt concrete above the sand-asphalt base which also would help to account for the reasonably good performance observed. Any sand-asphalt mix constructed at a high air void content would show greater rutting and less fatigue life than the same mix compacted to a higher density and hence lower air void content. Considering these factors, therefore, sand-asphalt mixes should be designed and initially compacted to as low an air void content as practical (high density). When a high density is not achieved in the field, however, the mix still will exhibit a reasonably large fatigue life due to the important beneficial effects of heavy traffic.

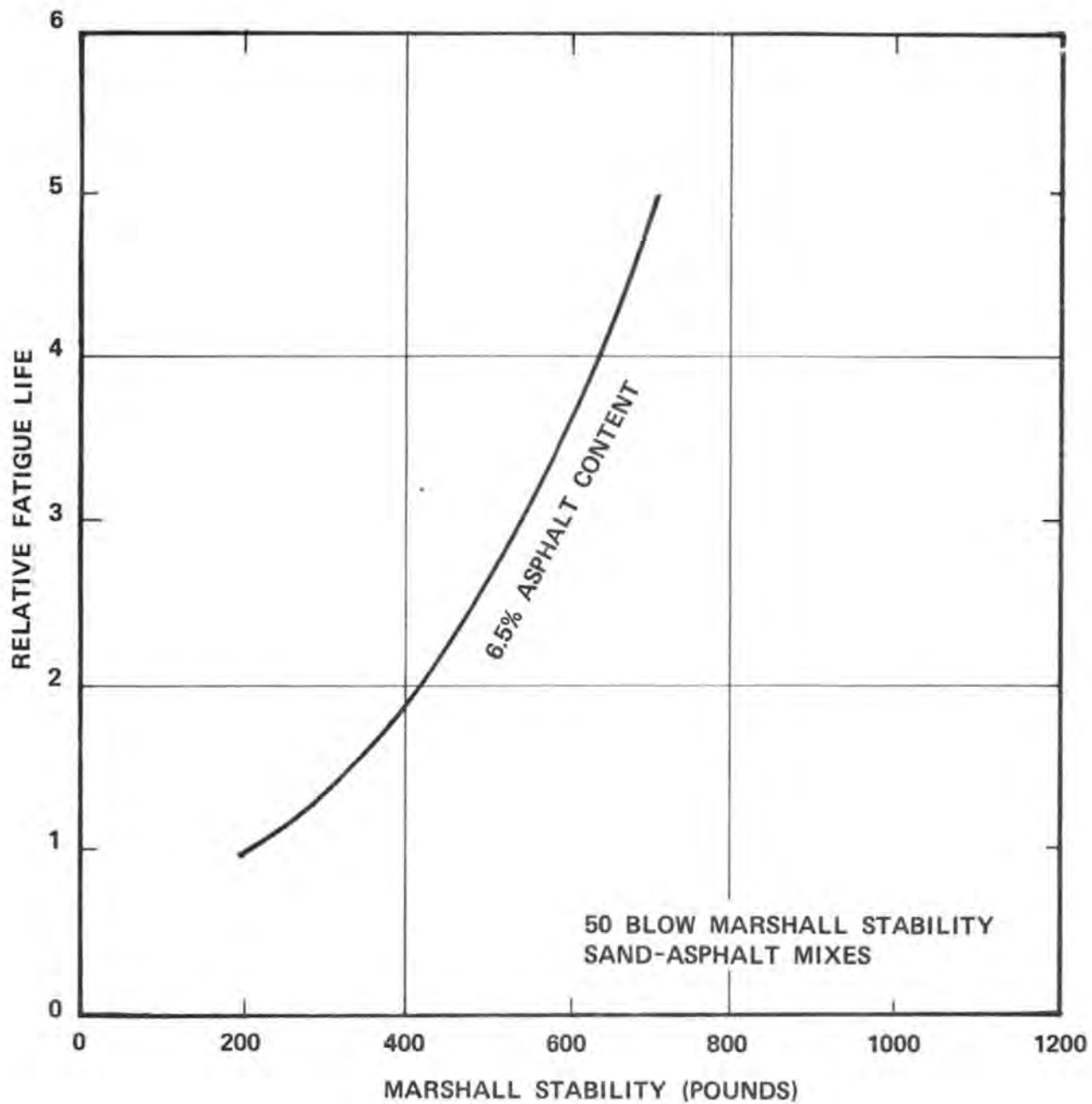


FIGURE 64. RELATIVE INFLUENCE OF INCREASING MARSHALL STABILITY ON FATIGUE LIFE OF SAND AND SAND/STONE ASPHALT MIXES

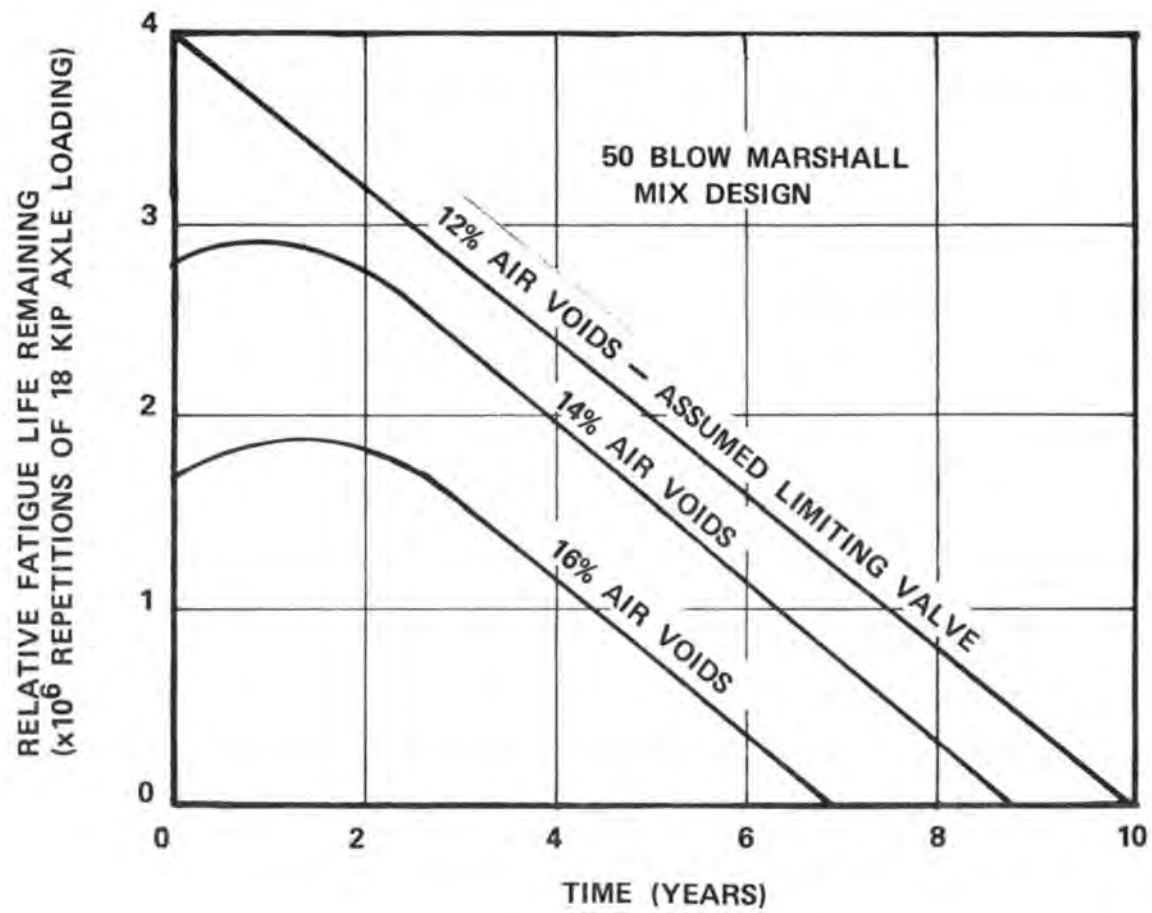


FIGURE 65. EFFECT OF TRAFFIC COMPACTION ON FATIGUE LIFE OF MIXES HAVING DIFFERENT AIR VOIDS

Rutting Performance

To exhibit good fatigue life, a sand-asphalt mix should have a relatively high asphalt content. As the asphalt content increases, however, the resulting rutting in the mix also increases. Field experience shows that rutting in sand-asphalt mixes containing up to 7 percent asphalt content generally does not cause problems. Therefore, if reasonable caution is exercised during design and construction with respect to rutting as discussed in this chapter, rutting should not be a serious problem.

Important variables affecting rutting in sand-asphalt and sand-stone blends are asphalt content, Marshall stability (or air void content), and the characteristics of the aggregate. Presently, the specific basic characteristics of the mix affecting rutting are only partially defined. Undoubtedly, the aggregate top size and gradation, angularity, and amount and type of fines combine to influence the frictional resistance of the mineral skeleton and hence the susceptibility of the mix to rutting. In a sand-stone blend, the percent stone appears to be reflected in increased stability with increased stone content.

An increase in asphalt content from 5.5 to 6.5 percent should increase the tendency to rut by 40 to 70 percent. Changing from a conventional crushed stone black base mix to a pure sand-asphalt mix at the same asphalt content should increase rutting in the base by about 50 to 100 percent or more. The amount of rutting in a mix can be significantly reduced and the fatigue life increased if a sand-stone blend is used, as illustrated in Figs. 50, 51, and 52. Sand-stone blends should optimize the use of locally available materials while minimizing the poorer fatigue and rutting characteristics of sand-asphalt mixes. Blends that contain 15 to 50 percent sand should retain most of the desirable characteristics of a crushed stone mix and should utilize a significant quantity of local materials while reducing the overall cost of the mix. Previous studies indicated that Marshall stability and air voids in the mix were found to be related to rutting. For similar sand-stone mixes (i.e., mixes that use the same materials in different proportions), the relationship between rutting and Marshall stability and air voids was relatively well defined. Since a reasonably well defined relationship was found between Marshall stability and air void content (Fig. 48), either one of these variables could be used as a preliminary guide for limiting rutting.

Marais [24] found that the ratio of Marshall stability to flow was a more reliable indicator of the rutting characteristics of mixes than Marshall stability alone. Because of the relatively small changes of flow with asphalt content in the tests performed in this investigation, the stability to flow ratio was not found to be as good an indicator of rutting as Marshall stability (or air voids) when considered with the asphalt content. Other mixes, however, might show a better correlation between rutting and the stability to flow ratio.

The theoretical method proposed in Chapter 3 for estimating rutting in pavements that contain sand-asphalt and sand-stone blends should be used when a reasonably reliable estimate of rut depth is required. A

preliminary estimate of the susceptibility of a base mix to rutting can be obtained from the generalized design relationship given in Fig. 66. This relationship uses the test results and the theoretical procedure summarized by equation (15). Rut depth in the figure is given as either a function of asphalt content and 50 blow Marshall stability of the mix or as a function of asphalt content and air void content. The rut depth in the base is given for a pavement section having a 3.5 in. (90 mm) thick asphalt concrete surfacing and a 7 in. (180 mm) thick sand-asphalt or sand-stone blend base ($\lambda_1 = 3.0$). The pavement is assumed to rest on a fair subgrade ($E = 4,000$ psi) and the rut depths given are for approximately 1,200,000 equivalent, 18 kip single axle loads. The total rut depth of the section is obtained by adding the rut depth in the sand-asphalt base obtained from the figure to that which occurs in the surfacing. Estimates of rut depth in the 3.5 in. (90 mm) surfacing for from 4.5 to 6.5 percent asphalt content mixes are shown on the figure. As a simplification, for small variations in surface course thickness, the rut depth is assumed to be independent of base thickness.

The scatter in data upon which the chart was prepared was reasonably large, so the rut depths obtained from the figure should be considered as only rough, preliminary estimates. An indication of the accuracy can be obtained by estimating the rutting using both the measured Marshall stability and the air voids content. The proposed preliminary design chart overpredicts the rut depths observed at the Marianna Test Road (0.41 in. calculated compared with 0.30 in.) and at the Lake Wales Test Road (0.51 in. compared with a measured value of 0.47 in.).

Allowable Rut Depth

The allowable rut depth which a pavement can undergo is controlled by both safety and structural considerations. If a sufficient amount of water ponds in a rut, hydroplaning or loss of skid resistance will occur. The amount of rutting that occurs before ponding depends upon the cross-slope of the pavement and upon the transverse width of the rut. On a rolled asphalt construction in England, Lister and Addis [58] found rut depths greater than approximately 0.5 in. (13 mm) resulted in the ponding of water on pavement that had a 2.5 percent cross-slope. Lister and Addis also found that the optimum time for overlaying a rolled asphalt pavement corresponded to a rut depth of approximately 0.4 in. (10 mm) measured with a 6 ft. (1.8 m) straightedge. The 0.4 in. (10 mm) rut depth is the limiting value of rutting before loss of structural strength starts to occur. In the United Kingdom, a rut depth of 0.75 in. (19 mm) is generally defined as pavement failure.

The PSI value of sections at the AASHO Road Test were found by Lister and Addis [53] to be inversely proportional to rut depth (Fig. 67). This relationship, together with estimated rut depths, can be used to indirectly estimate the corresponding PSI value of pavements having sand-asphalt and sand-stone blend bases. Consider, for example, the average conditions at the Marianna Test Road. For all sections, the average rut depth in 1972 after 8 years was 0.27 in. (7 mm), and the average PSI value was 3.91. From Fig. 66, the PSI value corresponding to a rut depth of 0.27 in. (7 mm) measured with a 6 ft. straightedge (1.8 m) would

be 3.65 compared with the observed value of 3.91 giving a predicted PSI value slightly on the safe side. At the Lake Wales test road, the average rut depth in the sand-asphalt base sections was 0.43 in. (11 mm). The corresponding PSI value obtained from the relationship developed by Lister and Addis was 3.2. The above two examples suggest that the chart relating PSI and rut depth developed by Lister and Addis may be conservative for pavements with sand-asphalt bases.

The field inspections made during this investigation showed that the rutting in sand-asphalt base pavements developed was relatively wide which possibly accounted for the higher than expected PSI rating. To take into consideration the width of the rut, Verstraeten *et al.* [65] developed rut criteria for use in Belgium based on the transverse slope of the rut. For four-lane interstate and primary type highways, they recommended that the rut slope $\delta/\Delta\delta$ should not exceed 0.02, and for two-lane primary and secondary highways the rut slope should not exceed 0.03. The rut slope $\delta/\Delta\delta$ is defined as the maximum rut depth divided by the half width of the associated rut. For a 6 ft. (1.8 m) wide rut, the width usually used to measure rut depth, the maximum allowable depth would be 0.72 in. (18 mm) for interstate and primary type pavements and 1 in. (25 mm) for secondary roadways.

For highways in Switzerland, Huschek [66] proposed a 4 mm limiting water film on the surface. To satisfy this criteria, Huschek indicated that the rut depth must be less than 0.7 in. (18 mm) for a 2.5 percent cross-slope, which agreed with the rut depth criteria proposed by Verstraeten *et al.* [65]. Huschek concluded that a limiting rut depth of 0.7 in. (18 mm) corresponded to a PSI value of about 3.1, a somewhat higher value than that measured at the AASHO Road Test.

In the United States, the average rut depth has been found in a nationwide survey to be approximately 0.18 in. (6 mm) on primary and interstate highways using conventional asphalt concrete construction. Therefore, in the past, rutting has not been a nationwide problem, and in some areas, rut depths greater than 0.25 in. (6 mm) have been considered to be excessive. In Europe, however rutting has been the primary distress mode for many years. Of significance is the fact that allowable rut depth criteria were developed in Europe based on safety considerations and varied from 0.5 to 1 in. (12 to 25 mm). From the work of Lister and Addis [58] and from a structural viewpoint, a rut depth of 0.4 in. (10 mm) is the optimum time to overlay a rolled asphalt pavement that has rutting as the failure mechanism. In general, rolled asphalt pavements would be more flexible than conventional asphalt concrete mixes used in the United States. The hypothesis can be made, therefore, that for conventional pavements, the optimum rut depth for overlaying should be somewhat less than for rolled asphalt pavements. For mixes using asphalt contents greater than about 6 percent, the overlay criteria proposed by Lister and Addis is probably reasonably valid.

In the future, use of sand-asphalt mixes will require that more realistic rut depth criteria be applied to pavement design. For the present time, an allowable average design rut depth of 0.4 in. (10 mm)

is recommended for primary and interstate pavements and 0.6 in. (15 mm) for secondary roads constructed using high asphalt contents and/or sand-asphalt mixes. If necessary, an allowable rut depth of 0.5 in. (13 mm) could be used for primary and interstate highways and 0.75 in. (19 mm) for secondary roads. Rut depths up to 0.59 in. (15 mm) were measured at the Marianna Test Road. Rut depths of 0.5 in. (13 mm) or slightly higher also were measured at the Palm Beach, Lake Wales, and Crestview Test Roads in Florida [61]. No problems due to this level of rutting were reported. The rut depth profile developed in pavements with sand-asphalt bases extended laterally over a relatively large width so that the rut slope was reasonably small and not easily noticed.

Structural Thickness Design

At the present time, the required structural section can be most readily determined using the AASHTO Interim Guide [18]. Of course, other more mechanistic design methods based on the fundamental fatigue and rutting modes of distress could be used. When sand-asphalt, sand-stone blends and/or high asphalt content courses are to be used in important projects, rutting can be limited to a tolerable level using the method developed in Chapter 3 for calculating rut depths. A preliminary estimation of the level of rutting can be obtained by using Fig. 66.

Based on past experience (Fig. 67), a pavement having a rut depth of 0.7 in. (18 mm) will have a PSI of approximately 2.5, while one with a rut depth of 0.5 in. (12 mm) will have a PSI value in the vicinity of 3.0. Therefore, from a practical viewpoint, the structural design of pavements having sand-asphalt bases should be based on a PSI of not less than 2.5. For pavements using thick sand-asphalt layers and/or sand-asphalts that may undergo excessive rutting, justification can be given for using a PSI design value of 3.0.

Following the AASHTO Interim Guide design approach, the required weighted structural number, SN, can be obtained from the nomograph given in Fig. 68. The design nomograph given is for a PSI value of 2.5 which should give satisfactory design for most type construction and materials. Use of this nomograph to obtain the weighted structural number, SN, requires knowledge of the soil support value, S, which indicates the load-carrying characteristics of the subgrade. Figure 69 can be used as a general guide in estimating the soil support value. A conservative value of the laboratory test results should be used in estimating the soil support value. An analysis of the results of a field performance study in Maryland and also past experience indicated that the soil support value based on the CBR value is often too high. Therefore, caution should be exercised in selecting a conservative soil support value. The soil support values given in Table 27, which are based on the AASHTO soil classification, can be used as a general guide in establishing upper limiting soil support values. The equivalent number of 18 kip single axle loadings, also required in the AASHTO design procedure, can be estimated using Table 28. Finally, the regional factor, R must be estimated (Fig. 70). The regional factor attempts to account for the variations in climatic factors including freeze-thaw, rainfall, and other environmental effects.

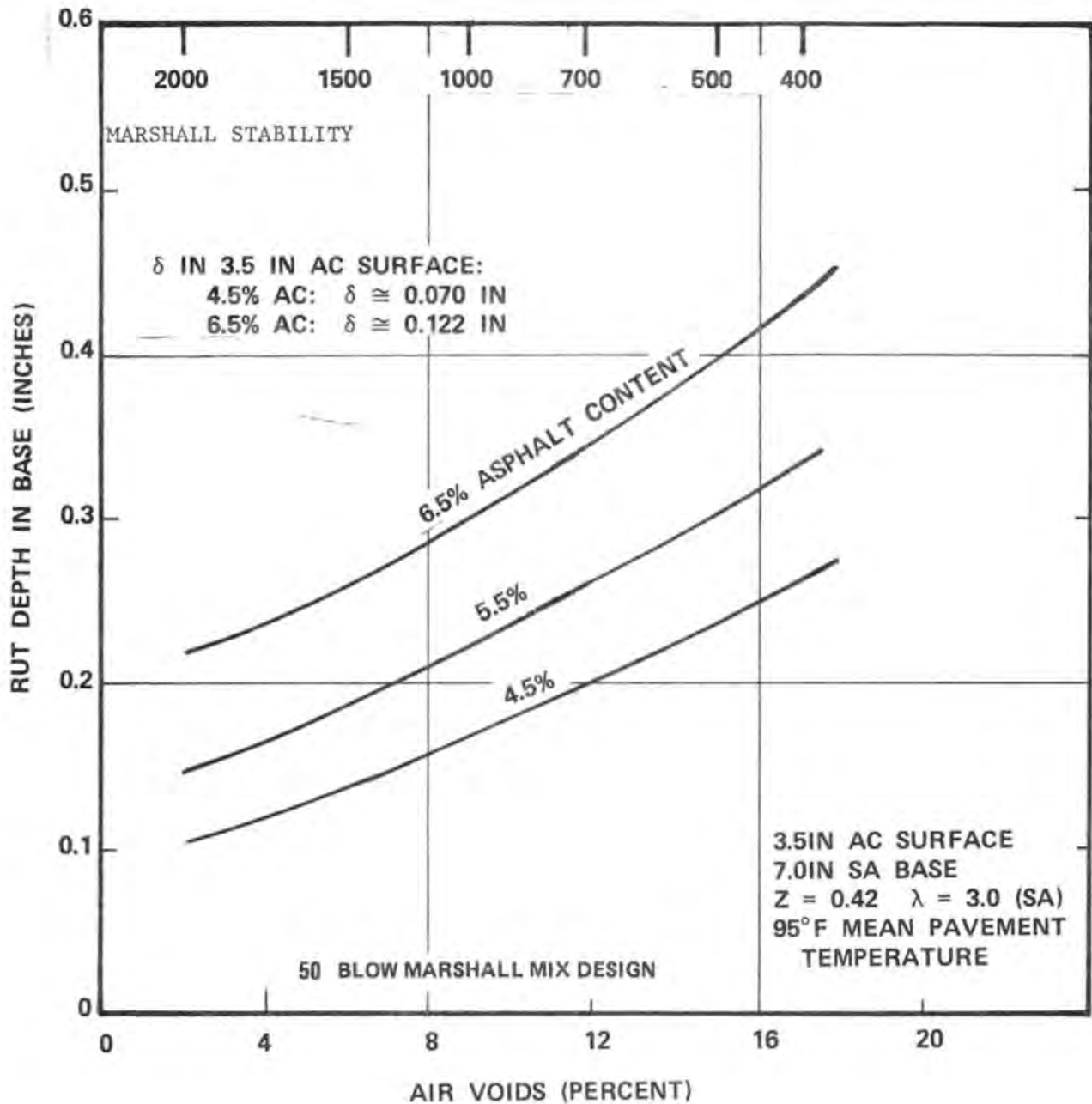


FIGURE 66. DESIGN RELATIONSHIP FOR ESTIMATING PRELIMINARY RUT DEPTHS IN SAND-ASPHALT AND SAND/STONE BLEND BASES 10.5 INCH STRUCTURAL SECTION SUPPORTED BY A FAIR SUBGRADE

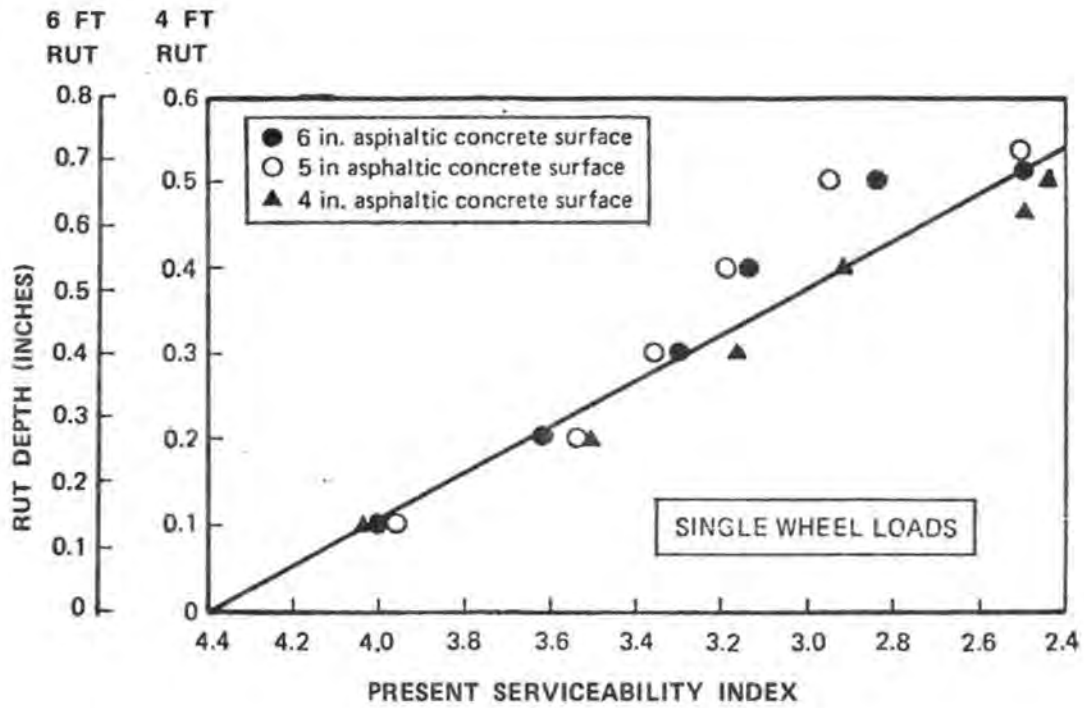


FIGURE 67. RELATIONSHIP BETWEEN RUT DEPTH AND PRESENT SERVICEABILITY INDEX FROM AASHO ROAD TEST DATA (AFTER LISTER AND ADDIS)

A pavement section(s) must then be selected having at least the required weighted structural number obtained from Fig. 68. The weighted structural number of a section is estimated from the following expression:

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \dots \dots \dots (20)$$

where: a_1, a_2, a_3 = Structural coefficients for the surface, base and subbase, respectively

D_1, D_2, D_3 = Actual thickness of the surface, base and subbase courses, respectively (in inches)

Recommended structural coefficients for the surface and base courses are given in Table 29 for construction utilizing sand-asphalt and sand-cement pavement sections. The actual value of the structural coefficients can vary greatly depending upon the quality of materials used, level of stabilization, construction specifications and the quality control program followed during construction. Therefore, careful consideration should be given to the above factors in selecting appropriate structural coefficients for design. To account for variation in materials, different classes of construction are given for each of the surface and base course materials. In general, the higher class of construction should be used where practical to optimize the life of the pavement by taking advantage of the dramatic increase in fatigue life and durability of materials stabilized with slightly higher levels of asphalt and Portland cement [17].

A comparison of the predicted and observed performance (as reflected by the PI value which is defined in Chapter 2) of pavement sections in Maryland indicates that use of the soil support value (S) obtained from the AASHTO Interim Guide [18] procedure using the measured CBR value of the subgrade generally gives a predicted pavement life considerably greater than that measured. Therefore, limiting soil support values were developed based on the AASHTO subgrade classification of the subgrade (Table 27). In estimating the life of the pavement using the AASHTO Interim Guide procedure, it is recommended that the smaller soil support value obtained from Table 27 or from test results be used. This procedure for selecting soil support values gives better agreement between measured and observed performance although even then predicted performance is generally better than that observed. At the present time, the Georgia Department of Transportation usually uses a soil support value of 2.0 in the Piedmont Province and 4.5 to 5.0 in the Coastal Plain. The corresponding regional factors used are 1.8 for the Atlanta area of the Piedmont and 1.4 in the Coastal Plain.

Sand-Cement Base Construction

The use of sand-cement in pavement construction in coastal plain areas should be given more attention. Sand-cement bases 6 in. or 8 in. (150 or 200 mm) in thickness overlaid by a triple surface treatment offers a very cost-effective type construction for lightly trafficked

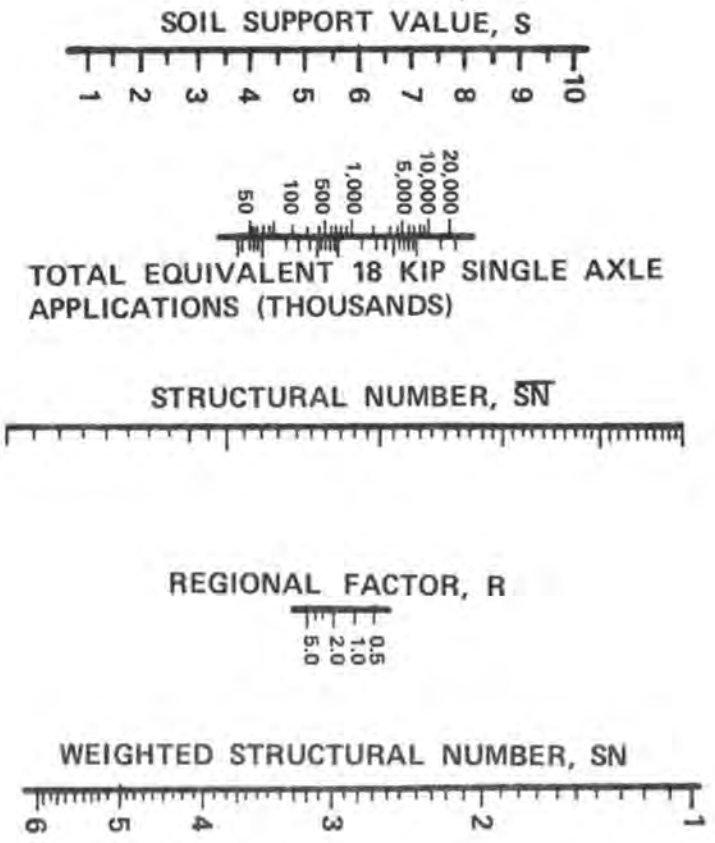


FIGURE 68. AASHO INTERIM DESIGN CHART FOR A PRESENT SERVICEABILITY INDEX VALUE OF 2.5

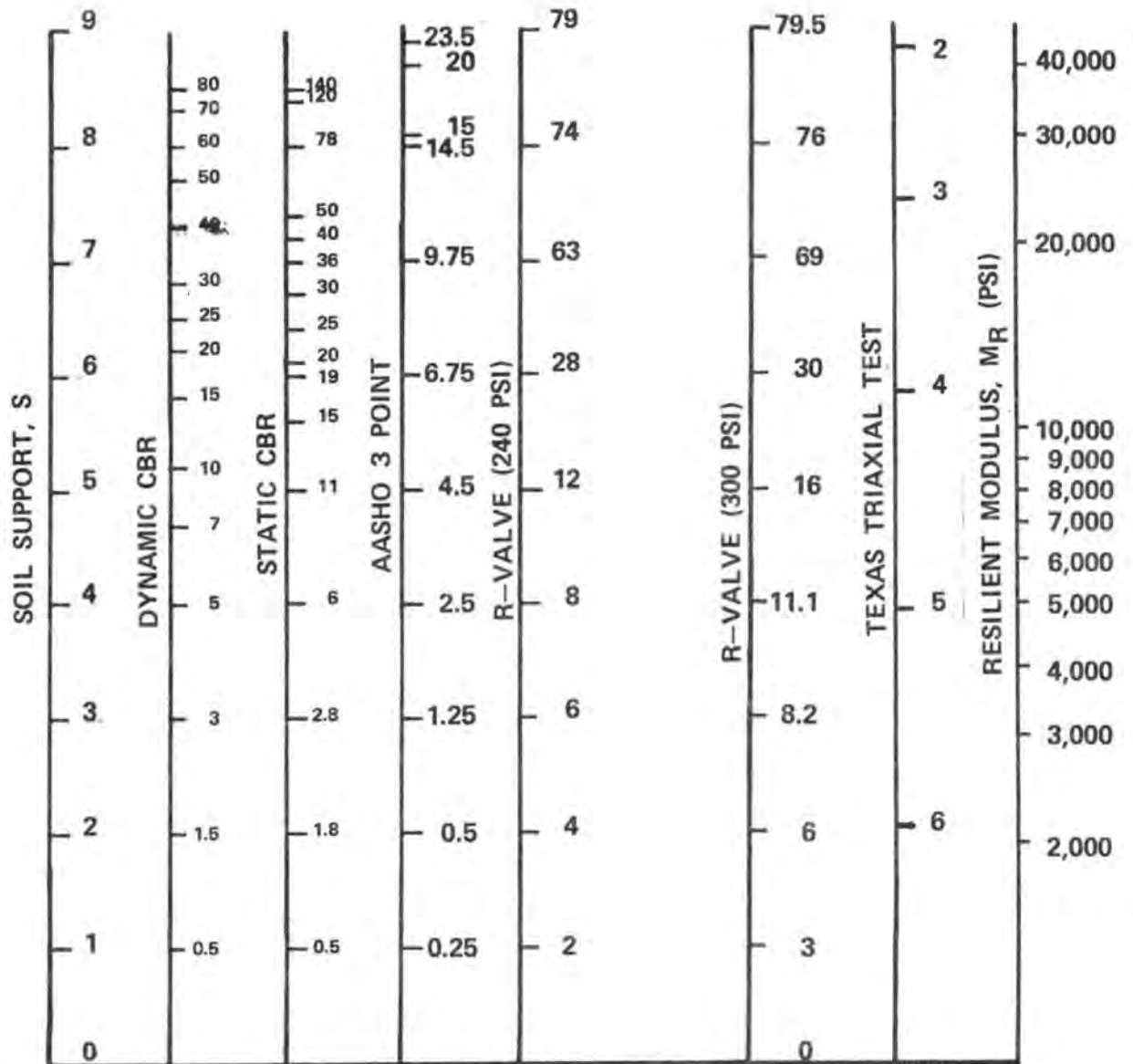


FIGURE 69. SOIL SUPPORT VALVES CORRELATED WITH ROUTINE MATERIAL TESTS (AFTER Van Til et al, NCHRP 128)

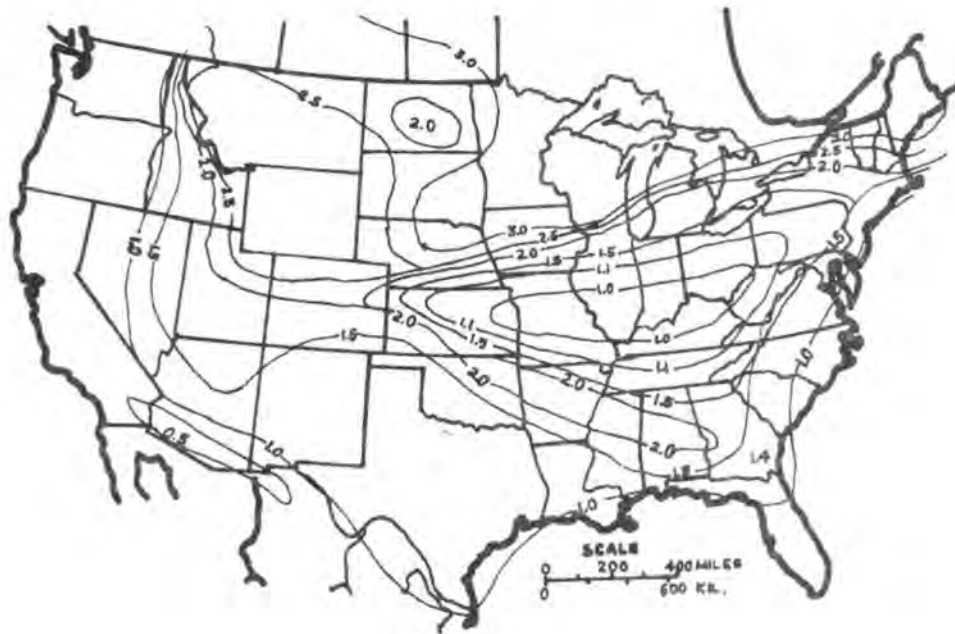


FIGURE 70. REGIONAL FACTORS FOR USE IN AASHTO INTERIM GUIDE DESIGN METHOD (AFTER Van Til et. al., NCHRP Report 128)

Table 27 . Limiting Soil Support Values Based on
AASHTO Soil Classification

| Classification | Description | Upper Soil Support Value |
|----------------|---|--------------------------|
| A-1a | Largely gravel but can include sand and fines | 6.5 |
| A-1b | Gravelly sand or graded sand; may include fines | 6 |
| A-2-4 | Sands, gravels with low plasticity silt fines | 5 |
| 2-2-4 | Micaceous silty sands | 2.5 - 3.0 |
| A-2-5 | Sands, gravels with plastic silt fines | 4 |
| A-2-6 | Sands, gravels with clay fines | 4.0 - 5.0 |
| A-2-7 | Sands, gravels with highly plastic clay fines | 4.0 |
| A-3 | Fine sands | 4.5 |
| A-4 | Low compressibility silts | 4.0 |
| A-5 | High compressibility silts, micaceous silts and micaceous sandy silts | 2.5 - 3.5 |
| A-6 | Low to medium compressibility clays | 3.5 - 4.5 |
| A-7 | High compressibility clays, silty clays and high volume change clays | 3-4 |

Table 28. Total Daily Traffic Loadings -
Both Directions⁽¹⁾

| <u>Classification</u> | <u>Total Traffic</u> ⁽²⁾ |
|-----------------------|-------------------------------------|
| Light | 100 - 2,000 |
| Medium | 2,000 - 20,000 ⁽³⁾ |
| Heavy | > 30,000 |

1. *Thickness Design*, The Asphalt Institute, MS-1, August, 1970, p. 12.
2. Approximate Heavy Truck Traffic usage is as follows:
 - City Streets (local): 5 - 15%
 - Urban Highways
 - Primary: 5 - 15%
 - Interstate: 5 - 10%
 - Local Rural Roads: 15% or less
3. More than two lanes is required for traffic volumes greater than 2,000 to 6,000 vehicles per day.

Table 29. Recommended AASHTO Interim Guide Structural Coefficients for Thickness Design

| Structural Layer | Class | Structural Coefficient | General Requirements |
|--|-------|------------------------|---|
| 1. Surface and Binder Course (Weighted Avg.) | | | |
| Asphalt Concrete | 1 | 0.48 | > 6.0% AC; 2-4% Air Voids; > 1500 lb. Marshall Stability ⁽¹⁾ |
| | 2 | 0.44 | > 4.8% AC; 2-6% Air Voids; > 1200 lb. Marshall Stability |
| | 3 | 0.35 | < 4.4% AC; 2-8% Air Voids; > 700 lb. Marshall Stability |
| Sand Asphalt | 1 | 0.35 | > 5.8% AC; <14% Air Voids; > 550 lb. Marshall Stability ⁽¹⁾ |
| | 2 | 0.27 | < 4.8% AC; <18% Air Voids; > 400 lb. Marshall Stability |
| 2. Base Course | | | |
| Crushed Stone (Untreated) | 1 | 0.14 | Well graded; 1-1/2 in. or greater top size; 3-8% fines; 100% T-180 compaction |
| Asphalt Concrete | 1 | 0.34 | > 5.8% AC; 2-4% Air Voids; > 1200 lb. Marshall Stability ⁽¹⁾ |
| | 2 | 0.28 | < 4.8% AC; <8% Air Voids; > 1200 lb. Marshall Stability |
| Sand Asphalt | 1 | 0.25 | > 5.8% AC; 14% Air Voids; > 600 lb. Marshall Stability ⁽¹⁾ |
| | 2 | 0.17 | < 4.5% AC; 18% Air Voids; > 350 lb. Marshall Stability |
| Sand Cement | 1 | 0.24 | > 600 psi, 7 day Compressive Strength |
| | 2 | 0.18 | > 400 psi, 7 day Compressive Strength |
| 3. Inverted Structural Section - Experimental ⁽²⁾ | | | |
| Unstabilized Sand Base | | 0.10 to 0.12 | Clean, medium to coarse sand with less than 4 to 8 percent fines |
| Unstabilized sand - crushed stone blend | | 0.16 | |

- NOTES: 1. Given Marshall Stabilities are for a 50 blow Mix Design
2. Structural section consisting of unstabilized clean sand or crushed stone placed between a sand-cement base and asphalt concrete surface course. Use structural coefficients for sand-cement base and asphalt concrete surface course given above.

roads. Inverted construction consisting of a cement stabilized subbase underlying an unstabilized crushed stone base also deserves further consideration for moderate to heavily trafficked roadways. This type construction has been successfully used in New Mexico and Virginia and makes optimum use of material properties while at the same time reduces or delays reflection cracking. In light to moderately trafficked roadways, a well compacted, unstabilized sand could be used as the base material for the inverted construction provided techniques could be developed for paving over the sand base. Appropriate design methods for selecting the required cement content for sand-cement construction has been given elsewhere [35,67].

The fatigue tests performed as a part of this study indicate that an unconfined compressive strength of 300 psi (2070 kN/m²) gives fatigue strengths equal to or greater than asphalt concrete black bases. Because of the high rainfall occurring in the southeast, sand-cement bases are subjected to severe conditions of wetting and drying. Also, significant variations in cement content occur because of practical problems with mixing the sand and cement. Therefore, a minimum design unconfined compressive strength of, where practical, at least 400 psi (2760 kN/m²) should be used to insure a minimum amount of deterioration of the base with time. One of the most important factors in the design of cement stabilized bases is the use of a sufficient amount of cement.

Soluble Salt Content

Blight *et al.* [69] have found that soluble salt contents greater than 2 percent can cause deterioration of an asphalt concrete pavement. Also, even for lower salt contents, the surfacing may be rapidly abraded away, although the primary effect of rapid abrasion is to increase skid resistance. Excessive concentrations of salts can also affect the performance of Portland cement concrete. The Portland Cement Association [69] recommends that when the sum of dissolved carbonates and bicarbonates of sodium and potassium salts in mixing water exceeds 0.1 percent (1,000 ppm) and for 400 ppm of salts of manganese, tin, zinc, copper, lead tests for salting time and 28-day strength should be made.

Since the sands obtained from the Altamaha and Ogeechee Rivers were taken from areas of tidal fluctuations, the dissolved salt content of a small sample of sand was tested from the mouth of the Altamaha (boring A-4) and Ogeechee Rivers (boring B-12), and also from boring A-9 along the Altamaha River. The soluble salt contents obtained from these tests are given in Table 31. The very small salt contents found in these sands should not cause any deterioration problems in sand-asphalt, sand-cement, or Portland cement concrete.

Use of River Sands As Concrete Sand and For Beach Replenishment

The average gradation of the sands sampled in the Ogeechee and Altamaha Rivers falls within the allowable gradation limits specified for concrete sand by ASTM C-33 except for the No. 30 sieve. In the vicinity of the No. 30 sieve, the sands are slightly finer than allowed by ASTM, and hence some sieving would be required. Although washing may be

Table 30. Selected Structural Coefficients Used by Various Transportation Organizations in the AASHO Interim Guide Design Method

| Pavement Component | Structural Coefficient | | | |
|--|------------------------|---------------------|------|-------------|
| | Fla. | Ga. | Md. | S.C. |
| I. Surface and Binder Course (a_1) | | | | |
| Asphalt Concrete | (5) 0.2 - 0.4 | (1) 0.44 | 0.44 | 0.44 |
| Bituminous Surfacing | - | - | - | 0.35 |
| II. Base Course (a_2) | | | | |
| Asphalt Concrete | (4) 0.21 - 0.30 | 0.30 | 0.28 | 0.34 |
| Sand-Asphalt | | 0.12 | 0.28 | 0.20 - 0.25 |
| Soil Cement | 0.22 | 0.20 | 0.28 | 0.20 |
| Graded Aggregate | | 0.18 ⁽²⁾ | 0.14 | 0.12 - 0.20 |
| Cement Stabilized Graded Aggregate | | 0.22 | 0.28 | 0.34 |
| Sand Aggregate | - | - | 0.14 | - |
| Sand-Clay CBR > 49 | 0.12 | - | - | - |
| Limerock CBR > 80 | 0.15 | - | - | - |
| Limerock Stabilized Base CBR > 56 | 0.12 | - | - | - |
| III. Subbase (a_3) | | | | |
| Graded Aggregate | | 0.14 | - | - |
| Topsoil or Sand-Clay | | 0.10 | - | - |
| Gravel or Screenings | - | - | 0.07 | - |
| Soil Aggregate | - | - | - | 0.08 - 0.12 |
| Cement Stabilized Earth | - | - | - | 0.15 |

1. Georgia uses a coefficient of 0.44 for surface and binder to a depth of 4.5 in. (110 mm).
2. When compacted to 100% of T-180 density.
3. Subbase coefficients are used in Georgia below a depth of 12 in. (300 mm).
4. The Florida DOT uses the following structural coefficients for different base mixes: (1) Type I, 500 lb. Marshall stability, $a_2 = 0.21$; (2) Type II, 750 lb. Marshall stability, $a_2 = 0.25$; (3) Type III, 1,000 lb. Marshall stability, $a_2 = 0.30$.
5. The Florida DOT uses the following structural coefficients for different surface mixes: Type S1, 1,000 lb. Marshall stability, $a_1 = 0.40$; Type S2, 1,000 lb. Marshall stability, $a_2 = 0.20$; and Type S3, 750 lb. Marshall stability, $a_1 = 0.30$.

Table 31. Soluble Salt Contents of Selected Sands From the Altamaha and Ogeechee Rivers

| Soluble Salt | Composition of Water Extract mg/eg./100 grams soil | | |
|--------------------------------|---|--------|--------|
| | A | B | C |
| Calcium (Ca) | 0.84 | 0.02 | 0.08 |
| Magnesium (Mg) | 1.20 | 0.08 | 0.12 |
| Sodium (Na) | 2.39 | 0.01 | 0.04 |
| Potassium (K) | 0.01 | 0.01 | 0.01 |
| Carbonate (CO) | 0 | 0 | 0 |
| Bicarbonate (CO ₃) | 0.0 | 0.04 | 0.04 |
| Chloride (Cl) | 2.85 | 0 | 0 |
| Sulfate (SO ₄) | 0.05 | 0.11 | 0.21 |
| Phosphate (PO ₄) | 0.005 | 0.002 | 0.002 |
| Nitrate (N) | < 0.01 | < 0.01 | < 0.01 |

Sample Locations:

Sample A: A-4 Near Mouth of Altamaha River
 Sample B: B-9 Middle Section of Ogeechee River
 Sample C: B-12 Near Mouth of Ogeechee River

required to remove some of the organics, the Altamaha and Ogeechee River sands offer a good source of concrete sands. Also, because of the fine to medium gradation and lack of fines, these sands offer a reasonably good sand source for beach replenishment although a coarser sand would perform better.

General Discussion

Sand-asphalt and sand-stone blend asphalt concrete can be successfully used as base courses, surfacings, and leveling courses. Also, use of gap-graded surface mixes with the finer portion consisting of local sands should be given more consideration. Gap-graded surface mixes have been found to be fatigue resistant and have been used successfully for a number of years in both the United Kingdom and South Africa (Chapter 2, p. 32). A reasonably comprehensive discussion of the present use of sand and sand-asphalt by selected transportation organizations was given in the previous section of this chapter. At the present time, two different approaches are followed in the design of sand-asphalt base mixes. Florida designs a sand-asphalt mix having typically 6 1/2 to 7 1/2 percent asphalt content and a 50-blow Marshall stability greater than 500 lbs. (2.2 kN) and often greater than 700 lbs. (3.1 kN). This type sand-asphalt base mix is usually placed beneath asphalt concrete surfacings typically 3 in. (75 mm) in thickness and used for moderate to heavy traffic loading conditions. On the other hand, Maryland and South Carolina use a mix having typically 4 to 5 percent asphalt content. This type mix is generally placed beneath 5 to 6 in. (125 to 150 mm) of asphalt concrete and used under moderate to heavy traffic loading conditions.

Sand-asphalt has been used in the construction of pavements in Georgia since only about 1974. Georgia has followed a mix design practice followed by Florida on the one hand and South Carolina and Maryland on the other. For most type materials, the asphalt content tends to be somewhat less than that used by Florida, but greater than South Carolina and Maryland. Generally the asphalt-cement content used is between 5.5 and 6.5 percent with 5.5 percent and 6.5 to 7 percent being common for certain type materials. Sand-asphalt bases, when used, have been generally placed under 6 in. (150 mm) or more of asphalt concrete. Extensive use has recently been made of sand-asphalt for leveling and resurfacing in both Georgia and South Carolina. The design criteria presently being used by Georgia is summarized in Table 25. Although extensive field performance using this criteria has not been gained, the design criteria are generally consistent with the findings of this study.

An increase in fatigue resistance and decrease in rutting potential is directly related to an increase in 50 blow Marshall stability and inversely related to air voids content of the mix. The fatigue tests performed as a portion of this investigation indicate that the laboratory fatigue resistance of a sand-asphalt mix can be increased by a factor of approximately four by increasing the asphalt content from 4 1/2 to 5 1/2 or 6 percent. Likewise an increase in Marshall stability from 350 lbs. (1.5 kN) to 700 lbs. (3.1 kN) should increase the fatigue life by a factor on the order of three. Based on both the observed field

performance of pavements with sand-asphalt bases and also the laboratory fatigue tests, the recommendation is made that for at least moderate to heavy traffic conditions and asphalt concrete surfacings 3 to 4 in. (75 to 100 mm) in thickness, the higher quality sand-asphalt base construction should be used having Marshall stabilities in the range of 600 to 700 lbs. (2.2 to 3.1 kN).

For thicker asphalt concrete surface courses or light traffic conditions, lower stability and/or asphalt content mixes can be successfully used. For this type mix and construction, the sand-asphalt base very likely functions more like a subbase and has considerably lower strengths (and hence lower base course coefficients) than the higher quality sand-asphalt mixes. For either type mix, to maximize fatigue life the stability of the mix should be made as great as practical and the air voids in the mix should be minimized. Typical guide specifications are given in Appendix C for the construction of sand-asphalt bases.

The use of sand-stone blend asphalt concrete base course mixes offers an excellent way on some projects to reduce the overall cost of the mix while at the same time obtaining a high quality asphalt concrete base. Sand-stone blend mixes can be used having up to 40 or 50 percent sand. These mixes can be designed to have good fatigue properties and reasonably low asphalt contents in the range of 4 1/2 to 5 1/2 percent. At the same time, such mixes should experience on the order of 25 percent less rutting than a pure sand-asphalt. Use of this type mix tends to optimize the fatigue and rutting characteristics while at the same time large asphalt contents are not required. These mixes are also suitable for use under heavy traffic including stop and go traffic flow conditions. Sand-stone blend mixes will probably be used more in the future.

Sand-asphalt mixes can also be successfully used for surface courses on lightly trafficked roadways and also for leveling courses and overlays. One excellent way to reduce the cost of asphalt concrete surface mixes is to permit the replacement with sand of the finer portion of a conventional crushed stone surface mix. Florida allows the use of up to 25 percent sand in such mixes. Finally, gap-graded surface mixes such as those used in South Africa and the United Kingdom should be tried in the United States. These mixes have been found to exhibit excellent fatigue properties and were discussed in Chapter 2.

In the past, considerable discussion has arisen concerning what constitutes a suitable sand for use in sand-asphalt mixes. In general, the sand equivalent has been found in this study and others [23,24] not to be a very good indicator of the quality of a sand for use in sand asphalt. As a result, sands having sand equivalents as low as 15 can be successfully used in sand asphalt mixes. Clay balling in some materials may become a problem when the sand equivalent is less than 22 to 24 when these materials are used in a drum dryer. Likewise sands with sand equivalent values greater than approximately 70 are too coarse and require the addition of fines. Generally, the fines content should be approximately equal to or greater than the asphalt content of a mix [22]. Experience and the laboratory tests performed in this study

indicate that some relatively small amount of clay is actually desirable in a sand-asphalt mix. The desirable amount of clay as determined by the elutriation test would appear at this time to be in the vicinity of 4 to 6 percent with 7 percent being an upper limit. For these reasons, the sand equivalent test is not a very valid indicator of potential performance, and sands should only be rejected if the sand equivalent is less than 20 and for some materials as low as 15. Of course, the sand should be angular and well-graded. The specific criteria developed for gap-graded mixes by Freeme [23] previously given in Chapter 2, p. 37, can also be used as a general guide for sand-asphalt mixes.

Quite good performance has been obtained from unstabilized sand-slag blend bases in Maryland (Chapter 2, p. 19). Unstabilized sand-crushed stone or sand-slag blend bases could be effectively used on light to moderately trafficked roadways as the cost of transportation and stabilizing agents continue to rise. This type of unstabilized mix could also be used over a sand-cement subbase to give an inverted type construction. Finally, as discussed in the last section, more use should be made of sand-cement base construction in areas where surface shrinkage reflection cracks are not aesthetically objectionable. Methods for minimizing reflection cracking in sand-cement bases have been described in Chapter 2,

CHAPTER 9

CONCLUSIONS

Approximately five and one-half tons of sand were obtained from 39 locations on the Ogeechee and Altamaha Rivers for testing. The sand samples were obtained using a specially designed air-water uplift system mounted on a barge. Sand samples were also obtained from a local sand pit. The Altamaha and Ogeechee River materials sampled were generally a clean, uniform subrounded to subangular, fine to medium sand. The Altamaha River sand typically had one percent passing the No. 200 sieve, while the Ogeechee River sand had 0.5 percent passing. The sand equivalent values of the Ogeechee River sands were generally between 80 and 100, while those for the Altamaha River sands were somewhat lower varying from 60 to 100. Trace to moderate amounts of organics usually consisting of fibrous material were found in many of the sand samples.

The Ogeechee and Altamaha River sands were found to be suitable for beach replenishment because of the relatively clean nature of the sand. The fine to medium gradation of these sands should not result in serious erosion problems. The average gradation of the Ogeechee and Altamaha River sands sampled fell within the allowable gradation limits specified for concrete sand by the American Society for Testing Materials (ASTM C-33), except for the No.30 sieve size which was slightly finer than that allowed. In some instances, the sampled sands were sieved or washed to remove the organics for use as concrete sand. The very low soluble salt content found in three selected sand samples should not cause deterioration problems in sand-asphalt, sand-cement or Portland cement concrete mixes.

An extensive series of fatigue and rutting tests was performed to evaluate the use of the river and pit sands in both sand-asphalt and sand-cement base course construction. The fatigue tests used consisted of placing a 3 in. by 3 in. by 20 in. long beam specimen on an elastic subgrade having a modulus of subgrade reaction of 215 pci (5,915 gm/cc). The test was performed at a temperature of 80°F (27°C) by subjecting the beam to a constant repetitive load until fatigue failure occurred. The rutting tests used for the sand-asphalt and sand-stone blend mixes consisted of performing a repeated load triaxial test on cylindrical specimens of sand-asphalt base course material.

Sand-Asphalt Bases

Fatigue and rutting tests were performed on a wide range of sand-asphalt mixes having varying asphalt contents and varying blends of sand and crushed stone. Specific conclusions based on field inspections made in Florida, Georgia, Maryland and South Carolina and also the laboratory fatigue and rutting test results are summarized as follows:

1. Sand-asphalt and sand-stone blend asphalt mixes can be successfully used as base courses, surfacings, and as leveling courses.
2. Rutting in pavements constructed using a sand-asphalt base is typically between 0.3 and 0.6 in. (8 to 15 mm). The rut depths observed in pavements having sand-asphalt bases is typically on the order of twice the rutting found in pavements using conventional asphalt concrete construction. As a result, more consideration must be given to rutting in the mix design of sand-asphalt bases compared with conventional mixes. A study of the literature and field observations indicates that an allowable rut depth is approximately 0.4 to 0.5 in. (10 to 13 mm) on primary and interstate pavements and 0.6 in. (15 mm) on secondary roadways. An approximate rut depth of 0.4 in. (10 mm) has been found to be the limiting value of rutting before loss of structural strength starts to occur in a pavement structure.
3. A practical method for predicting rutting was developed in Chapter 3 which considers the appropriate stress state in the pavement using the Z-function approach. Use of this method requires performing one repeated load triaxial test or preferably two tests and averaging the results on a sand-asphalt mix.
4. The repeated load test is performed at the mean temperature at which rutting occurs in the pavement structure. A confining pressure, σ_3 of 5 psi (34 kN/m²) is used together with a repeated axial deviator stress, $\sigma_1 - \sigma_3$, of 25 psi (170 kN/m²). The test results obtained for the standard test conditions are then modified using hyperbolic correction functions for the stress level and number of repetitions to which the base will be subjected under in-service loading conditions.
5. Important variables affecting rutting in sand-asphalt and sand-stone blends are asphalt content, Marshall stability (or air void content which was related to Marshall stability), and the characteristics of the aggregate. The characteristics of the finer part of the sand-asphalt mix apparently play a very important part in the mix performance, although the gradation and top size also effect performance.
6. Fatigue characteristics of the mix can generally be controlled by limiting the void content to 12 to 14 percent, using asphalt contents greater than approximately 5 1/2 to 6 percent and designing a mix having a Marshall stability as high as practical. Secondary variables having the most influence on fatigue performance were found to be angularity and gradation of the sand used in the mix.

7. The laboratory fatigue resistance of the sand-asphalt mix tested was increased by a factor of approximately 4 by changing the asphalt content from 4 1/2 to 5 1/2 or 6 percent. Likewise, an increase in Marshall stability from 350 lbs. (1.5 kN) to 700 lbs. (3.1 kN) resulted in an increase in fatigue life by a factor of approximately three.
8. Decreasing the air voids from 16 to 12 percent approximately doubles the fatigue life based on the laboratory test results. However, because of the beneficial effect of traffic on the higher void mixes discussed in Chapter 8, the actual difference in performance in the field would be less than indicated by the laboratory fatigue test results. Traffic compaction effects at least partially explain the reasonably good performance of sand-asphalt bases constructed with relatively high initial air void contents. Nevertheless, considering all factors, sand-asphalt mixes generally should be designed and initially compacted to as low an air void content as practical.
9. The use of sand-stone blend asphalt concrete base course mixes offers an excellent way on some projects to reduce the overall cost of the mix, while obtaining a high quality asphalt concrete base at the same time. Sand-stone blend mixes can be used having up to 40 or 50 percent sand. These mixes can be designed to have good fatigue properties, and reasonably low asphalt contents in the range of 4 1/2 to 5 1/2 percent. Sand-stone blend mixes should experience on the order of 25 percent less rutting than a pure sand-asphalt.
10. Sand equivalent is not a good indicator of the quality of the sand for use in sand-asphalt. As a result, sands having sand equivalents as low as 15 can be successfully used in sand-asphalt mixes. In general, the fines content in a mix should be approximately equal to or greater than the asphalt content of the mix. A small amount of clay on the order of 4 to 6 percent is actually desirable in a sand-asphalt mix.
11. When a drum dryer is used, clay balling may become a problem. Since screening cannot be used to remove the clay balls, a limiting sand equivalent on the order of 25 may be required when a drum dryer is used. If a conventional asphalt plant is used, clay balls can be removed by screening.

Sand-Cement Construction

More use should probably be made in the Coastal Plain area of Georgia of sand-cement construction in areas where surface shrinkage reflection cracks are not aesthetically objectionable. If a sufficient level of cement stabilization is used, the reflection cracks do not seriously damage the structural integrity of the pavement. Sand-cement bases 6 to 8 in. (150 to 200 mm) in thickness overlaid by a triple surface treatment offers a very cost-effective construction for lightly trafficked roadways. The laboratory fatigue tests indicate that an

unconfined compressive strength of 300 psi (2,070 kN/m²) gives strengths equal to or greater than asphalt concrete black bases. Because of the high rainfall in the southeast, sand-cement bases are subjected to severe conditions of wetting and drying. Also, significant variations in cement content occur because of practical problems with mixing the sand and cement together. Therefore, a minimum design unconfined compressive strength, where practical, of at least 400 psi (2,760 kN/m²) should be used to ensure a minimum amount of deterioration of the base with time.

Additional conclusions are as follows:

1. The cost of a sand-cement base has generally been found to be cheaper than either that of sand-asphalt or asphalt concrete base construction.
2. Some reflection cracking will occur on the surface when a cement stabilized base is used. The amount of reflection cracking appearing on the surface can be minimized by: (a) using a granular base material having a minimum clay content, (b) compacting the base to a high density at or slightly below optimum water content, (c) using the highest penetration asphalt cement in the surfacing that will satisfy stability requirements, and (d) waiting until initial shrinkage cracking has occurred in the base before placing the surfacing.
3. Double and triple surface treatments compared with asphalt concrete surfacings were found by Nordling to result in the development of fewer reflection shrinkage cracks on the surface of light to moderately trafficked pavements. An effective technique that can be used is to initially apply a double surface treatment and then after one to two years, place an additional surface treatment to help cover the shrinkage.
4. Inverted pavement construction consisting of placing an unstabilized crushed stone base over a cement stabilized sand subbase should be given more consideration for moderate to heavily trafficked roadways. This type construction has been successfully used in New Mexico and Virginia and makes optimum use of material properties while at the same time reduces and/or delays reflection cracking.
5. Relatively rapid deterioration of some pavements has been found to be due to non-uniform blending of the cement during mixing and placing, which can result in localized areas having insufficient strength. The plant-mixed cement stabilized bases have shown consistently better uniformity than bases mixed in-place.

Recommendations

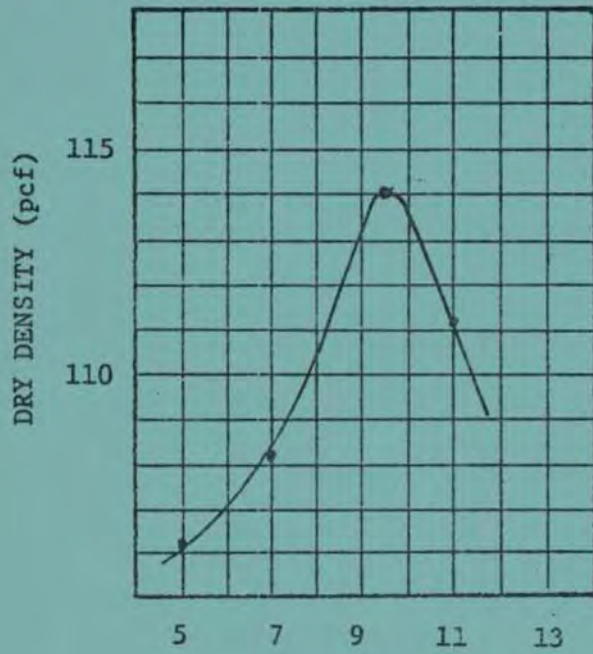
The following recommendations were developed from the investigation of the use of local sands in sand-asphalt and sand-cement base mixes:

1. The 50 blow Marshall mix design procedure should be used as a basic method for designing sand-asphalt mixes. For moderate to heavy traffic conditions and asphalt concrete coverings of 3 to 4 in. (75 to 100 mm), a Marshall stability should be used of 600 lbs. (2.7 kN). Where necessary, mineral filler, fines, or crushed stone should be added to the sand to give sufficient stability. The sand-asphalt mix should be designed to have an air void content in the range of 12 to 14 percent and an asphalt content on the order of 5 1/2 to 7 percent. The mix should be compacted to at least 95 to 96 percent of the 50 blow Marshall maximum density.
2. When an asphalt concrete surfacing thicker than approximately 6 in. (150 mm) is used above a sand-asphalt base, a mix can be used having a Marshall stability as low as 350 to 400 lbs. (1.6 to 1.8 kN) and as asphalt content in the range of 4.5 percent. For this type construction, the sand-asphalt base probably acts more like a subbase, and adds only a relatively small amount of strength to the pavement structure.
3. Under heavy start and stop traffic conditions and/or thin asphalt concrete coverings, sand-stone blend asphalt mixes should be used which have a stability of at least 1,000 lbs. (4.4 kN). This type of mix will have a low air void content and can be used with a relatively low percentage of asphalt in the mix. Sand-stone blends will also have good fatigue and stability properties. This type mix can be placed in lifts up to 6 in. (150 mm) thick.
4. For primary and interstate highways, an allowable rut depth of 0.4 in. (10 mm) should be used in design. Rut depths as great as 0.6 in. (15 mm) should not cause any serious problems from the standpoint of structural performance or safety. The rut depth which should occur in the pavement can be estimated using the theoretical method developed in Chapter 3. Also, a simple design chart is given in Chapter 8 which can be used for making preliminary estimates of relative rut depths.
5. The asphalt content and air voids in a sand-asphalt mix have a controlling effect on fatigue performance. To maximize fatigue life, the air voids in a base mix should be made as small as possible, preferably in the range of 12 to 14 percent. The fatigue life of a mix can be very significantly increased by small increases in asphalt content. The recommendation, therefore, is made that the asphalt content be as high as practical and still satisfy rut depth criteria.
6. More use should be made in pavement construction of sand-cement bases where reflection cracking is not aesthetically objectionable. A sand-cement base should be constructed where practical having an unconfined laboratory compressive strength at 7 days of at least 400 psi (2,760 kN/m²).

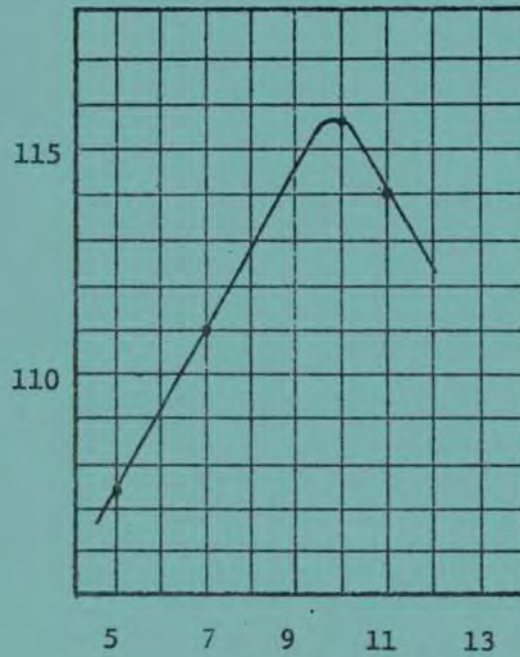
7. Consideration should be given to using inverted pavement construction where unstabilized crushed stone or sand/gravel mixtures are placed above a cement stabilized base. This type construction will minimize the use of stabilizing agents and also reduce reflection cracking which normally would be more prevalent when a conventional sand-cement base is used.

APPENDIX A
MOISTURE-DENSITY AND STRESS-STRAIN RELATIONSHIPS
FOR SAND CEMENT BLENDS

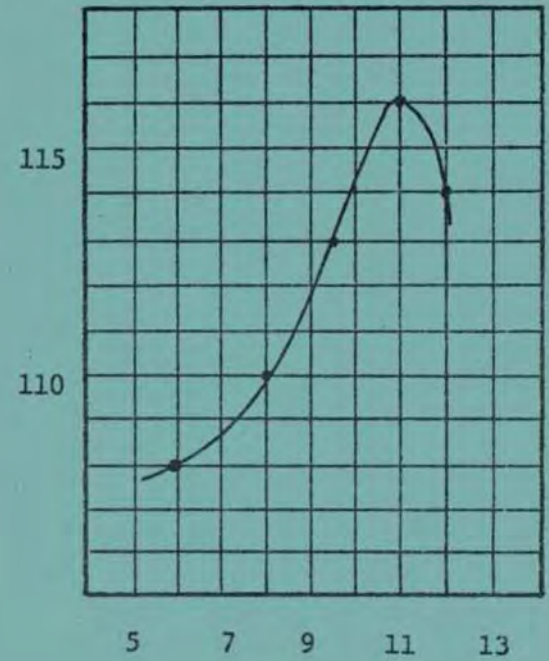
95% Sand
5% Stone Fines
6% Cement



95% Sand
5% Stone Fines
8% Cement

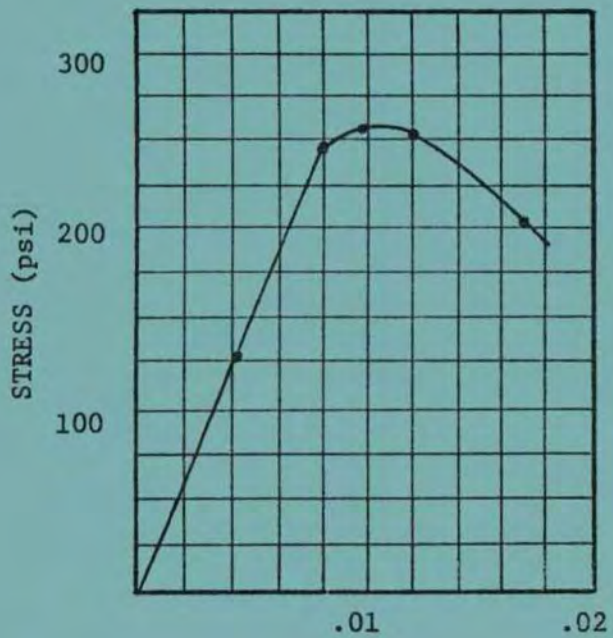


95% Sand
5% Stone Fines
10% Cement

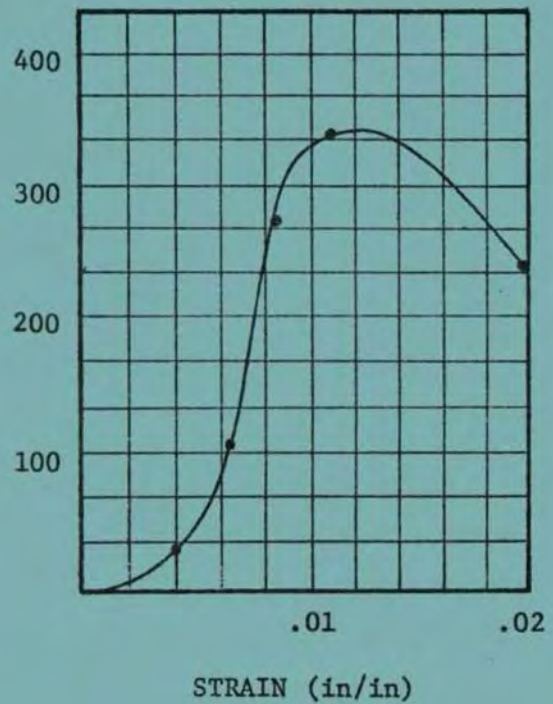


WATER CONTENT (%)

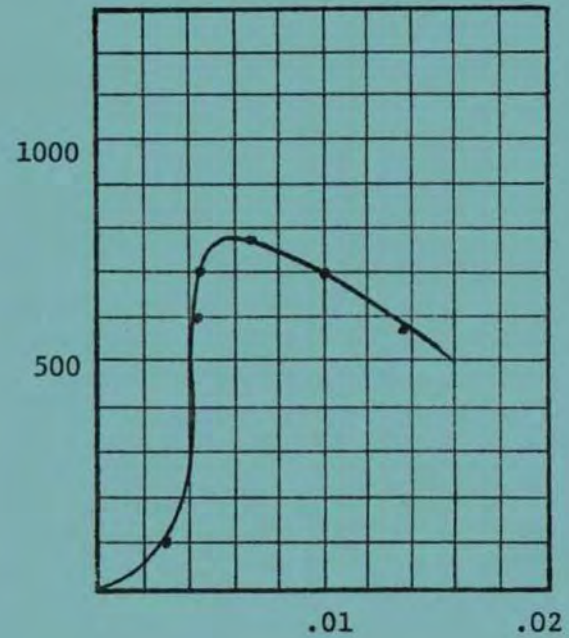
95% Sand
5% Stone Fines
6% Cement



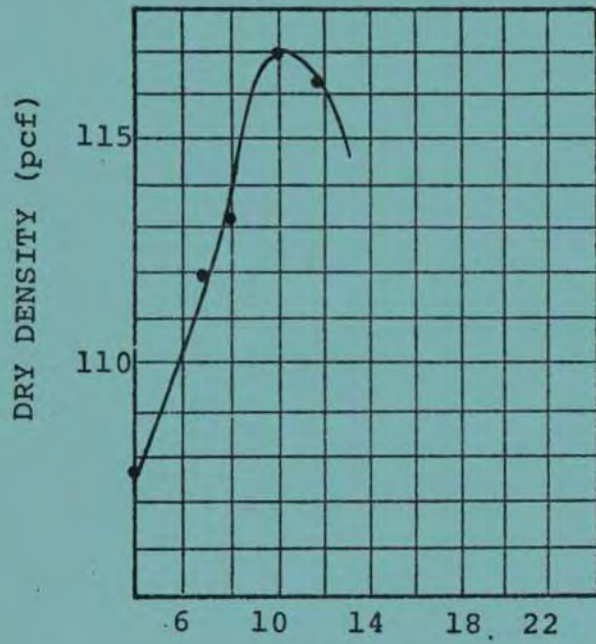
95% Sand
5% Stone Fines
8% Cement



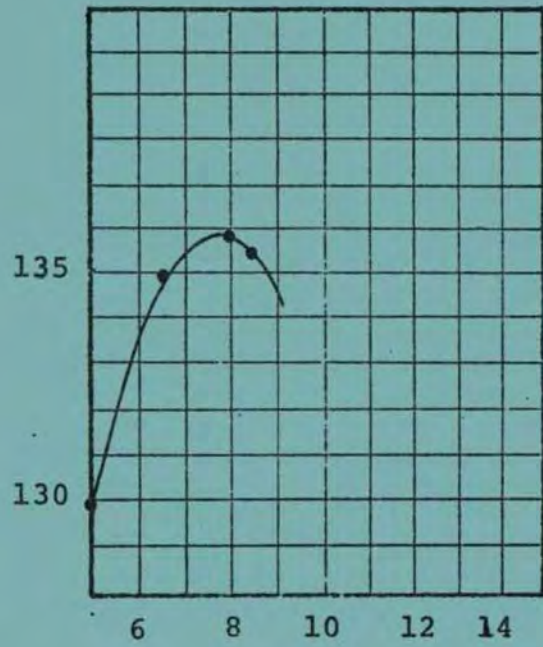
95% Sand
5% Stone Fines
10% Cement



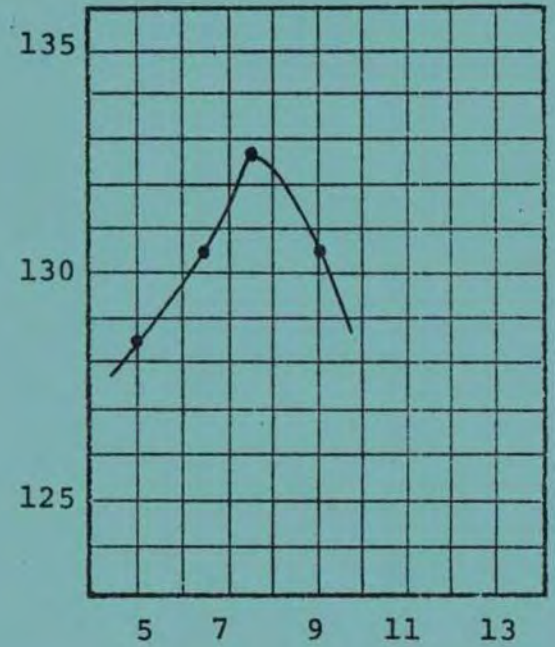
93% Sand
7% Fly Ash
9.5% Cement



69.75% Stone
23.25% Sand
7% Fly Ash
7% Cement

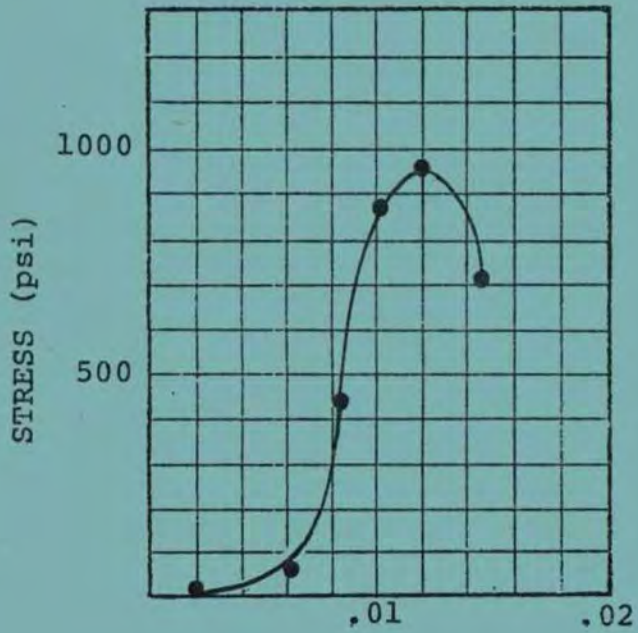


46.5% Stone
46.5% Sand
7% Fly Ash
6.9% Cement

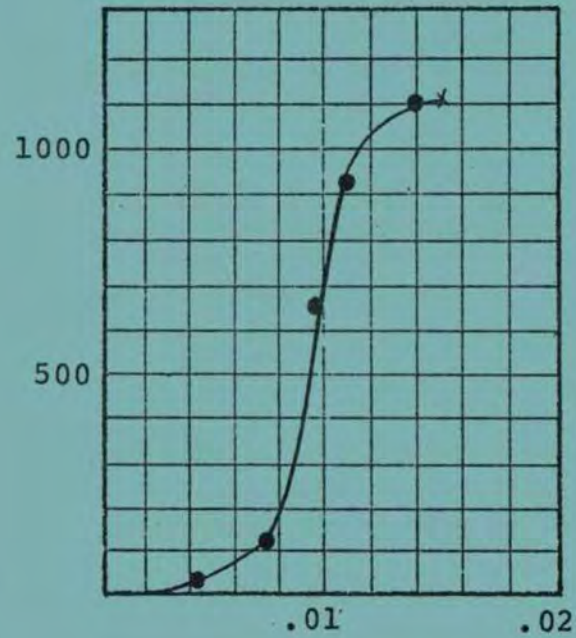


WATER CONTENT

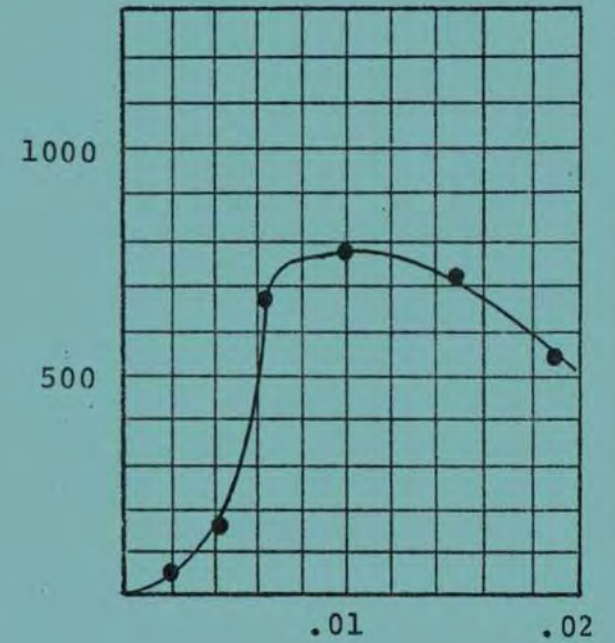
93% Sand
7% Fly Ash
9.5% Cement



69.75% Stone
23.25% Sand
7% Fly Ash
7% Cement

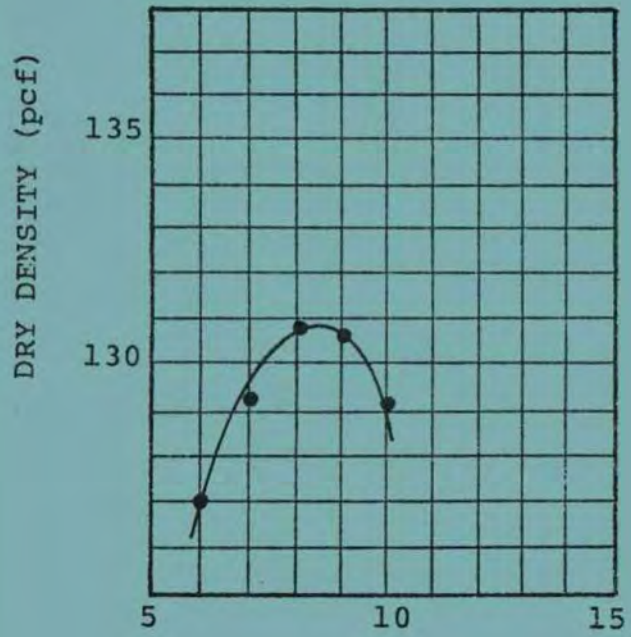


46.5% Stone
46.5% Sand
7% Fly Ash
6.9% Cement

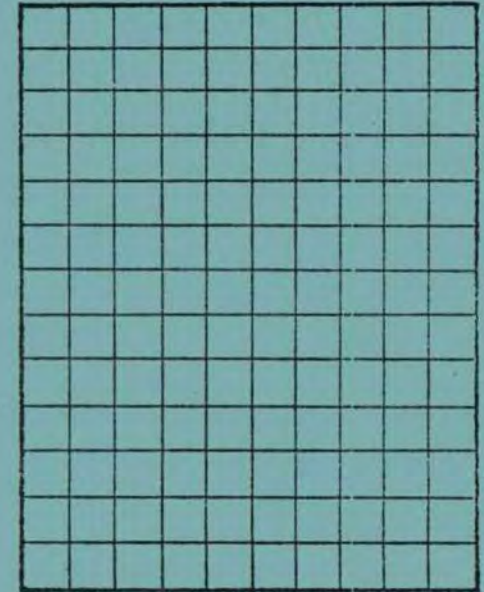
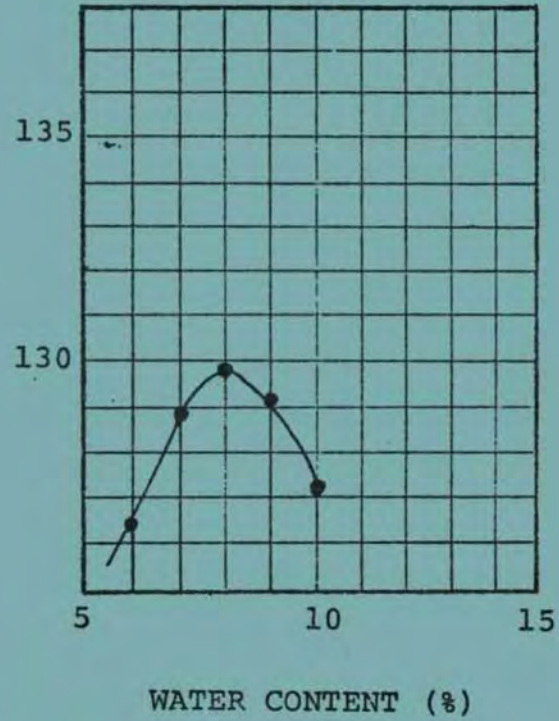


STRAIN (in/in)

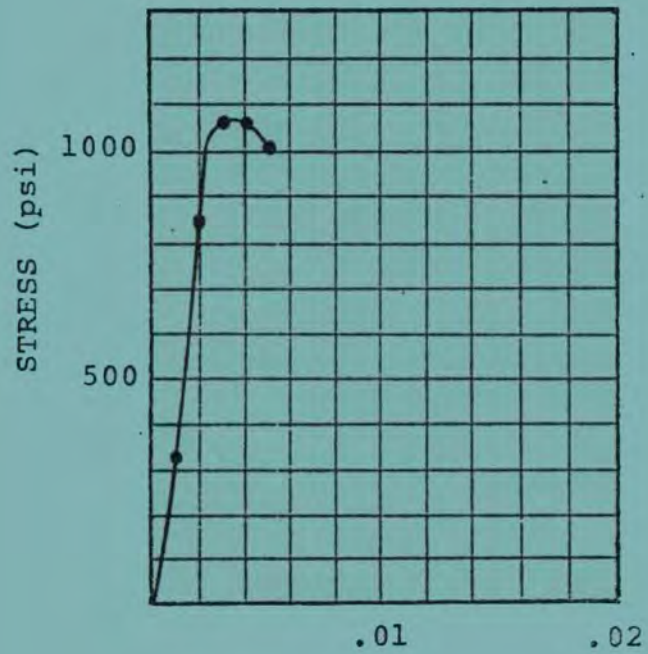
46.5% Stone
46.5% Sand
7% Stone Fines
8% Cement



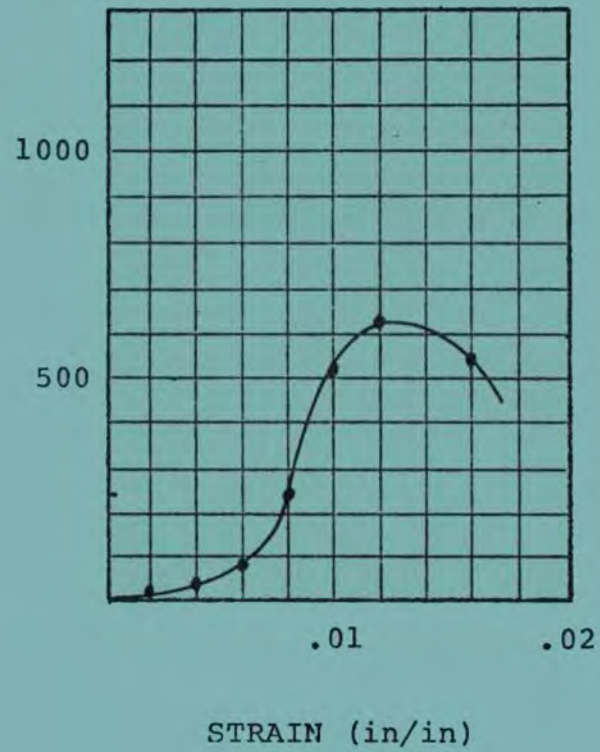
46.5% Stone
46.5% Sand
7% Stone Fines
6% Cement



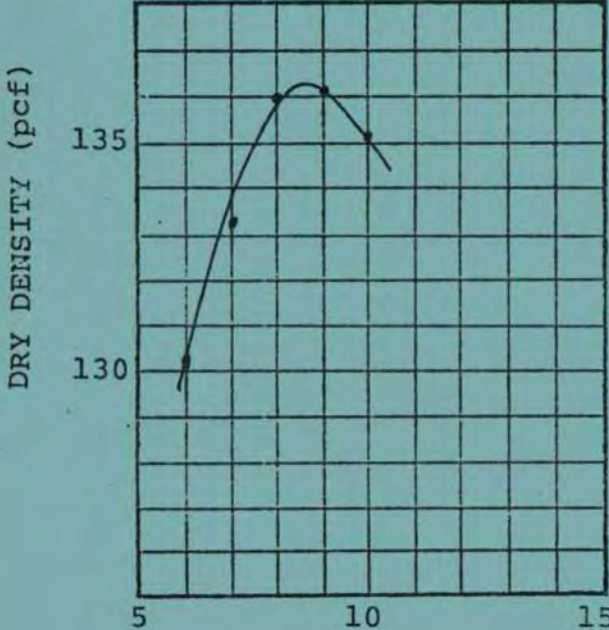
46.5% Stone
46.5% Sand
7% Stone Fines
8% Cement



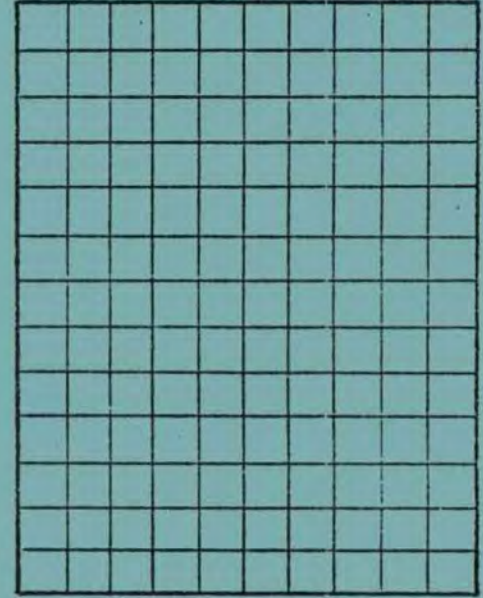
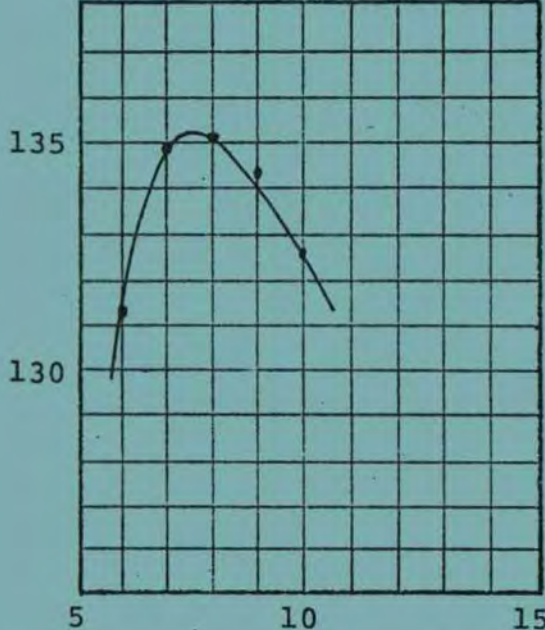
46.5% Stone
46.5% Sand
7% Stone Fines
6% Cement



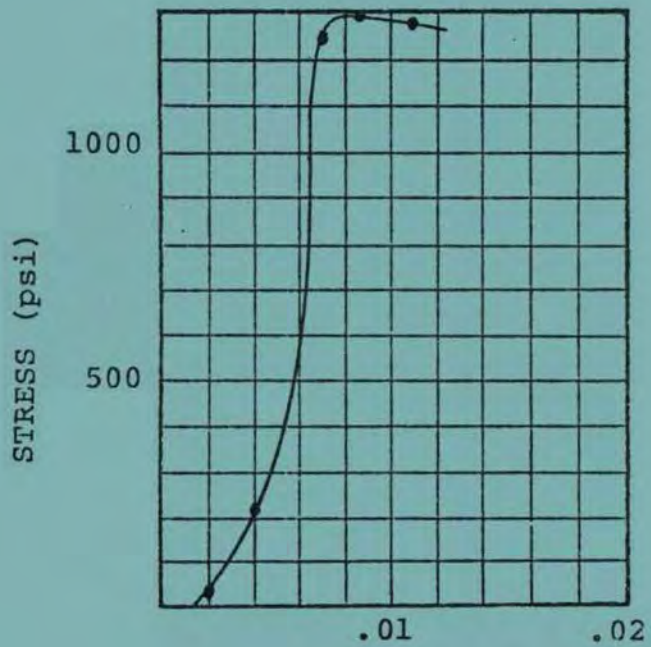
69.75% Stone
23.25% Sand
7% Stone Fines
8% Cement



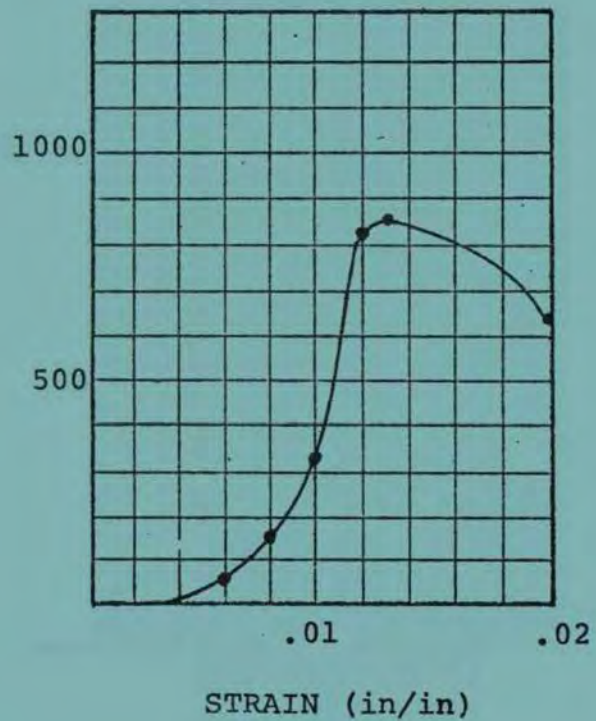
69.75% Stone
23.25% Sand
7% Stone Fines
6% Cement



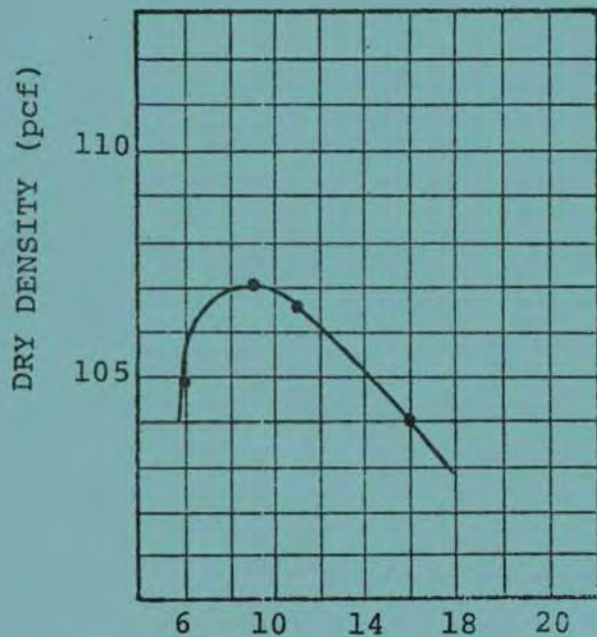
69.75% Stone
23.25% Sand
7% Stone Fines
8% Cement



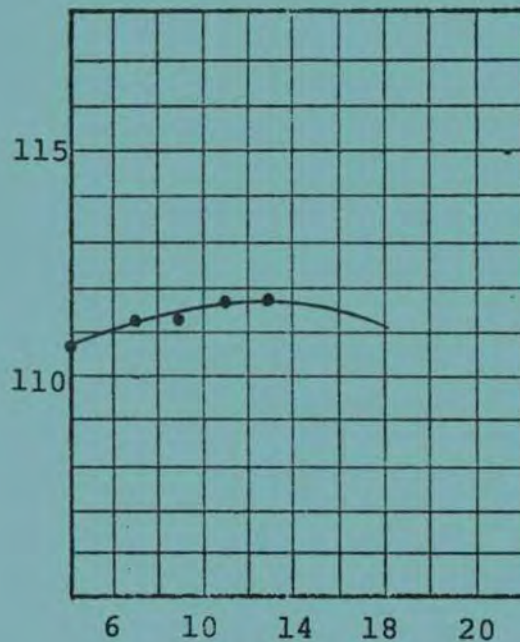
69.75% Stone
23.25% Sand
7% Stone Fines
6% Cement



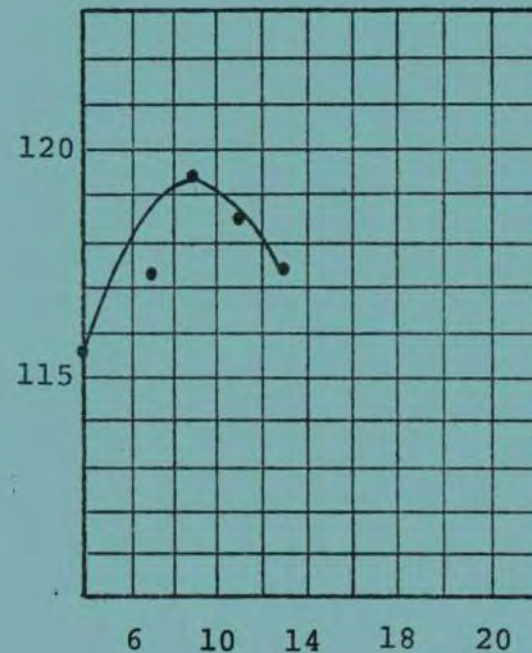
100% River Sand
6% Cement



75% River Sand
25% Norcross Stone
4% Cement

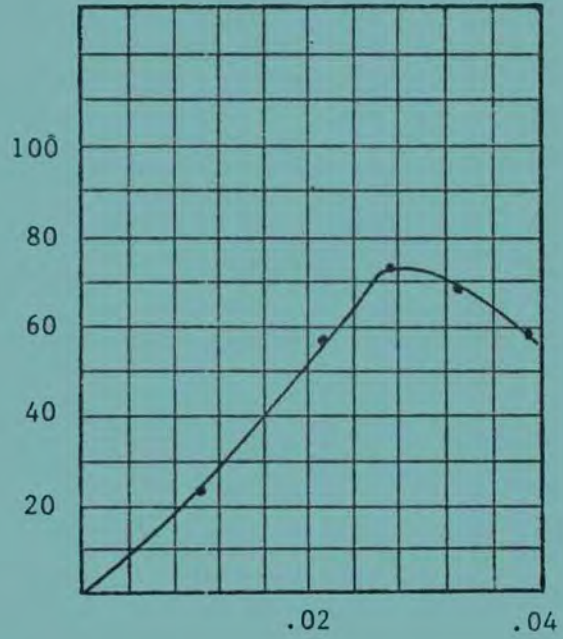


50% River Sand
50% Norcross Stone
4% Cement

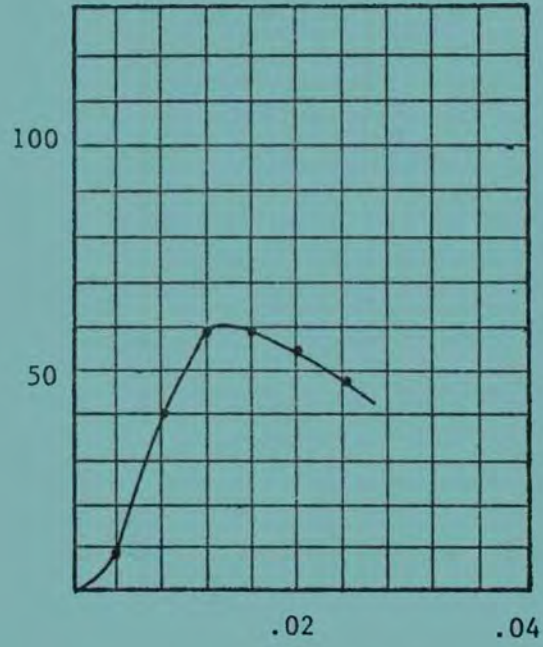


WATER CONTENT (%)

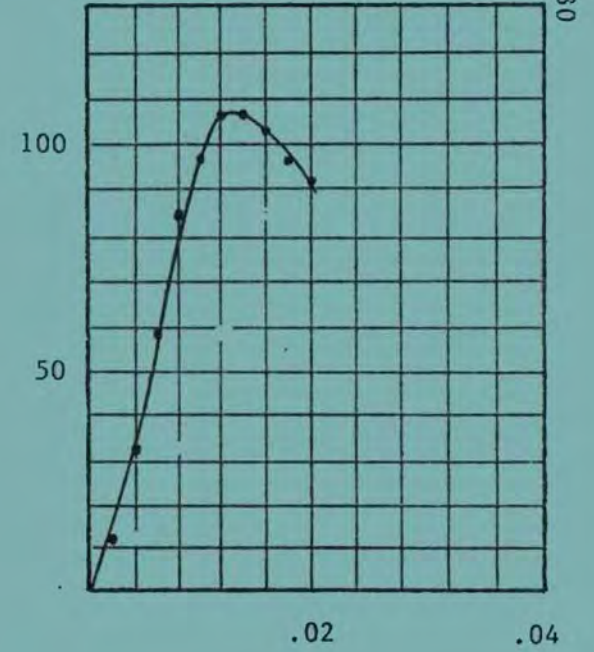
100% River Sand
6% Cement



75% River Sand
25% Norcross Stone
4% Cement



50% River Sand
50% Norcross Stone
4% Cement



STRAIN (in/in)

DS

180

APPENDIX B
DESIGN EXAMPLE

DESIGN EXAMPLE

The following design example is presented to illustrate the AASHTO Interim Guide Method. The data given in the design problem were taken from the Marianna Test Road [16].

DESIGN INFORMATION:

Traffic: 1.5×10^6 equivalent 18 kip axle loads

Subgrade: Compacted A-2-4 sand having a dynamic modulus of 12,000 - 22,000 psi (83,000 to 152,000 kN/m²); Field CBR value of 50 to 70.

Design: (a) A 3 in. (75 mm) asphalt concrete cover will be used consisting of a 1 in. (25 mm) asphalt concrete surfacing (2000 lb. Marshall Stability, 4.5 percent asphalt content).
(b) A sand-asphalt base wherein thickness is to be established (670 lb. Marshall Stability, 6.5 percent asphalt content).

STRUCTURAL DESIGN:

The structural design for the condition given above is as follows:

1. Soil Support Value: From Fig. 69 for a resilient modulus of 10,000 to 20,000 psi (70,000 to 140,000 kN/m²) the soil support value would be 6 to 8, and for a CBR value of 50 to 70, the soil support value would be approximately 7.5 to 8.0. Based on Table 27, however, the limiting soil support value for an A-2-4 soil is 5.0. Considering the very high modulus and CBR values measured for the subgrade, and the fact that the subgrade had only 2.9 percent fines passing the number 200 sieve, a design soil support value of 5.5 will be used.
2. Regional Factor: From Fig. 70, the regional factor is 1.0.
3. Weighted Structural Number: For the above information, a weighted structural number of 3.0 is obtained from Fig. 68 (PSI = 2.5).
4. Structural Coefficients: For the given construction, the weighted asphalt content of the surfacing and binder is [1 in. (5.5) + 2 in. (4.5)]/3 = 4.8%. From Table 29, $a_1 = 0.44$. The sand-asphalt base has an asphalt content of 6.5 percent and stability of 670 lbs. (298 N), therefore, from Table 29, $a_2 = 0.25$.
5. Sand-Asphalt Base Thickness: From step 3, the required weighted structural number was found to be 3.0, which must equal the right-hand side of equation 20, giving:

APPENDIX C
GUIDE SPECIFICATIONS
FOR
SAND-ASPHALT BASE CONSTRUCTION

Guide Specifications for Sand-Asphalt Base Construction

The following specifications are given for guidance in developing a suitable sand-asphalt mix design and detailed construction specifications. The specifications given in this section summarize the important aspects of sand-asphalt construction, but are not intended to be a complete, detailed set of construction specifications. These specifications are based on the results of the field inspections made in Florida, Georgia, Maryland, and South Carolina, the current practices of each state and the results of the laboratory fatigue and rutting tests performed on the sand-asphalt and sand-stone blend asphalt concrete mixes. These specifications are felt to reflect reasonably well the current state of the art in the design and construction of sand-asphalt bases.

Asphalt Cement: AC-20 viscosity grade
2,000 to 2,400 poises at 140°C ¹
300 poises at 275°

Add silicone to the asphalt cement at the rate of 25 cc of silicone to each 5,000 gal. of asphalt cement. Blending of the silicone mixture shall be done prior to shipment.

Aggregate: The aggregate shall be composed of local sands, a blend of local sand or a local sand and an additive such as mineral filler, commercial sand, or crushed stone screenings or other approved materials.

Sand: The local sand shall be sharp and non-plastic, shall contain not more than seven percent by weight of clay, shall be composed of hard, durable grains free of loam, roots, and other deleterious substances, and shall be suitable for use in a bituminous mix, as determined by laboratory tests.

If the local sand deposit consists of stratified layers of varying characteristics and gradation, the Contractor shall employ such means as necessary (such as vertical excavation) to secure a uniform blend. Should the loss of fines during drying operations be such that the stability of the mixture is reduced below the minimum specified, the Contractor shall add mineral filler or other approved material in such quantities as necessary to compensate for the loss in stability. Any clay present shall be of a type which will not produce clay

¹ Asphalt cements with viscosities less than 2,000 poises may tear during the rolling operation. If experience with the asphalt cement to be used has shown that tearing will not occur during construction, the allowable viscosity range can be increased to 1,500 to 2,400 poises at 140°C.

balls in the mixture that cannot be removed by scalping at the plant.

Mineral Filler: Mineral filler is not required unless needed to produce the specified stability. The mineral filler-aggregate blend shall meet the gradation requirements given for sand. Mineral filler shall be thoroughly dry and free from lumps and shall meet the following gradation:

| Sieve Size | Minimum Percent Passing by Dry Weight |
|------------|---|
| 30 | 100 |
| 100 | 90 |
| 200 | 65 |

Mix Proportions: The prescribed constituents shall be combined in such proportions as to produce a mixture conforming with the following composition limits by weight:

| | |
|-------------------|--------|
| Mineral Aggregate | 92-96% |
| Asphalt Cement | 4-8 % |

For highly absorptive aggregate, the upper limit of asphalt content can be raised by not more than 2 percent. The mixture shall meet the exact formula set up for the project within an allowable job tolerance of plus or minus 0.4 percent asphalt content. When combined in the proportions for the job mix formula, the sand equivalent shall be greater than 20 as determined by the AASHTO Designation T-176.

The combined aggregate for the sand-asphalt job mix shall satisfy the following gradation requirements:

| Sieve Size (Square Openings) | Total Passing (% by Weight) | Permissible Variation (% by Weight of Total Mix) |
|---------------------------------|--------------------------------|---|
| 3/4 in. | 100 | - |
| 3/8 in. | 85 - 100 | ± 5.0 |
| No. 8 | 60 - 100 | ± 12.0 |
| No. 30 | 25 - 100 | ± 8.0 |
| No. 200 | 3 - 15 | 3.0 |

The mixture shall be spread only when the air temperature in the shade away from a heat source is 40°F or more and rising, and there is no evidence of a frozen base. All aggregates to be blended or proportioned shall be placed in separate bins at the cold hopper and proportional by means of securely positioned calibrated gates or other approved devices.

Mineral filler, if required, shall be weighed-in separate from the other aggregates. The asphalt cement and the aggregates shall be heated and dried before screening to between 260°F and 325°F.¹ The temperature of the mix at the time of placing shall be within 30°F of the mixing temperature.

Mix Design: The sand-asphalt and sand-stone blend asphalt concrete mixes shall be designed using the 50 blow Marshall mix design method as specified by ASTM D-1559 test procedure. The required Marshall stability and air voids for each class of construction is as follows:

| Type | Marshall Stability (lbs.) | Required Field Compaction | Air Voids (% total mix) | Comments |
|------|---------------------------|---------------------------|-------------------------|--|
| 1 | 1000 | 96 | 6 | Sand-crushed stone blends; high traffic; placed beneath tin surfacings |
| 2 | 600 | 96 | 14 | Moderate traffic conditions; placed beneath 1 1/2 to 3 in. surfacing |
| 3 | 350 | 95 | 18 | Placed beneath 5 to 8 in. asphalt concrete for moderate to high traffic; light traffic such as subdivisions used beneath 1.5 in. to 2.5 in. asphalt concrete |

Compaction of Sand-Asphalt Mixture

The sand-asphalt base material shall be compacted in lifts of 3 to 4 in. (75 to 100 mm); lift thicknesses up to 6 in. (150 mm) can be used if the required grade and smoothness is obtained. Sand-stone blend asphalt concrete can be compacted in lifts up to 6 in. (150 mm) in thickness. Each lift shall be compacted to the density given in the above table according to the type mix.

For each paving train in operation, the contractor shall furnish a separate set of rollers with operators. The rolling shall be performed

¹The Asphalt Institute [71] has found that due to lower laying temperatures, in some cases it may be necessary to reduce dryer temperatures and lower the angle of the dryer to hold material in it for a longer period of time. Sometimes parallel dryers are used because of reduced temperatures.

in the sequence indicated unless otherwise permitted by the Engineer:

1. At least one unballasted 8 to 12 ton (70 to 110 kN) two-axle tandem steel wheel roller shall be used for the initial breakdown rolling operation.
2. Two self-propelled pneumatic tired rollers equipped with at least seven smooth tread pneumatic tires of equal size which are capable of exerting average contact pressures varying from 40 psi (280 kN/m²) (unballasted) to 100 psi (690 kN/m²) with ballast. The wheels of the roller shall be spaced so that one pass will accomplish one complete coverage equal to the width of the roller. There shall be a minimum of 0.25 in. (6 mm) overlap of the tracking wheels. The wheels shall oscillate but not wobble. The weight of the roller should be adjusted to the maximum that can be used without undue rutting of the pavement, probably 40 to 60 psi (290 to 410 kN/m²).

After the longitudinal joints and edges have been compacted, rolling shall start longitudinally at the sides and gradually progress toward the center of the pavement. This holds true except on super-elevated curves where rolling shall begin on the low side and progress to the high side, overlapping on successive trips by at least one-half the width of tandem rollers. The rollers shall move at a slow but uniform speed with the drive roll or wheel nearest the paver. The speed shall not exceed 2 mph (1.2 km/hr) for steel-wheeled rollers or 4 mph (2.4 km/hr) for pneumatic-tired rollers. The line of rolling shall not be changed suddenly or the direction of rolling suddenly reversed. If rolling causes upward displacement of the material, the affected areas shall be loosened at once with lutes or shovels and the loose material restored to the original grade before being rolled. Heavy equipment or rollers shall not be permitted to stand on the finished surface before it has been compacted and has thoroughly cooled.

When paving in single width, the first lane placed shall be rolled in the following order:

1. Transverse joints
2. Outside edge
3. Initial breakdown rolling, beginning on the low side and progressing toward the high side
4. Second and final compaction rolling, same procedure as (3)
5. Finish rolling

Transverse and longitudinal joints shall be carefully constructed and thoroughly compacted to provide a smooth riding surface.

Care shall be exercised in consolidating the course being laid along the entire length of the edges. Before it is compacted, the material along unsupported edges shall be slightly elevated with a tamping tool or lute. This will permit the full weight of the roller wheel to bear on the material to the extreme edges of the mat. In rolling pavement

edges, roller wheels shall extend 2 to 4 in. (50 to 100 mm) beyond the pavement edge. Only experienced roller operators shall be used for this work.

The rolling procedure found by Maryland to be most effective is suggested as a starting point for field compaction. The equipment used, rolling sequence and densities obtained have been previously described in Chapter 8. For moderate to large jobs, this sequence would be as follows:

Breakdown Rolling. Breakdown rolling shall immediately follow the rolling of the longitudinal joint and edges. Rollers shall be operated as close to the paver as possible without causing undue displacement. The breakdown roller shall be operated with the drive roll or wheel nearest the finishing machine. Exceptions may be made by the Engineer when working on steep slopes or super-elevated curves. When both pneumatic-tired rollers and tandem rollers are used, the tandem rollers shall work directly behind the paver followed by the pneumatic-tired rollers. The two-axle tandem roller shall do the breakdown rolling.

Second Rolling. At least two passes of the pneumatic-tired roller previously described shall be used for the second rolling. The second rolling shall follow the breakdown rolling as closely as possible and while the paving mix is still at a temperature that will result in maximum density from this operation. The contact pressure shall be 40 to 60 psi (280 to 400 kN/m²) and shall not cause displacement of the mix that cannot be remedied by the additional rolling sequence. This rolling shall be followed by one pass with the steel wheel roller to smooth the surface.

Final Compaction. At least two passes of the second pneumatic-tired roller shall be applied at a tire pressure between 60 and 80 psi (400 and 550 kN/m²).

Finish Rolling. Finish rolling is done solely for the purpose of smoothing the surface. It should be accomplished using two-axle tandems or three-axle tandems steel wheel rollers while the sand-asphalt is still warm enough for the removal of roller marks.

REFERENCES

1. Martin, R., and Hicks, R. G., *Evaluation of Offshore Sand and Gravel Deposits as Construction or Specialty Materials*, Georgia Marine Science Center, University System of Georgia, Technical Report 75-3, January (1975).
2. Walker, F. K., "Utilization of Marginal Materials in Georgia Asphalt Pavements," *Proceedings*, The Association of Asphalt Paving Technologists, Vol. 45 (1976).
3. Harding, J. L., and Woolsey, J. R., *Exploration Techniques for Aggregate Resources in Coastal Georgia*, Georgia Marine Science Center, University System of Georgia, Technical Report Series No. 75-1, January (1975).
4. *Application for Department of the Army Permits for Activities in Waterways*, U. S. Army Corps of Engineers, Savannah District, Georgia (1974).
5. Rogers, C. F., Cashell, H. D., and Irick, P. E., "Nationwide Survey of Pavement Terminal Serviceability," *Highway Research Board*, Research Record 42 (1963), pp. 26-40.
6. Barksdale, R. D., "Practical Application of Fatigue and Rutting Tests on Bituminous Base Mixes," Paper presented at the 1978 Annual AAPT Meeting, Lake Buena Vista, Florida, February (1978).
7. Epps, J. S., and Monismith, C. L., "Influence of Mixture Variables on the Flexural Fatigue Properties of Asphalt Concrete," *Proceedings of the A.A.P.T.*, Volume 38 (1969), pp. 423-464.
8. Pell, P. S., "Fatigue of Asphalt Pavement Mixes," *Proceedings of the 2nd International Conference on the Structural Design of Asphalt Pavements*, Ann Arbor (1967), pp. 577-593.
9. Niehoff, J. W., *An Investigation Into the Potential Use of Marginal River Sands as Base Materials*, M.S. Research Report, School of Civil Engineering, Georgia Institute of Technology, August (1977), 140 pp.
10. Miller, R. H., and Collins, R. J., "Waste Materials as Potential Replacements for Highway Aggregates," *Transportation Research Board*, NCHRP Report 166, Washington, D.C. (1966).
11. Marek, C. R., et al., "Promising Replacements to Conventional Aggregates for Highway Use," *NCHRP Report No. 135*, Highway Research Board, Washington, D.C. (1972).
12. Witczak, M. W., Lovell, C. W., and Yoder, E. J., "A Generalized Investigation of the Potential Availability of Aggregate by Regional Geomorphic Units Within the Conterminous Forty Eight States," *Highway Research Board*, HRR No. 353, Washington, D.C. (1971), pp. 31-42.

13. Walker, Frederick K., "Utilization of Marginal Materials in Georgia Asphalt Pavements," *Proceedings*, The Association of Asphalt Paving Technologists, Vol. 45 (1976).
14. Godfrey, K. A., "Sand and Gravel - Don't Take Them for Granted," *Civil Engineering*, Vol. 47, No. 3, March (1977), pp. 55-57.
15. Miller, R. H., and Collins, R. J., "Waste Materials as Potential Replacements for Highway Aggregates," *National Cooperative Highway Research Program*, Highway Research Board, Report No. 166, Washington, D.C. (1976).
16. Stromberg, F. J., *Investigation of Base Courses for Flexible Pavements*, Maryland State Highway Administration, Bureau of Research, Final Report, September (1972).
17. Barksdale, R. D., "Performance of Asphalt Concrete Pavements," *Proceedings of the ASCE*, TE1, January (1977), pp. 55-73.
18. *AASHO INTERIM GUIDE FOR DESIGN OF PAVEMENT STRUCTURES*, American Association of State Highway Officials, Washington, D.C. (1972).
19. Krukar, M., and Cook, J. C., *Evaluation and Analysis of Results from Experimental Rings No. 1-4*, Research Project Y-993, Washington State University, Pullman, Washington (1971).
20. Stuart, E., Miyaoka, Y., Skok, E. C., and Winck, C., "Field Evaluation of an Asphalt Stabilized Sand Pavement Using the Elastic Layered System," *Proceedings*, The Association of Asphalt Paving Technologists, Vol. 43 (1974).
21. Bushing, H. W., Roberts, F. L., Rostron, J. P., and Schwartz, A. E., *An Evaluation of the Relative Strength of Flexible Pavement Components*, Department of Civil Engineering, Clemson University, Research Project No. 522, May (1971).
22. Acott, S. M., *The Development and Mix-Design of Gap-Graded Asphalt*, National Institute for Road Research, Report RB/1/75, Pretoria, South Africa (1975).
23. Freeme, C. R., *The Selection of Sands Suitable for Gap-Graded Mixtures*, National Institute for Road Research, Report RB/2/75, Pretoria, South Africa (1975).
24. Marais, C. P., "Tentative Mix Design Criteria for Gap-Graded Bituminous Surfaces," *TRANSPORTATION RESEARCH BOARD*, Research Record (1975), pp. 132-137.
25. *Rolled Asphalt (Hot Process)*, British Standards Institution, BS594: 1961, London (1962).
26. Fuller, W. B., and Thompson, "The Laws of Proportioning Concrete," *Transactions*, American Society of Civil Engineers, Vol. 59 (1907).
27. Hveem, F. N., "Gradation of Mineral Aggregates for Dense Graded Bituminous Mixtures," *Proceedings*, Association of Asphalt Paving Technologists, Vol. 11, Chicago (1940), pp. 315-330.

28. Hudson, S. B., and Davis, R. L., "Relationship of Aggregate Voidage to Gradation," *Proceedings, Association of Asphalt Paving Technologists*, Vol. 34 (1965), pp. 574-593.
29. Lees, G., "The Rational Design of Aggregate Gradings for Dense Asphaltic Compositions," *Proceedings, Association of Asphalt Paving Technologist*, Vol. 39, Kansas City (1970).
30. Duthie, J. L., "The Marshall Stability Test in the Design of Hot Rolled Asphalt Wearing Courses," Paper presented at a joint meeting of the Institution of Highway Engineers and the Society of Chemical Industry (Road and Building Materials Group), South Africa, January (1974).
31. Heukelom, W., "The Role of Filler in Bituminous Mixtures," *Proceedings, The Association of Asphalt Paving Technologists*, Vol. 34 (1965), pp. 396-429.
32. Brien, D., "A Design Method for Gap-Graded Asphaltic Mixes," *Roads and Road Construction*, May (1972), pp. 140-145.
33. *Essentials of Soil-Cement Construction, Inspection, and Field Control*, Portland Cement Association, Skokie, Illinois (1969).
34. *Soil-Cement Construction Handbook*, Portland Cement Association, Chicago, Illinois (1956).
35. *Soil-Cement Laboratory Handbook*, Portland Cement Association, Engineering Bulletin (1971), 64 p.
36. Yedavally, S. P., and Anderson, K. O., "Simplified Soil-Cement Mix Design Method for Alberta Sands," Highway Research Record No. 379, *Highway Research Board*, Washington, D.C. (1972), pp. 1-9.
37. Zube, E., Gates, C. G., *Service Performance of Cement Treated Bases as Used in Composite Pavements*, California Division of Highways, Research Report M & R 633208, paper presented at Annual Highway Research Board Meeting, January (1969).
38. McGhee, K. H., *Pavement Design and Performance Studies*, Final Report on Phase C, Virginia Highway Research Council, Research Report BHRC 70-R44, May (1971).
39. Barksdale, R. D., and Vergnolle, R. R., "Expansive Cement Stabilization of Bases," *Highway Research Board*, Highway Research Record No. 255, Washington, D.C. (1968), pp. 30-41.
40. Wang, Jerry W. H., "Use of Additives and Expansive Cements for Shrinkage Crack Control in Soil-Cement: A Review," *Highway Research Board*, Highway Research Record No. 442, Washington, D.C. (1973), pp. 11-21.

41. Norling, L. T., "Minimizing Reflective Cracks in Soil-Cement Pavements: A Status Report of Laboratory Studies," *Transportation Research Board*, TRR 442 (1973), pp. 22-33.
42. Johnson, V. W., "Comparative Studies of Combinations of Treated and Untreated Bases and Subbases for Flexible Pavements," *Highway Research Board*, Bulletin No. 289.
43. *The AASHO Road Test*, Highway Research Board, Special Report 22, Publication 360, Washington, D.C. (1955).
44. Fossberg, P. E., Mitchell, J. K., and Monismith, C. L., "Cracking and Edge Loading Effects on Stresses and Deflections in a Soil-Cement Pavement," *Highway Research Board*, Highway Research Record No. 379, Washington, D.C. (1972), pp. 25-38.
45. Wang, Mian-Chang, Moulthrop, K., and Nacci, V. A., "Performance of Soil-Cement Test Pavement in Rhode Island," *Highway Research Board*, HRR No. 379, Washington, D.C. (1972), pp. 52-61.
46. Zube, E., and Gates, C. G., *Service Performance of Cement Treated Bases as Used in Composite Pavements*, California Division of Highways, Research Report M&R 633208, paper presented at Annual Highway Research Board Meeting, January (1969).
47. Burns, C. D., and Ahlvin, R. G., *Behavior of Full-Depth Bituminous Pavements Under Heavy Aircraft Loadings*, Waterways Experiment Station.
48. Phang, W. A., *Four Years Experience at the Trampton Test Road*, Department of Highway, Ontario (1969).
49. McNaughton, E., and Rand, E., *Peru Experimental Project, Interim Report*, State of Maine, Department of Transportation, Materials and Research Technical Paper 70-6P, April (1970).
50. Freeme, C. R., *The Selection of Sands Suitable for Gap-Graded Mixtures*, Internal Report No. RB/2/75, National Institute for Road Research, South Africa, Pretoria (1975).
51. *Soil-Cement Laboratory Handbook*, Portland Cement Association, Skokie, Illinois.
52. State Highway Department of Georgia, *Sampling, Testing and Inspection Manual*, Division of Materials and Tests.
53. The Asphalt Institute, *The Asphalt Handbook*, Manual Series No. 4, (MS-4) (1970).
54. Barksdale, R. D., and Miller, J. H., "Development of Equipment and Testing Techniques for Evaluating Fatigue and Rutting Characteristics of Asphalt Concrete Mixes, School of Civil Engineering,

Georgia Institute of Technology, Research Report SCEGIT-77-147, July (1977).

55. Van de Loo, P. J., "A Practical Approach to the Prediction of Rutting in Asphalt Pavements, The Shell Method," Paper presented at the Annual Meeting of the Transportation Research Board, Washington, D.C. (1976).
56. Van de Loo, P. J., "Creep Testing, A Simple Tool to Judge Asphalt Mix Stability," *Proceedings*, The Association of Asphalt Paving Technologists, Vol. 43 (1974).
57. Hofstra, A., and Klomp, A. J. G., "Permanent Deformation of Flexible Pavements Under Simulated Road Traffic Conditions," *Proceedings*, Third International Conference on the Structural Design of Asphalt Pavements, Vol. 1, London (1972), pp. 613-621.
58. Lister, N. W., Addis, R. R., "Field Observations of Rutting in Practical Implications," Paper prepared for Symposium on Rutting in Asphalt Concrete Pavements, Annual Meeting of the Transportation Research Board, Washington, D.C., January (1976).
59. Barber, E. S., "Calculation of Maximum Pavement Temperatures from Weather Reports," *Highway Research Board*, Bulletin No. 168 (1957).
60. Barksdale, R. E., *Repeated Load Test Evaluation of Base Course Materials*, Research Report, School of Civil Engineering, Georgia Institute of Technology, Atlanta, Georgia (1976).
61. *Summary Data Reports for Study P-1-63, Flexible Pavement Design*, Florida Department of Transportation, Research Report 196, Office of Materials and Research (1976).
62. Ngowtrakul, S., *Rutting in Asphalt Concrete*, Special Research Report, Georgia Institute of Technology, Atlanta, Georgia (1976).
63. Godwin, H. G., and McNamara, R. L., *Summary Data Reports for Study P-1-63 Flexible Pavement Design*, Florida Department of Transportation, Office of Materials and Research, Research Report No. 196, August (1976).
64. Schroeder, J. A., *Rutting in Sand Asphalt Bases*, School of Civil Engineering, Georgia Institute of Technology, Special Research Report, December (1977).
65. Verstraeten, J., Romain, J. E., and Veverka, V., "The Belgium Road Research Center's Overall Approach to Asphalt Pavement Structural Design," *Proceedings*, Fourth International Conference on Structural Design of Asphalt Pavements, Vol. 1, Ann Arbor, Michigan, August (1977), pp. 298-324.

66. Huschek, S., "Evaluation of Rutting Due to Viscous Flow in Asphalt Pavements," *Proceedings, Fourth International Conference on Structural Design of Asphalt Pavements, Vol. 1, Ann Arbor, Michigan, August (1977)*, pp. 297-508.
67. Robnett, Q. L., Lai, J. S., and Boley, D. L., *A Manual for Soil Stabilization in Bikeway Construction*, School of Civil Engineering, Georgia Institute of Technology, SCEGIT-77-153, September (1977), 120 p.
68. Blight, G. E., Stewart, J. A., and Theron, P. F., "Effects of Soluble Salt on Performance of Asphalt," *Proceedings, The Second Conference on Asphalt Pavements for Southern Africa, Durban, South Africa (1974)*, pp. 3-1 through 3-13.
69. DESIGN AND CONTROL OF CONCRETE MIXES, Portland Cement Association, Skokie, Illinois, July (1968), pp. 19-20.
70. Briscoe, O. E., *Deep Lift Construction on Maryland's State Roads*, Maryland Department of Transportation, Research Report, Paper presented at the 6th Annual Maryland Asphalt Paving Conference, College Park, Maryland, February (1968).
71. *Hot-Mix Sand Asphalt Base*, The Asphalt Institute, Misc.-67-2, College Park, Maryland, March (1967).

