TRANSPORTATION RESEARCH RECORD 741

Performance of Pavements Designed with Low-Cost Materials

TRANSPORTATION RESEARCH BOARD

COMMISSION ON SOCIOTECHNICAL SYSTEMS
NATIONAL RESEARCH COUNCIL

NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1980

Transportation Research Record 741
Price $4.20
Edited for TRB by Susan Singer-Bart

modes
1 highway transportation
4 air transportation

subject areas
24 pavement design and performance
33 construction

Library of Congress Cataloging in Publication Data
National Research Council. Transportation Research Board.
Performance of pavements design with low cost materials.

(Transportation research record; 741)
Reports prepared for the 59th annual meeting of the Transportation Research Board.
1. Pavements–Design and construction–Congresses.
2. Road materials–Congresses. I. Title. II. Series.
TE7.H5 no. 741 [TE251] 380.5s [625.8] 80-17917
ISBN 0-309-02999-6 ISSN 0361-1981

Sponsorship of the Papers in This Transportation Research Record

GROUP 2–DESIGN AND CONSTRUCTION OF TRANSPORTATION FACILITIES
R. V. LeClerc, Washington State Department of Transportation, chairman

Pavement Design Section
W. Ronald Hudson, University of Texas at Austin, chairman

Committee on Strength and Deformation Characteristics of Pavement Sections
Amir N. Hanna, Portland Cement Association, chairman
Richard D. Barksdale, Stephen F. Brown, Gaylord Cumberledge, Jim W. Hall, Jr., R.G. Hicks, Ignat V. Kalcheff, William J. Kenis, Thomas W. Kennedy, Erland Lukanen, Kamran Majidzadeh, Lutfi Raad, J. Brent Rauhut, Quentin L. Robnett, Jatinder Sharma, Gary Wayne Sharpe, James F. Shook, Eugene L. Skok, Jr., T.C. Paul Teng

Lawrence F. Spaine, Transportation Research Board staff

The organizational units and officers and members are as of December 31, 1979.

Contents

Authors of the Papers in This Record

Barksdale, Richard D., School of Civil Engineering, Georgia Institute of Technology, Atlanta, GA 30332
Grau, Robert W., Geotechnical Laboratory, U.S. Army Engineer Waterways Experiment Station, P.O. Box 631, Vicksburg, MS 39180
Haas, Ralph, University of Waterloo, Waterloo, Ontario, Canada
Hicks, R. G., Department of Civil Engineering, Oregon State University, Corvallis, OR 97331
Huddleston, Ira J., U.S. Forest Service, Willamette National Forest, 1509 West First Street, Eugene, OR 97402
Kalcheff, Ignat V., National Crushed Stone Association, 1415 Elliot Place, N.W., Washington, DC 20007
Kennedy, Thomas W., College of Engineering, University of Texas, Austin, TX 78712
Machemehl, Charles A., Jr., Vulcan Materials Company, P.O. Box 80730, Atlanta, GA 30366
Packard, R. G., Portland Cement Association, 5410 Old Orchard Road, Skokie, IL 60077
Potts, Charles F., Office of Materials and Research, Florida Department of Transportation, P.O. Box 1029, Gainesville, FL 32602
Ruth, Byron E., University of Florida, Gainesville, FL 32601
Smith, Lawrence L., Office of Materials and Research, Florida Department of Transportation, P.O. Box 1029, Gainesville, FL 32602
Vinson, Ted S., Department of Civil Engineering, Oregon State University, Corvallis, OR 97331
Yrjanson, W. A., American Concrete Paving Association, 2625 Clearbrook Drive, Arlington Heights, IL 60005

Utilization of Marginal Aggregate Materials for Secondary Road Surface Layers

Robert W. Grau

The purpose of this study was to develop methodology and procedures for the use of marginal-quality aggregates (aggregates that do not meet existing specifications) in the construction of surface pavement layers of asphalt and portland cement concrete. This paper describes the properties of the bituminous and zero-slump mixes made from marginal aggregate materials and the testing of a specially designed test section, which consists of both flexible and rigid pavement surface layers made of marginal-quality aggregate. Also presented are construction techniques and the behavior of the marginal aggregate mixes subjected to accelerated 4.5-Mg (5-ton) military dump truck and two types of tracked vehicle traffic. Recommended properties of asphalt and portland cement concrete mixes made of marginal aggregate materials are presented to aid the engineer in constructing paved surfaces that are to be used only for limited traffic and short periods of service life.

The cost of constructing pavements is continually rising due to increasing cost of labor, materials, and equipment. Current U.S. Army Corps of Engineers design procedures allow only the use of high-quality aggregates in asphaltic concrete (AC) and portland cement concrete (PCC) mixtures. Many military operations require paved surfaces for roads, airfields, heliports, and parking aprons, which have only limited traffic and short periods of service life. Therefore, if satisfactory AC and PCC pavement mixes can be made by using marginal materials (aggregates) that do not meet existing specifications but withstand limited traffic for short periods of service life, a considerable savings in construction cost can be realized.

To study the concept of using marginal materials in AC and PCC pavement surface layers, two specially designed test sections were constructed and subjected to accelerated test traffic. The primary test vehicle was an M51 4.5-Mg (5-ton) dump truck that was operated at gross loads of 18 583, 18 685, 19 936, and 22 114 kg (40 920, 41 145, 43 900, and 48 695 lb). A limited amount of tracked-type vehicle traffic was applied to both test sections by using an M113 armored personnel carrier and an M48A1 tank at gross weights of 8628 kg (19 000 lb) and 46 871 kg (103 210 lb), respectively.

DESIGN AND CONSTRUCTION OF THE TEST SECTIONS

The rigid and flexible pavement test sections were designed to determine the effects of traffic during various weather conditions on the PCC and AC pavement surface mixes made of marginal materials. Both test sections were constructed of sufficient thicknesses to prevent base course or subgrade failures during the traffic period. A plan and profile of the rigid and flexible pavement test sections are shown in Figures 1 and 2, respectively.

The subgrade for both test sections was classified as a lean clay according to the Unified Soil Classification System and had a liquid limit (LL) of 34 and a plasticity index (PI) of 12. The average strength of the subgrade for the flexible pavement test sections was about 25 California bearing ratio (CBR). The base course material for the two test sections was classified as a gravelly-clayey sand that had a LL of 37 and a PI of 24. The 15-cm (6-in) thick gravelly-clayey sand base material for the flexible pavement text section was stabilized with 4 percent lime, which resulted in a strength of above 150 CBR. Plate bearing and thickness measurements made on the finished unstabilized base material for the rigid pavement test section indicate that the average K-value 15.9 kg/cm^2 (575 lb/in^3) and that the thickness was about 10.2 cm (4 in).

Zero-Slump Concrete Mixes and Placement Procedures

A mix design for five zero-slump concrete mixes was prepared by using four different aggregate materials (designated S2, G1, G2, and G3) and type 1 portland cement. Classification data for these materials are shown in Figure 3. Mix 1 was designed according to American Concrete Institute (ACI) standard 211.13-75 by using aggregates S2 and G3 and was placed in test items 1 and 2. This mix was designed as a high-quality concrete mix and had a 28-day flexural strength of 5.17 MPa (750 lbf/in^2). Mixes 2-5 were designed by using the same cement factor [306.7 kg/m^3 (517 lb/yd^3)], as for mix 1. Aggregate materials G2, S2, G3, and G1 were used in mixes 2, 3, 4, and 5, respectively. Mixes 2-5 were placed in test items 3-6, respectively. Properties of the in-place mixes are shown in Table 1.

The concrete in test item 1 was placed by end dumping the mix onto the surface of the base and then spreading the mix with a motor grader. The concrete in all other test items was placed with an asphalt finisher. A 11 353-kg (25 000-lb) tandem vibratory roller was used to compact all mixes.

AC Mixes and Placement Procedures

The primary variables in the flexible pavement's 13 test items were the asphalt content and the aggregates used in the mixes. An 85-100 penetration grade asphalt was used in all of the mixes. The AC mix placed in item 1 was a standard mix that used 1.9-cm (0.75-in) maximum-size crushed limestone minus 4.75-mm (No. 4) limestone screenings and sand filler. A gradation curve of the blended stockpile aggregates used for this bituminous mixture is shown as curve 1 in Figure 4. This gradation meets requirements for conventional bituminous concrete to be used for roads, streets, and heliports or airfields that are not subjected to fuel spillage or to traffic by aircraft that have high-pressure tires [greater than 0.69 MPa (100 lbf/in^2)]. The bituminous mixes placed in items 2-13 were made from either one of four basic marginal materials or blends of these materials. Classification data of each of these bituminous mix aggregates are shown in Figure 4.

The optimum asphalt contents, as determined by the Marshall design criteria for the marginal materials, appeared to be slightly high for the sand mixes. This is

Figure 1. Layout of rigid pavement test section.

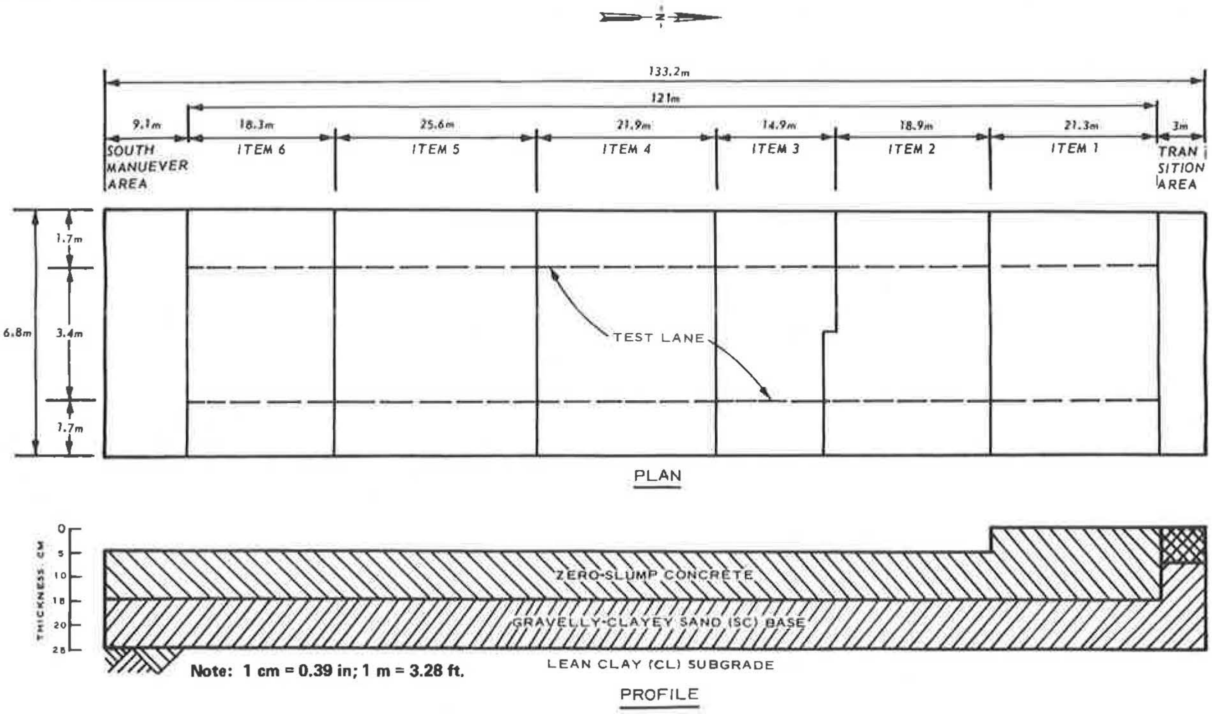
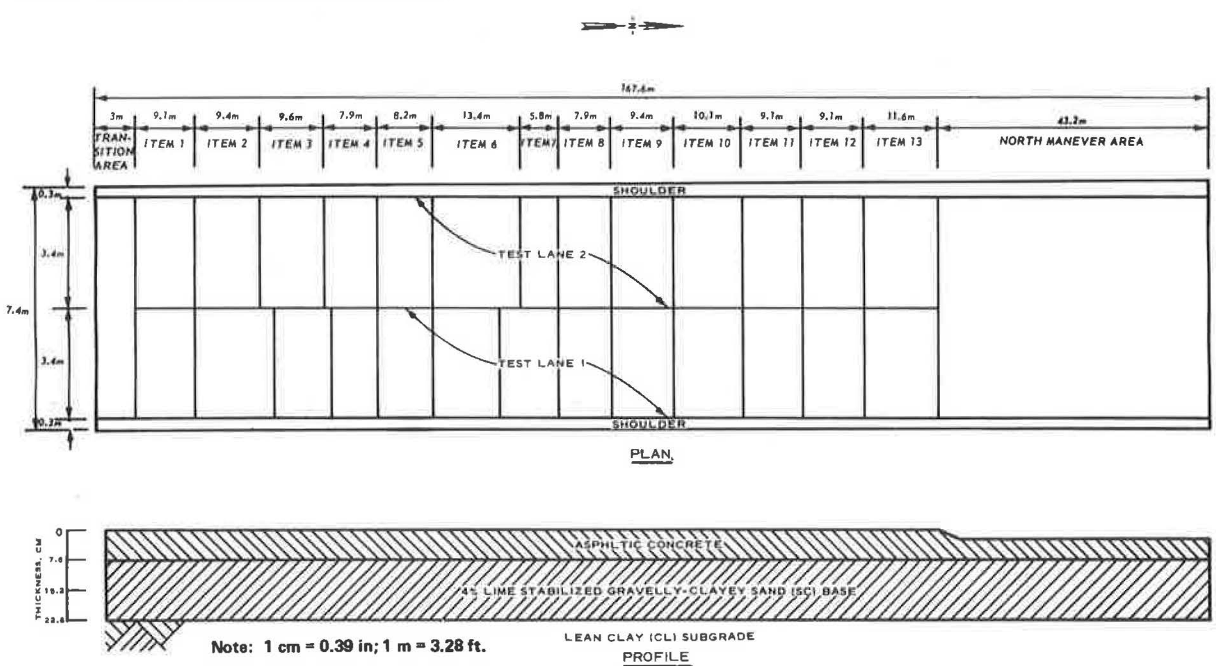


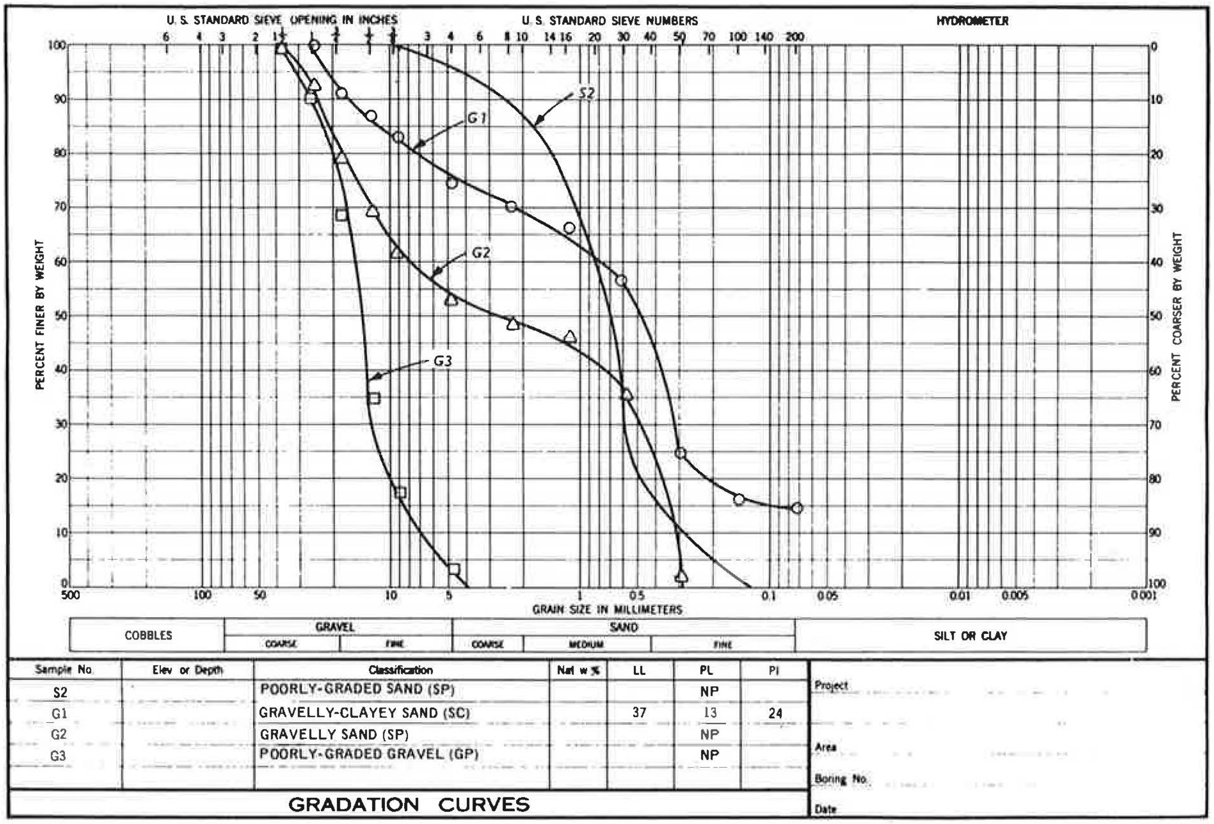
Figure 2. Layout of flexible pavement test section.



caused by the high percentage of voids in the mineral aggregates and the voids criteria used with the Marshall method of design. It was felt that the proper asphalt content for the sand mixes should be just enough to coat the aggregate particles plus the asphalt that would be absorbed in the aggregate pores. Therefore, the asphalt content required to provide the amount absorbed and a 6-micron film thickness based on the surface area of the aggregate were calculated for each mix. A comparison of the optimum asphalt content as determined by the Marshall design procedure and the surface area method for the various materials used in the test section plus the actual asphalt contents and resulting stabilities of the various mixes used in each of the test items is given in Table 2.

Conventional placement and compaction equipment was used during construction of the flexible pavement test section. In some instances, initial efforts to compact the mixes at normal rolling temperatures of 139°C (250°F) or higher were unsuccessful because of the low

Figure 3. Grading curves of zero-slump mix aggregates.



NOTE: 1 cm = 0.39 in.

Table 1. Properties of in-place zero-slump concrete mixes.

Test Item	Mixture Number	Cement Factor (kg/m³)	W/C Ratio	S/A Ratio	28-Day Field Flexural Strength (MPa)
1	1	306.7	0.32	0.32	5.17
2	1	306.7	0.32	0.32	5.17
3	2	306.7	0.37	0.53	4.72
4	3	306.7	0.67	1.00	4.07
5	4	306.7	0.25	0.0	2.41
6	5	306.7	0.57	0.67	0.86

Note: 1 kg/m³ = 1.7 lb/yd³; 1 MPa = 145.04 lbf/in².

stability of the mix. Therefore, the mix was allowed to cool before rolling. The rolling temperatures of the various marginal material mixes ranged from about 72°C to 133°C (130°F to 240°F).

PERFORMANCE OF TEST SECTIONS DURING TRAFFIC

Traffic tests were performed on one lane located in the center of the rigid pavement test section and on two separate lanes of the flexible pavement test section. Lane 1 of the flexible pavement test section was trafficked only when the pavement temperature was 44°C (80°F) or lower, and lane 2 was trafficked when the pavement temperature was 44°C (80°F) or higher.

Rigid Pavement

The three failure conditions (initial crack, shattered slab, and complete) used for judging plain, nonreinforced rigid pavements were used to judge the performances of the zero-slump mixes during traffic. The total amount of test traffic applied to the test section is shown in Table 3.

Test results indicate that the zero-slump concrete mixes made of pit-run gravelly sand, poorly graded sand, and gravelly-clayey sand performed very well during traffic. The mixes made from these materials were placed in items 3, 4, and 6. Only slight minor cracking was observed in any of these items after traffic. However, note that the surface of item 6 tended to become slippery during wet-weather traffic. Item 6 contained the mix made from the highly plastic gravelly-clayey sand.

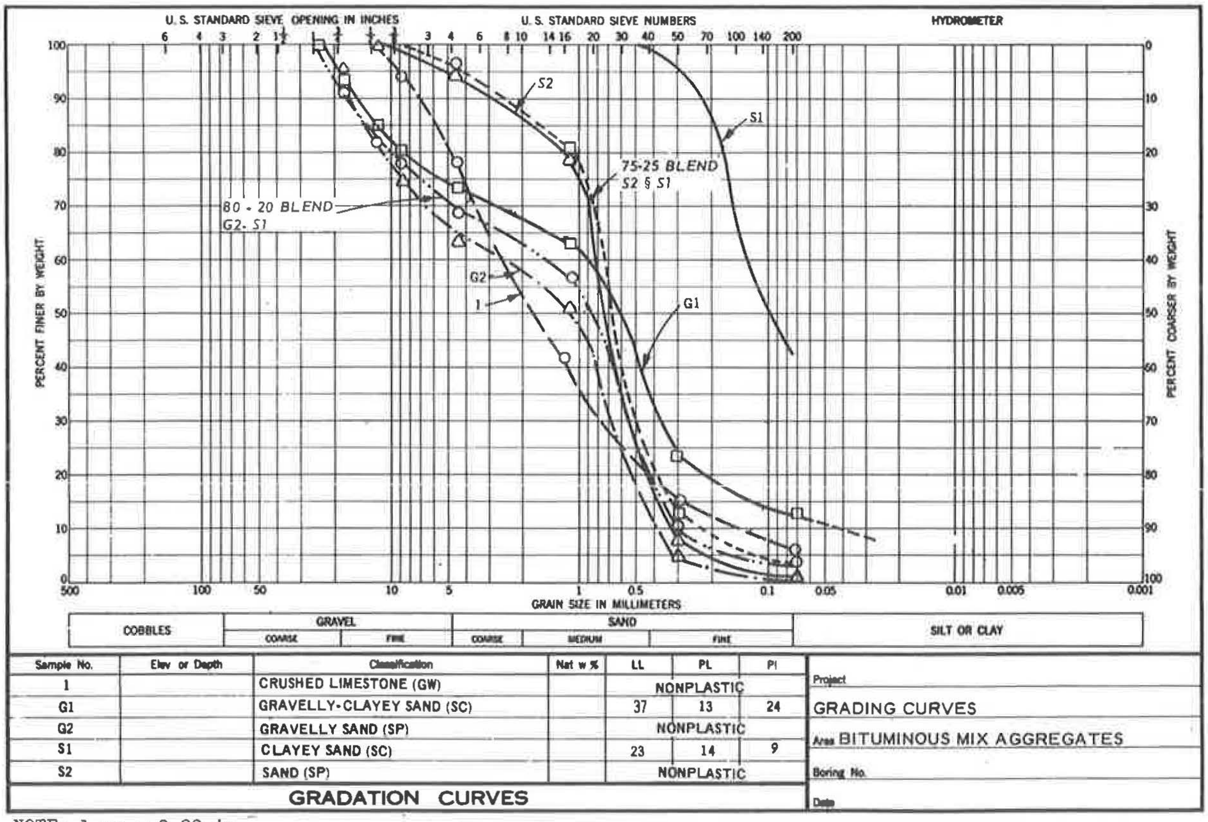
The mix made with the open-graded gravel (G3) and placed in item 5 failed because of severe raveling. Only 5390 operations were applied to this item before failure, as compared with about 125 000 operations applied to items 3, 4, and 6 without a failure.

The standard zero-slump mix placed in item 2 failed after 18 245 operations. However, it is believed that the poor behavior of this mix was caused by construction problems.

Although the placement procedures resulted in the surface of item 1 being slightly rougher than the surface of items 2-6, the riding quality of item 1 was satisfactory for truck traffic. The surface of item 1 was rougher because the mix in this item was placed by a motor grader under adverse conditions as compared with placement by an asphalt finisher in all other items.

After the M51 4.5-Mg (5-ton) dump truck traffic, track-type vehicle traffic was applied to the test section. There was no distress observed in items 1, 3, 4, and 6 after 130 operations (326 passes) of an M113 personnel

Figure 4. Grading curves of bituminous mix aggregates.



NOTE: 1 cm = 0.39 in.

Table 2. Asphalt contents and stabilities of the marginal material bituminous mixes.

Material	Test Item	Asphalt Content (%): Optimum by Marshall Mix Design	By Surface Area Method	Used in Test Item	Stability (N)
Crushed limestone, 1	1	5.7	5.5	5.8	6601
Gravelly sand, G2	2	7.5	5.0	7.6	1201
	3	7.5	5.0	6.7	721
	4	7.5	5.0	5.4	663
Gravelly-clayey sand, G1	5	9.0	8.7	5.5	3180
	6	9.0	8.7	5.2	2976
	7	9.0	8.7	4.3	3140
	13	9.0	8.7	6.7	3069
80 percent G2	8	6.2	7.0	6.2	1268
20 percent S1	9	6.2	7.0	5.6	3011
Concrete sand, S2	10	9.8	5.2	6.4	320
75 percent S2	11	7.8	7.4	7.4	894
25 percent S1	12	7.8	7.4	5.9	623

Note: 1 N = 4.4 lbf.

carrier and 50 000 operations (20 passes) of an M48A1 tank. Since items 2 and 5 were considered failed prior to track-type traffic, the performance of these items during traffic was not considered.

Flexible Pavement

Since hot bituminous mixes are placed on base courses (a) to provide a smooth riding surface, (b) to waterproof the base against the penetration of surface water, and (c) to protect the base from the raveling effects of traffic, a test item was considered failed when any of the following conditions occurred:

1. Surface depressions of 5.1 cm (2 in) or more,
2. Surface cracking to the extent that the pavement was no longer waterproof,
3. Severe surface raveling to significant depths (for these tests, 5.1 cm or greater), or
4. Severe shoving (for these tests, resulting rut depths of 5.1 cm or more).

The total amount of test traffic applied to test lanes 1 and 2 is given in Table 4.

Test results indicate that the bituminous mixes made with the gravelly-sand aggregate (items 2-4) performed as well structurally under pneumatic-tired traffic as did the bituminous mix made with high-quality crushed

Table 3. Summary of test traffic applied to the rigid pavement test section.

Test Vehicle	Gross Weight (kg)	Number of Passes	Equivalent 80-kN S-Axle Load Operations
M51	18 685	1750	27 222
M51	22 114	9906	528 320
M113	8 629	326	578
M48A1	46 871	20	222 222

Note: 1 kg = 2.202 lb; 1 kN = 0.225 kips.

Table 4. Summary of test traffic applied to the flexible pavement test section.

		Lane 1		Lane 2	
Test Vehicle	Gross Weight (kg)	Number of Passes	Equivalent 80-kN S-Axle Load Operations	Number of Passes	Equivalent 80-kN S-Axle Load Operations
M51	18 583	5008	63 658		
M51	18 685	1750	27 222		
M51	19 936	310	7 000		
M51	22 114	1010	53 867	8896	474 453
M113	8 629	326	578	326	578
M48A1	46 871	20	222 222	20	222 222

Note: 1 kg = 2.202 lb; 1 kN = 0.225 kips.

stone (item 1). However, during the traffic period, exposure of the smooth and polished surfaces of some of the larger aggregate was observed in items 2 to 4. Test items 6 and 7, which consisted of lean bituminous mixes that had asphalt contents of 5.2 and 4.3 percent, respectively, made with gravelly-clayey sand, performed unsatisfactorily because of raveling. The rate of raveling in these items increased considerably during wet-weather traffic. Test items 5 and 13, constructed with this same aggregate at asphalt contents of 5.5 and 6.7 percent, respectively, performed much better and had less raveling and rutting. These items performed satisfactorily for the entire traffic period. Substantial rutting [1.9-4.8 cm (¾-1⅞ in)] in depth developed in items 8-12 during hot-weather traffic. However, cross-section measurements show that the rutting was more pronounced in the outside wheel paths of these items than it was in the inside wheel paths. The outside wheel path was near the edge of the pavement, and the rutting was primarily caused by internal shoving, which could be eliminated by lateral containment of the mix (paving the shoulders). Although the aggregate gradation of the mixes placed in items 8 and 9 was the same and the aggregate gradation of the mixes placed in items 11 and 12 was the same, rutting was more severe in items 8 and 11 during traffic than it was in items 9 and 12. It is believed that the larger ruts that occurred in items 8 and 11 were partially caused by the higher asphalt content of the mixes placed in these items. The asphalt content of the mix placed in item 8 was 6.2 percent as compared to 5.6 percent for the mix placed in item 9, and the asphalt contents of the mixes placed in items 11 and 12 were 7.4 and 5.9 percent, respectively. A slight amount of the rutting that occurred in items 2-13 is also attributed to consolidation of the mix under traffic because in Table 4 the densities of the bituminous mixes within the wheel paths after traffic were greater than the densities of the respective mixes either prior to traffic or after traffic and between the wheel paths.

All 13 test items withstood the straight-pass-type traffic applied with the M113 and M48A1 tracked vehicles without any noticeable distress.

CONCLUSIONS

Based on the results of laboratory and field tests performed on the marginal material mixes, the following conclusions are believed warranted:

1. The concept of using marginal materials in the making of hot bituminous AC mix and zero-slump PCC is applicable for pavements that are to be used as secondary roads, streets, parking lots, storage areas, or for relatively short-service-life pavements.
2. AC can be made from a wide range [almost any material with 100 percent passing the 3.8-cm (1.5-in) sieve to about 15 percent passing the 75-μm (No. 200) sieve] of coarse-grained soils.
3. The Marshall design procedure can be used for designing all marginal material hot bituminous mixes, except those made with sands that contain little or no fines. The surface area method should be used to design hot bituminous mixes made with sands.
4. If laboratory test facilities are not available, the asphalt content of hot bituminous mixes made with marginal-aggregate materials should range between about 5.5 and 6.5 percent.
5. The stability of a bituminous mix made from a marginal material should be 136 kg (300 lb) or greater, and the retained stability of this mix should be at least 50 percent.
6. Bituminous mixes made of highly plastic aggregate materials can be expected to ravel, especially during wet-weather traffic.
7. Highly plastic aggregate materials that are to be used in a hot bituminous mix or a zero-slump concrete mix should be thoroughly processed prior to incorporating asphalt cement or portland cement to ensure a uniform mixture.
8. The sand-aggregate ratio of a marginal material to be used in a zero-slump concrete mixture should be 25 or more.
9. Satisfactory placement of zero-slump concrete can be accomplished with a conventional base course spreader, asphalt finisher, or motor patrol.
10. Zero-slump concrete can be adequately compacted with heavy vibratory rollers.

ACKNOWLEDGMENT

This paper was prepared from data obtained in an investigation sponsored by the Office of the Chief of Engineers, U.S. Army, Washington, D.C., entitled Utilization of Marginal Construction Materials for LOC. The responsibility for the conduct of the investigation was assigned to the Geotechnical Laboratory of the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Econocrete Pavements: Current Practices

W.A. Yrjanson and R.G. Packard

This report represents a compilation of recent experience in the United States with the construction and design of econocrete pavements (lean concrete that may be made with low-cost, locally available aggregates that do not meet conventional specifications). Interest in the use of econocrete has developed in the last few years due to the high cost and dwindling supplies of high-quality aggregates in some areas of the country. Described are the different uses of econocrete as subbases under concrete pavements, base courses under asphalt surfaces, composite concrete pavements, and shoulders. The report also discusses aggregate requirements and mix design for econocrete, laboratory investigations, and field research and gives current practices and recommendations for construction.

Greater use of local aggregates, substandard aggregates, and recycled pavement materials can bring about considerable economy in pavement construction. In many areas, the supply of high-quality aggregates is becoming depleted, which has caused greatly increased material costs and hauling costs.

To reduce pavement costs and preserve high-quality aggregates, the Federal Highway Administration (FHWA) in 1975 issued a notice (1) to encourage the use of econocrete in pavement subbase, base, composite pavements, and shoulder designs. Econocrete is a name that has been given to a portland cement concrete that may be made with a relatively low cement content and with low-cost aggregates or recycled materials that do not necessarily meet standards for normal concrete aggregates. In areas where high-quality aggregates are in short supply, substantial quantities of substandard local aggregates are often available. These can be used as an aggregate in econocrete base or subbase courses when a proper mix design is used to provide appropriate levels of strength and durability for these applications.

USES OF LEAN CONCRETE OR ECONOCRETE PAVEMENTS

In the last few years, in the United States, the use of lean concrete or econocrete pavements has increased. Since 1975, econocrete has been constructed on more than 50 highway and airport paving projects in 20 states. These projects include econocrete used as the following:

1. Subbase course under concrete pavement,
2. Composite concrete pavement,
3. Base course under asphalt surface, and
4. Shoulders adjacent to concrete pavement.

This paper presents recent developments in the uses of econocrete, aggregate requirements and mix proportioning, laboratory investigations and field trials, and construction methods.

Subbases

The greatest use of econocrete has been as a subbase under a conventional concrete pavement. This is a nonmonolithic construction, where the surface course of normal concrete is later placed on a hardened econocrete subbase.

Data for several projects are given in Table 1. Recycled pavement materials were used as aggregates in the econocrete for a number of these projects.

Figure 1 shows typical cross sections for highway projects in Georgia and Colorado. The subbase is built wider than the pavement or extended beneath the shoulders, which provides beneficial support for the pavement edge. Generally, this design has been used for pavements that will be subjected to high volumes and weights of traffic. One of the reasons for selecting an econocrete subbase (2) is to provide an erosion-resistant subbase surface. that should help inhibit joint faulting of undoweled joints.

Composite Concrete Pavements

In a composite concrete pavement, the surface course of full-strength concrete is cast monolithically with the lower-course econocrete, which results in full bond between layers. The bond is achieved by coarse-texturing or scarifying the surface of the lower course while the econocrete is still in the plastic state and then immediately placing the surface-course concrete. The monolithic action of a composite pavement results in an efficient structural design section.

Figure 2 shows typical cross sections of composite pavement projects constructed in Iowa and North Dakota. On these projects, the monolithic top course wraps around the edges of the base by about 38 mm (1.5 in) on either side. Data for several projects are given in Table 2. The Iowa projects used recycled materials as aggregates.

Composite concrete pavements have a great potential for economy, especially in areas where high-quality aggregates are in short supply. The practice has been to place a relatively thin monolithic surface course on a thicker econocrete lower course. As a result, a greater proportion of the pavement section uses less-expensive materials.

Base Courses Under Asphalt Surface

Concrete bases that have asphalt surfaces have been used for years by several cities for major arterial streets and by some state highway departments for ramps on Interstate highways. Usually on these projects, the aggregates have met normal concrete aggregate specifications and the cement contents are somewhat less than for normal concrete but not generally as low as for econocrete.

In many foreign countries, lean concrete bases have been used extensively for highways, streets, and airport pavements. This experience has been with both dry (compacted with rollers) and wet (compacted with internal vibration) lean concrete. The specifications vary somewhat from country to country—some permit lower-quality aggregates; most of them use low cement contents that give 28-day compressive strengths in the range of 6.9-13.8 MPa (1000-2000 lbf/in^2). The practice has evolved to use low-cement-content, low-strength mixtures so that the seriousness of reflection cracks in the asphalt surface is minimized.

In the United States in the last few years several projects have been constructed by using econocrete base courses. Table 3 is a partial list of these projects and Figure 3 shows some typical cross sections.

Shoulders

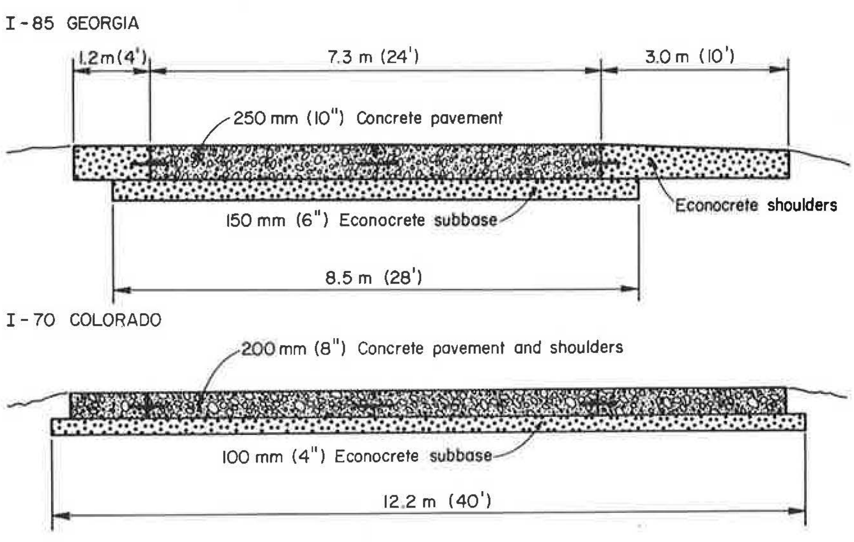
Econocrete shoulders may be constructed with new concrete pavements or rebuilt shoulders adjacent to

Table 1. Partial list of econocrete subbases under concrete pavements.

Location	State	Length (km)	Width (m)	Cement Content[a] (kg/m³)	Subbase Thickness (mm)	Surface Thickness (mm)	Year
Routes							
AZ-360, Superstition Freeway, Tempe	AZ	1.9	9.9	153	102	152	1977
CA-91, Artesia Freeway, Compton[b]	CA	4.8	15.2	139	122	203-229	1975
I-5, north from Stockton	CA	11.1	11.6	-[c]	127	203-229	
CA-1, Monterey	CA	3.4	11.6	-[d]	107	203-229	1972
CA-1, Tracy bypass near Ft. Ord, Mojave	CA						1970
I-70, Rifle	CO	16.9	12.2	148	102	203	1976
I-70, Arvada	CO	2.7	11, 4	133, 118[e]	152	203	1979
I-75, Sarasota area	FL	53.0	12.2	208	152	229	1979
I-85, LaGrange	GA	17.5	8.5	148	152	254	1976
Entrance ramps, Atlanta airport	GA			160	127		1979
I-72, Monticello	IL		7.9	148	102	203	1976
IL-78, Kewanee	IL	8.0	7.9	148	102	203	1975
Freeway Rt. 534, Burlington	IA	4.0	7.9	178	102	229	1976
US-30, Cedar Rapids	IA	9.0	7.9	178	102	229	1976
I-129, Sioux City	IA	0.8	7.6	178	102	216	1977
IA-520, Sioux City	IA	4.5	7.6	178	102	229	1978
I-680, Missouri River east to I-29	IA	5.0	7.6	178	102		1977
I-380, Cedar Rapids	IA	0.6	7.9	178	102	254	1978
I-580, Reno	NV	1.0					1979
I-80, Wendover	NV	8.0	12.8	175-219	102	203	1978
US-52, south of Winston-Salem	NC	8.7	8.5	119 minimum	102	229	1979
I-680, north of Ohio Turnpike, Youngstown	OH	4.0	12.8	-[f]	102	203	1971
test track		12.1	11.9	-[g]	102	254	
Route 220, Williamsport	PA	1.9	8.5	178	152	229	1979
I-77, Edgemoor	SC	20.0	8.5	139	152	229	1979
I-95, Dorchester Co.	SC	1.6	8.2	126	152	267	1975
I-77, near Blythewood	SC	15.1	8.2	126	152	229	1978
I-77, Columbia	SC	2.3	8.5	126	152	229	bid 1979
I-77, Richburg	SC	15.1	8.5	126	152	229	bid 1979
I-77, ramps, Fort Mill	SC	0.6	6.7	95, 133	152	229	1974
I-24, north of Nashville	TN	6.6	8.2	148	127	254	let 1977
I-80, west of Salt Lake City	UT	4.1	15.2	162	102	279	1979
I-15, Beaver	UT	24.1	12.8		102	241	1979
Airports							
Jacksonville International, runway keel[b]	FL		15.2	148	152	356	1975
Jacksonville NAS, apron	FL				152	254	
Tampa, runway, taxiway, and extension	FL		48, 25	107-130	152	406	1979
Standiford Field, Louisville, taxiway	KY		23	-[h]	305	356	1978
Shreveport, taxiway	LA		24.1	162	152	356	1975
Tupelo	MS			181	152	203	1978
Pittsburgh International, runway and taxiways	PA		30, 15	142	152	279-406	1979
Pittsburgh International, taxiway extension	PA		23	142	152	356	let 1979
Tocumen Airport, runway, taxiway, and apron	Panama			-[i]	229-279		1976

Note: 1 km = 0.62 mile; 1 m = 3.28 ft; 1 kg/m³ = 1.69 lbf/yd³; 1 mm = 0.039 in.
[a] Cement content of subbase. [b] Recycled aggregates. [c] 10 percent. [d] 8.5 percent. [e] Plus 59 kg/m³ fly ash. [f] 12 percent. [g] 7 percent. [h] 4 percent. [i] 6 percent.

Figure 1. Typical cross sections for econocrete subbases.



existing concrete pavements. The requirements for the shoulder concrete are not as demanding as for the main roadway pavement. Normally, shoulders carry very little traffic, and the abrasion resistance, strength requirements, and aggregate quality requirements are lower. However, the econocrete mix design should take into account the freeze-thaw durability requirements, depending on climatic considerations.

Econocrete shoulders should improve main-line pavement performance by providing a tight, sealed joint between pavement and shoulder and by reducing load deflections at the pavement edge.

Data on econocrete shoulder projects constructed recently are given in Table 4. On most of these projects, the shoulder-pavement joint is tied with deformed tie bars. Frequently, rumble strips are formed in the plastic concrete to discourage travel on the shoulder.

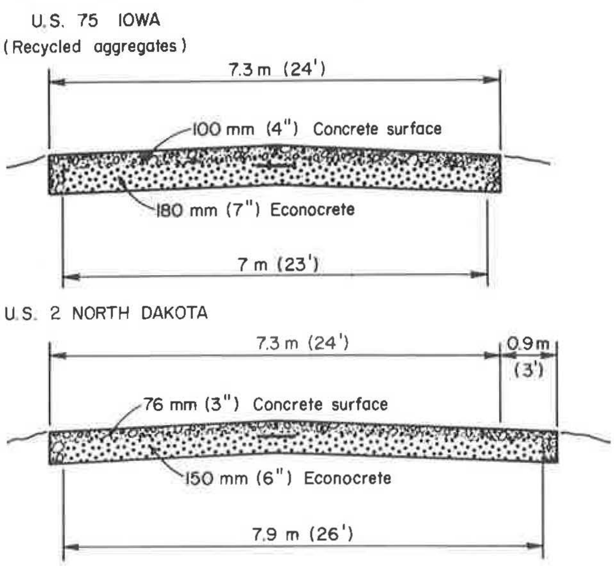
FLORIDA ECONOCRETE TEST ROAD

Important research is currently being conducted by the Florida Department of Transportation on the Florida Econocrete Test Road. This 10.6-km (6.6-mile) test road on US-41 north of Ft. Myers was opened to traffic in April 1977. Test sections of econocrete were built at three levels of cement contents [130, 160, and 202 kg/m^3 (220, 270, and 340 lb/yd^3)] by using an aggregate that was a limerock material from an excavation.

Some test sections had specially reinforced concrete surfaces, but of particular interest on the test road is the performance of the 14 sections of econocrete that had plain concrete or asphalt surfaces. In these, the lower courses of econocrete are 230 mm (9 in) thick with surface courses of 75 mm (3 in). The econocrete was constructed without joints. For the sections surfaced with concrete, the surface was bonded to the econocrete subbase, and joints in the surface concrete were spaced at 4.6 m (15 ft) in some sections and 6.1 m (20 ft) in others; right-angled and skewed joints were placed in different sections.

After 2.5 years of heavy traffic, accumulating more 80-kN (18 000-lbf) equivalent axle loads than were applied at the American Association of State Highway Officials (AASHO) Road Test, the 14 econocrete sections that had plain concrete or asphalt surfaces are in excellent condition. Continued observation of these sections by periodic inspections and measurements of rideability and other factors should provide valuable information on the performance of econocrete composite pavements.

Figure 2. Typical cross sections for composite concrete pavements.



AGGREGATE REQUIREMENTS AND MIX DESIGN

Some of the restrictive specification requirements for concrete aggregates relate to the performance characteristics of the exposed pavement surface—where substandard aggregates may cause undesirable surface conditions, such as lack of abrasion resistance, slippery pavements, or pop-outs. Many substandard or marginal aggregates that do not meet normal specifications may be acceptable when used in econocrete as a lower course in the pavement structure.

Aggregate gradation requirements for econocete are also not as strict as those for normal concrete. In many cases the regular gradation of aggregate from the crushing plant, or crusher-run as it is sometimes called, is satisfactory without the addition of sand. Aggregates that meet gradation specifications for untreated base course have also been used as is. Conventional concrete aggregates have also been used with modified mix designs.

Some specifications designate only the top size of aggregate and the amount that passes a 75-μm (No. 200) sieve. It is noted that gradation specifications should be modified to meet local aggregate gradations if suitable econocrete mixtures can be produced; then

Table 2. Partial list of composite concrete pavements.

Location	State	Length (km)	Width (m)	Cement Content[a] (kg/m^3)	Lower-Course Thickness (mm)	Surface-Course Thickness (mm)	Year
US-41, test road, Ft. Myers	FL	1.3	7.3	130	229	76	1976
US-41, test road, Ft. Myers	FL	1.8	7.3	160	229	76	1976
US-41, test road, Ft. Myers	FL	1.3	7.3	202	229	76	1976
Intersection in St. Joseph County	IN			279	152	51	1978
US-75, Rock Rapids[b]	IA	0.8	7.3	279	178	102	1976
US-75, Sioux City[b]	IA	1.6	7.3	335	← 229 full depth →		1976
IA-2, between Bedford and Clarinda[b]	IA	27.9	7.3	371	← 229 full depth →		1978
I-96, temporary, Detroit	MI			178	152	178	1976
US-31, Shelby Road, interchange ramp	MI			178	102	102	1976
US-2, Rugby to Leeds	ND	41.4	8.2	190	152	76	1977

Note: 1 km = 0.62 mile; 1 m = 3.28 ft; 1 kg/m^3 = 1.69 lbf/yd^3; 1 mm = 0.039 in.
[a] Cement content of lower course.
[b] Recycled aggregates.

Table 3. Partial list of econocrete base courses under asphalt surface.

Location	State	Length (km)	Width (m)	Cement Content (kg/m³)	Base Thickness (mm)	Surface Thickness (mm)	Year
CA-7, Long Beach Freeway	CA	10.3		136	320	137	1977
I-5, Santa Ana Freeway	CA	4.0		136	183	122	1977
CA-198, Coalinga	CA	9.5		160	244	61	1979
CA-198, Coalinga	CA	9.7	8.5	160	244	61	1978
Frontage Road, Santa Monica	CA	0.3		196	183	122	1979
Fifth Street and ramp, Santa Monica	CA	0.6		196	259	122	1979
US-41, test road, Ft. Myers	FL	0.6	7.3	130	229	76	1976
US-41, test road, Ft. Myers	FL	0.6	7.3	160	229	76	1976
Harrison Avenue, Rockford	IL	3.9	18.3	252	229	76	1978
US-83, Cole Harbor	ND	10.1	11.3	148[a]	152	38	1977
US-2 and 52 bypass, Minot	ND	4.0	11.3	148[a]	152	64	1978
US-2, west of Grand Forks	ND	20.6		148[b]	152	64	let 1979
Local road, Lock Haven	PA	0.2		112	152	64	1977

Note: 1 km = 0.62 mile; 1 m = 3.28 ft; 1 kg/m³ = 1.69 lbf/yd³; 1 mm = 0.039 in.
[a]Plus 59 kg/m³ fly ash.
[b]Plus 44 kg/m³ fly ash.

Figure 3. Typical cross sections for econocrete base courses.

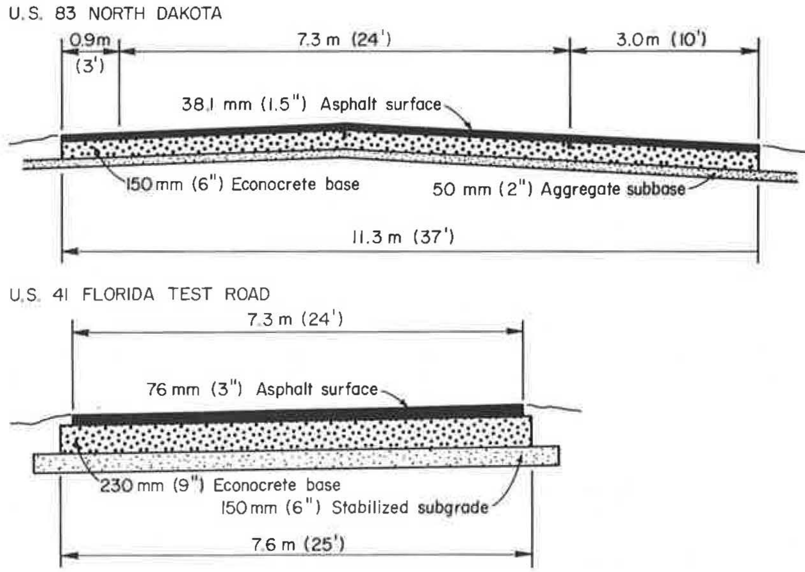


Table 4. Partial list of econocrete shoulders.

Location	State	Length (km)	Width (m)	Cement Content (kg/m³)	Thickness (mm)	Year
US-41, test road, Ft. Myers	FL	0.6	1.2, 2.4	202	152	1976
I-75, Sarasota area	FL		3.0		229-152	1979
I-85, LaGrange	GA	17.5	1.2, 3.0	252	152	1976
I-16, 56 km west of Savannah	GA	19.3	1.2, 3.0	252	254-152	1976
MI-14, west of Plymouth	MI	2.4		178, 208, 237	203-152	
US-52, Davidson-Forsythe Counties	NC	8.7	1.2, 3.0	267	229-152	1979
US-83, Coleharbor	ND	10.1	0.9, 3.0	148[a]	152[b]	1977

Note: 1 km = 0.62 mile; 1 m = 3.28 ft; 1 kg/m³ = 1.69 lbf/yd³; 1 mm = 0.039 in.
[a]Plus 59 kg/m³ fly ash.
[b]Surfaced with 38 mm of asphalt.

a gradation specification for the project can be written to control the variability of the aggregate.

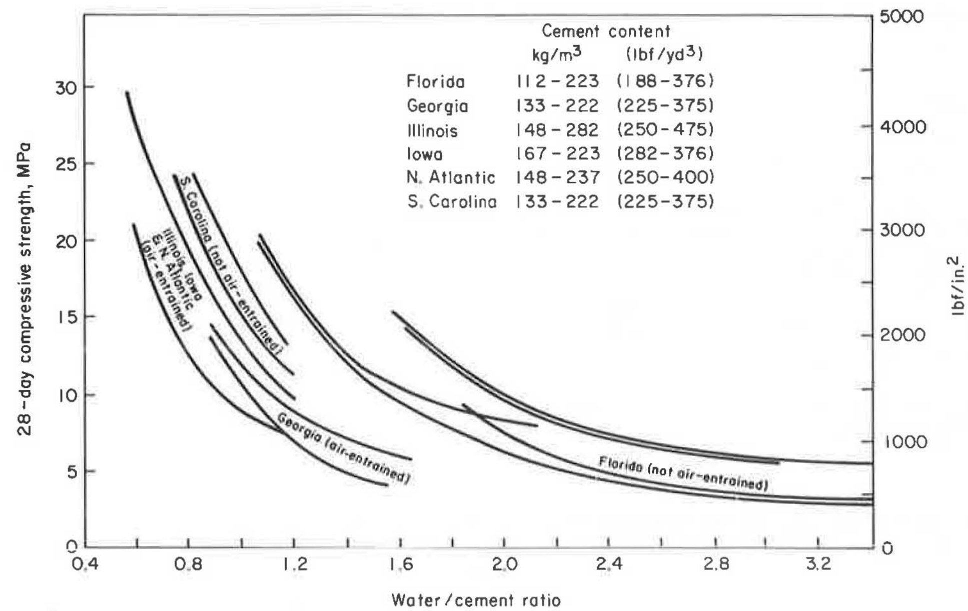
Data obtained from recent laboratory test programs and econocrete construction projects (3) indicate that a wide range of aggregates may be used. Some of these aggregates are materials not processed to the degree that normal aggregates are. Most have more fine material that passes the 150-μm and 75-μm (No. 100 and No. 200) sieves than is acceptable for normal concrete, but this is not necessarily objectionable for econocrete because the extra fines help supply needed workability for mixes that have low cement contents.

On several recent recycling projects, old concrete and asphalt pavements have been crushed and used as aggregates for econocrete.

Mix Design

For the proportioning of econocrete mixtures, the normal procedures and tests for concrete are followed with these exceptions: (a) a single aggregate is sometimes used rather than a combination of coarse aggregate and fine aggregate and (b) the cement content is usually less than that for normal concrete. A primary requirement is that the econocrete be workable—easy to mix and place, capable of adequate consolidation by

Figure 4. Compressive strength versus water-cement ratio.



vibration, and cohesive enough to resist excessive edge slumping when placed with a slip-form paver. The second requirement is that the hardened concrete have the level of strength and durability appropriate for the exposure conditions.

Workability of concrete depends primarily on the aggregate characteristics, air content, and the cement content. Since the cement content is low in lean concrete (which could cause poor workability for normal aggregates), the workability may be enhanced by (a) the existence of extra fines in the aggregate; (b) higher than normal amounts of entrained air; (c) addition of fly ash, water-reducing admixtures, or workability agents; or (d) a combination of these.

In the laboratory, trial mixes with selected cement contents are used to determine a mix design that will give the desired workability and slump [usually in the range of 25-75 mm (1-3 in)] for the aggregate or combination of aggregates. Typical gradation specifications for econocrete are given below (1 mm = 0.039 in).

Sieve Designation	Percentage Passing Sieve A	B	C
50 mm	100		
38.1 mm		100	
25.0 mm	55-85	70-95	100
19.0 mm	50-80	55-85	70-100
4.75 mm	30-60	30-60	35-65
425 µm	10-30	10-30	15-30
75 µm	0-15	0-15	0-15

Properties of the hardened lean concrete are then determined by strength tests and, if appropriate, durability tests. Strength requirements have not been definitely established but it is generally considered that the strength requirement will vary with the structural use and that high strengths are not required when the econocrete is not used as an exposed surface. Limited data on freeze-thaw durability of lean concrete indicate that high air contents may be required to achieve a high degree of resistance to concrete freeze-thaw tests. It has not been established whether the durability requirements need to be as stringent as those for a concrete surface course. It is expected that test requirements for econocrete used as a lower course of a pavement will be similar to those required for cement-treated bases. Additional research and performance experience are needed to better define durability requirements and appropriate laboratory tests. It appears that air contents on the high side of the range recommended for normal paving concrete may be needed for econocrete constructed in freeze-thaw areas. Freeze-thaw resistance requirements for econocrete used as an exposed surface (such as pavement shoulders) should be the same as for normal concrete.

Laboratory Investigations

Results of several recent laboratory test programs are described here to provide some preliminary guidelines for mix design. Results of the studies are described in more detail elsewhere (3).

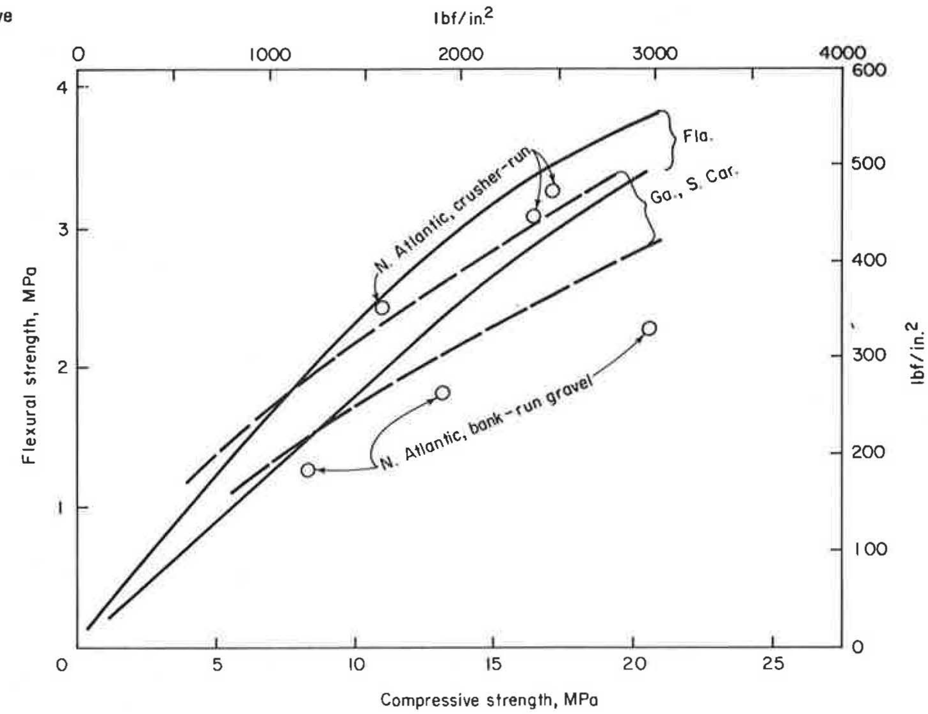
The Georgia Department of Transportation laboratories tested crushed stones from four sources (4). In the initial tests, a mix that contained 140 kg/m^3 (235 lb/yd^3) of cement without air-entraining or water-reducing admixture was found to be harsh and unworkable and exhibited heavy bleeding. As a result, all subsequent trial mixtures contained an air-entraining and a water-reducing admixture. These mixtures were cohesive and workable and appeared to be suitable for placing with conventional paving equipment.

The second phase of the Georgia investigation involved mixes that had cement factors of 133, 178, and 222 kg/m^3 (225, 300, and 375 lb/yd^3). Air contents for these mixes were kept at about 6.5 percent and slumps were in the range of 25-50 mm (1-2 in).

Compressive strengths for these mixes, as well as for the other studies discussed later, are shown in Figure 4, where the general relation of strength to water-cement ratio and air content is apparent.

The South Carolina Highway Department conducted tests initially on crushed stones from two sources with about 178 kg/m^3 (300 lb/yd^3) of cement and with air-entraining and water-reducing admixtures. Following these tests, additional aggregates (a crushed limestone and a granite-gneiss crushed rock) were tested with cement contents in the range of 133-222 kg/m^3 (225-375 lb/yd^3), air contents of about 3 percent, and slumps of about 12 mm (0.5 in). A water-reducing admixture was used but not an air-entraining admixture. These mixes had considerably higher strengths (see Figure

Figure 5. Flexural strength versus compressive strength.



4) than those in the first series of tests, which had higher air contents and slumps.

The Florida Department of Transportation conducted a test program on econocretes (5) made from four common sources of Florida base-course aggregates: Ocala limerock, a low calcium oolite, coquina, and a stabilizing grade of limerock. The aggregates were taken from the quarry without processing with the plus 50-mm (2-in) material removed. Mix designs with cement contents of 112, 167, and 223 kg/m³ (188, 282, and 376 lb/yd³) were made with no admixtures. Slumps ranged from 0 to 12 mm (0.5 in); air contents were 1-2.5 percent. These high-calcium aggregates generally contained more fines and had lower specific gravities and higher absorptions than normal. They required relatively high water contents to develop plastic mixes, which contributes to the high water-cement ratios shown in Figure 4.

Lower strengths were obtained than those determined in other studies on more conventional aggregates at equal cement contents. However, when compared with the pattern of strength versus water-cement ratio for other materials shown in Figure 4, the strengths are higher than normal—possibly due to some beneficial characteristics of the aggregates in their reaction with cement.

Laboratory studies and field trials were conducted in Illinois to determine mix design requirements of econocrete used as a subbase and as a shoulder. Aggregates included mixes of gravel with a natural sand and various base course aggregates. Cement contents were varied from 148 to 282 kg/m³ (250 to 475 lb/yd³), and air-entraining and water-reducing admixtures were used. Air contents were generally in the range of 6.5-8 percent and slumps were 19-38 mm (0.75-1.5 in). Twenty-eight-day compressive strengths between 11 and 32 MPa (1600 and 4700 lbf/in²) were obtained on these mixes.

The Iowa State Highway Commission conducted an extensive study of 27 different aggregate sources. Mix designs were made with three cement contents: 167, 195, and 223 kg/m³ (282, 329, and 376 lb/yd³). Air content for most of the mixes was in the range of 5.0-7.5 percent; slumps were generally between 20 and 50 mm (0.75 and 2 in). A water-reducing admixture was used in almost all mixes. The strengths for the conditions stated are plotted in Figure 4 and varied from 8 MPa (1200 lbf/in²) at a water-cement ratio of 1.27 to 27 MPa (3900 lbf/in²) at a ratio of 0.59.

A laboratory study (6) was conducted on two aggregates from North Atlantic states—a siliceous limestone and a bank-run gravel, both having excessive amounts of material passing a 75-μm (No. 200) sieve. Cement factors of 148, 193, and 237 kg/m³ (250, 325, and 400 lb/yd³) were used. The mixes were designed to contain from 6 to 8 percent air and had about a 25-mm (1-in) slump.

Substantial quantities of air-entraining admixture were required to generate the air void system, and the dosage increased as the cement content decreased.

The 28-day compressive strengths for these mixes fell in the same band in Figure 4 as the other air-entrained mixes of the Iowa and Illinois studies. These strengths ranged from 8.3 MPa (1200 lbf/in²) at a water-cement ratio of 1.08 to 21 MPa (3000 lbf/in²) at a water-cement ratio of 0.65.

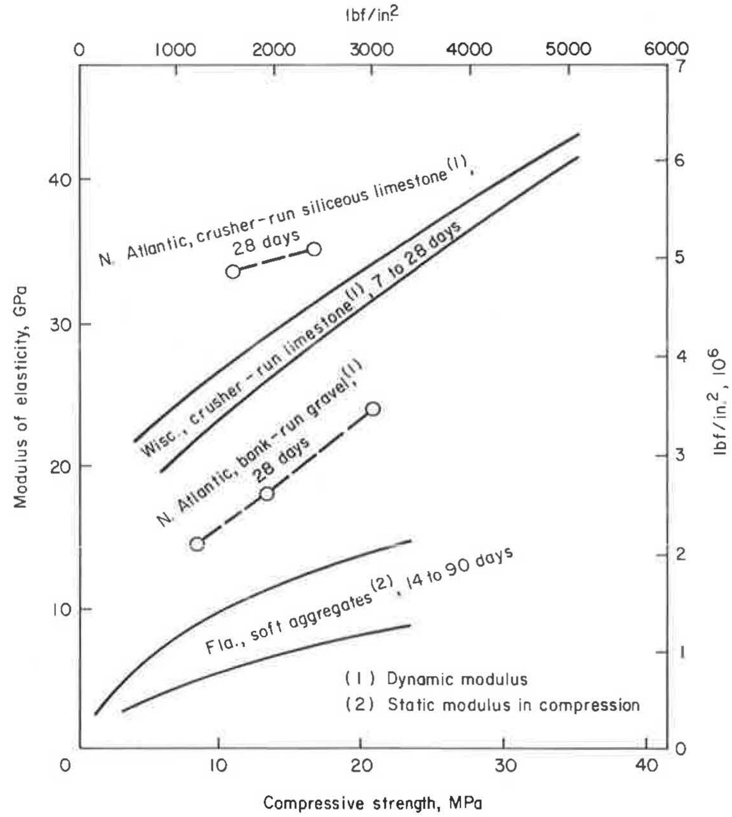
Three hundred cycles of concrete freeze-thaw tests (ASTM C666, procedure B) were also run on these mixes, and it was found that at least 7 percent air was required to provide resistance to the test conditions.

In some of these laboratory studies, flexural strengths and moduli of elasticity were determined as well as compressive strengths. Figures 5 and 6 illustrate the data obtained.

Summary of Material Requirements and Mix Design

Laboratory investigations and field installations indicate that the desirable properties of econocrete to be used as a base or subbase course are achieved with cement factors in the range of 120-210 kg/m³ (200-350 lb/yd³), giving 28-day compressive strengths between 5.2 and 10.4 MPa (750 and 1500 lbf/in²). For the lower course in a composite concrete pavement, cement factors and

Figure 6. Modulus of elasticity versus compressive strength.



strengths may be at these levels or higher—up to those for normal concrete. Other recommendations are to attain slumps in the range of 25-75 mm (1-3 in), and air contents equal to that recommended for normal concrete and somewhat greater [6-8 percent air for concrete made with 25-50 mm (1-2 in) maximum size aggregate] for freeze-thaw areas. Additional research is needed to better define appropriate tests and criteria for freeze-thaw resistance.

CONSTRUCTION OF ECONOCRETE

Econocrete components of a pavement structure are constructed in essentially the same manner and with the same equipment as normal concrete pavements. The only differences, depending on the application, may be (a) the jointing practice and (b) the treatment of the interface between the base or subbase and surface courses. The following recommendations are made based on current experience.

Joints

For subbases and base courses, joints in the econocrete are not considered necessary. Hairline cracks will develop in the econocrete; but experience has shown, for the low strength levels recommended and with the interlayer treatment discussed below, reflection cracking will usually not occur in concrete surfaces and will not be serious in asphalt surfaces.

For composite concrete pavements, the jointing practices should be the same for normal concrete pavements.

For shoulders, joints are placed to match the joint pattern in the main-line pavement or, for continuously reinforced main-line pavements, joints in econocrete are placed at short intervals—4-5 m (15-20 ft).

Interlayer Treatments

For subbases, the current practice is to leave the econocrete untextured to prevent mechanical bond with the concrete surface course and to apply an ample coat of wax-based concrete curing compound as a bond breaker. If the curing compound becomes worn off due to traffic or other causes, another coat should be applied before the surfacing concrete is placed.

For base under asphalt surface, the current practice is to tine or coarsely scarify the econocrete surface to promote mechanical bond and cure with asphalt emulsion or resin-based concrete curing compound.

For composite concrete pavements, the econocrete surface is coarsely tined. No curing compound is applied since the concrete surface course is placed immediately on top.

The American Association of State Highway and Transportation Officials has recently adopted a guide specification for econocrete (7) that discusses construction items in detail.

SUMMARY

In many areas of the United States, the supply of high-quality aggregate for pavement construction is becoming depleted. Materials from existing sources are becoming expensive or unavailable due to restrictive zoning, environmental controls, and appreciated land values. Due to these problems, a serious interest in the use of econocrete (a lean concrete made with local, low-cost aggregates not necessarily meeting conventional specifications) began in about 1975. In this paper, an attempt has been made to present a summary report on a number of paving projects that used econocrete for base and subbase courses, composite concrete pavements, and shoulders; to discuss laboratory investigations and field research; and to report current practices and recommendations for aggregate requirements, mix design, and construction methods for econocrete pavements.

REFERENCES

1. Pavement, Subbase, and Shoulder Designs Using Econocrete and the Initiation of NEEP Project No. 20—Experimental Pavement Construction Using Econocrete. Federal Highway Administration, Notice N5080.34, April 23, 1975.
2. California Trials with Lean Concrete Base. California Department of Transportation, Sacramento, Interim Rept., CA-DOT-TL-5167-3-75-37, Oct. 1975.
3. Econocrete, Report No. 1. American Concrete Paving Association, Arlington Heights, IL, Aug. 1975.
4. R.W. Allen. Lean Concrete Mix Design Studies for Use as Concrete Pavement Base Course. Office of Materials and Tests, Georgia Department of Transportation, Forest Park, 1973.
5. T.J. Larsen, H.W. Harling, and F.E. Howard. Strength Properties of Lean Concrete. Florida Department of Transportation, Gainesville, Research Rept. 80, Dec. 1973.
6. M.H. Wills, Jr. The Potential for Econocrete in the North Atlantic States. Materials and Mix Design Committee, American Concrete Paving Association, Arlington Heights, IL, Nov. 12, 1974.
7. Section 310—Econocrete, or Lean Mix Portland Cement Concrete Base Course. In Guide Specifications for Highway Construction, American Association of State Highway and Transportation Officials, Washington, DC, 1979.

Construction and Performance of Sand-Asphalt Bases

Richard D. Barksdale

Sand-asphalt base construction practices and field performance are described based on extensive field inspections and interviews with state transportation personnel in Florida, Georgia, Maryland, and South Carolina. The results are also summarized of laboratory fatigue and rutting tests performed on both sand-asphalt and sand-stone asphalt mixes. Laboratory studies indicate that the fatigue characteristics of a sand-asphalt mix can be generally controlled by (a) limiting the void content to 12-15 percent, (b) using asphalt contents greater than 5.5-6.5 percent, and (c) designing the mix with a Marshall stability as high as practical. Important variables that affect rutting in a sand-asphalt mix are asphalt content, Marshall stability (or air void content that appears to be related to Marshall stability), and the characteristics for the aggregate. The specific effects of these variables are presented for selected mixes. Sand-asphalt and sand-stone blend asphalt mixes can be successfully used as bases on primary and Interstate highways. Rutting in pavements constructed by using 150- to 200-mm (6- to 8-in) sand-asphalt base is typically between 8 and 15 mm (0.3 and 0.6 in). An allowable rut depth for design purposes of 10 mm (0.4 in) is recommended for primary and Interstate pavements. The 50-blow Marshall mix design method can be used for sand-asphalt bases, provided rutting and fatigue resistance of the mix is taken into account. The blending of up to 75 percent crushed aggregate with sand offers an excellent way to decrease rutting and increase fatigue life of the mix while still using local sand.

Due to rising energy costs, construction of pavements by use of local materials, often of low quality, has become a necessity. Pavements constructed by using sand-asphalt mixes, if not properly designed, may undergo excessive rutting or premature fatigue distress. The purpose of this paper is to investigate the use of sand-asphalt in base-course construction. The findings presented are the result of field inspections and interviews with personnel of four selected state transportation organizations and a comprehensive laboratory investigation of fatigue and rutting characteristics of sand-asphalt mixes.

SELECTED CONSTRUCTION PRACTICES AND FIELD PERFORMANCE

Sand-asphalt mixes are used in the southeastern portion of the United States, primarily in the coastal plain areas. The construction practices and field performance of sand-asphalt bases in Florida, Georgia, Maryland, and South Carolina are summarized in this section. Other southern coastal plain states also use sand-asphalt bases.

Florida

The Florida Department of Transportation has used sand-asphalt bases extensively throughout Florida and has used, to a much lesser extent, sand-stone-asphalt blends. Pavements in Florida that have sand-asphalt bases were found to show good performance and surface rutting usually less than 12 mm (0.5 in). A cross slope of 2 percent is used in Florida and no problems of ponding water were reported. The surface cracking that develops is typically longitudinal. Because of the favorable climate and good subgrade conditions that occur throughout most of the state [usually a California bearing ratio (CBR) of 15-25], relatively light structural sections are used in Florida. For pavements subjected to high volumes of traffic, a 75- to 130-mm (3- to 5-in) thick asphalt-concrete (AC) surfacing mix is placed over approximately 250 mm (10 in) of sand-asphalt base. A 300-mm (12-in) prepared subgrade is used below the base. For low-volume roads, a 40-mm (1.5-in) thick sand-asphalt surfacing is placed over 150-200 mm (6-8 in) of unstabilized limerock base.

In metropolitan areas that have concentrated traffic that require relatively high stability, a sand-asphalt base that has a stability of 3.3 kN (750 lbf) is sometimes specified. Usually, however, a sand-stone blend AC mix is used to meet higher stability requirements. This

Figure 1. Variation of pavement performance with sand-asphalt base thickness and base stability—Marianna test road.

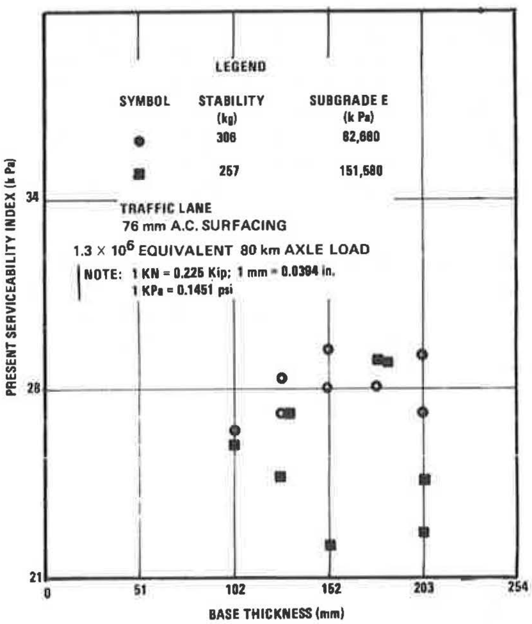
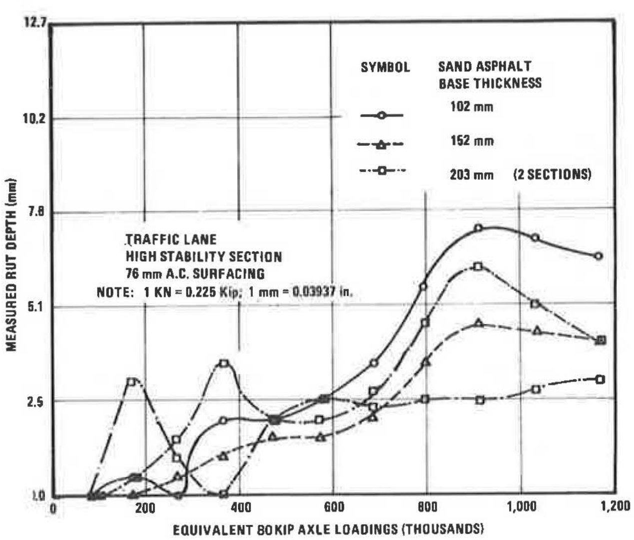


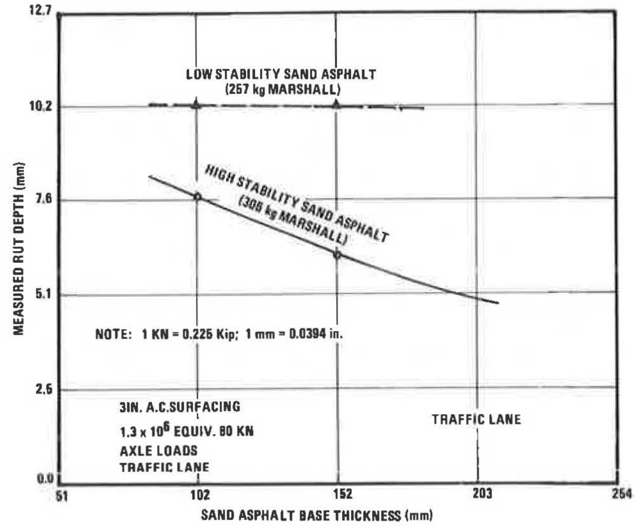
Figure 2. Effect of traffic loading and sand-asphalt base stability on rut depth at Marianna test road—traffic lane.



type mix has been used on about eight jobs. Sand-stone blend AC mixes can be placed in lifts up to 130-150 mm (5-6 in) in thickness and can have up to 25 percent sand. Natural sands are frequently used in the surface (up to 15-20 percent) and in binder courses. A 2-kN (500-lbf), 50-blow Marshall stability mix is generally used for sand-asphalt bases, although in some areas, such as West Palm Beach, mixes are sometimes used that have stabilities as low as 0.45-0.9 kN (100-200 lbf).

The air voids in sand-asphalt mixes are limited to 12 percent. For sand-asphalt bases, two sands or crushed-stone screenings are frequently blended to meet stability and gradation requirements. When possible, a well-graded sand that has angular grains is used. The rounded blow sands found in the Lake Wales area are

Figure 3. Rut depth as affected by sand-asphalt base thickness, base stability, and traffic lane—Marianna test road.



typically blended with 50 percent screenings for stability. Specifications allow up to 12 percent fines, but experience shows that 6-7 percent fines are usually required to meet stability requirements. Up to 7 percent clay can now be used in the sand.

In sand-asphalt bases, Florida typically uses 6.5-7.5 percent of an AC-20 viscosity grade asphalt cement that has a viscosity at 60°C (140°F) between 160 and 240 Pa·s (1600 and 2400 poises). Field experience indicates, however, that asphalt cements that have viscosities in the range of 200-240 Pa·s (2000-2400 poises) at 60°C have fewer problems during laying than asphalt cements that have lower viscosities. Silicone, which has been found effective in keeping the mix from tearing during laying and for use with adsorptive aggregates, is added at the rate of 1.5 parts/1 000 000 to the asphalt cement. The addition of more than 2 parts/1 000 000 silicone has been found to cause problems with the mix. The Maryland and Georgia transportation departments also add silicone to sand-asphalt mixes, although South Carolina does not.

Sand-asphalts are mixed at approximately 121°C (250°F) and placed at about 92 or 93 percent of the 50-blow Marshall maximum density. Even this density is sometimes relatively difficult to obtain in the field. Also, sand-asphalt lift thickness greater than 75 mm (3 in) has been found to result in rolling of the layer during compaction. If the sand-asphalt hangs under the screed of the paving machine, the stability of the mix is reduced by adjusting the cold gate at the plant, changing the blend, or increasing the asphalt content of the mix. Florida is considering the use of 1.5-2 percent crushed stone screenings in all mixes, which is similar to the practice followed in Louisiana.

Test Roads

The Marianna test sections inspected were in excellent condition after 1.2 million equivalent 80-kN (18-kip) axle loadings and had values of 22-29 kPa (3.2-4.2 lbf/in^2) (Figure 1), as reported elsewhere (1). Only minor longitudinal cracking was observed locally in some sections. The section used consisted of a 75-mm (3-in) thick AC surface and binder overlaying a sand-asphalt base 100-200 mm (4-8 in) in thickness. The test sections rested on an excellent prepared sand subgrade 0.6 m (2 ft)

Table 1. Paractice usually followed by the Georgia Department of Transportation for the use of surfacing and leveling or patching sand-asphalt mixes.

Vehicles per Day	Truck Traffic (%)	Allowable Mixes
0-499	<7	SA-1 for surface and leveling or patching[a]
0-499	>7	SA-2 for surface and patching[b]
500-999	<7	SA-2 for surface and leveling or 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

[a]Sand-asphalt mix 1 (SA-1) = 5.5-7.0 percent AC, 5-16 flow, 50-blow Marshall stability of 1.56 kN (350 lbf).
[b]Sand-asphalt mix 2 (SA-2) = 5.5-7.5 percent AC, 5-16 flow, 50-blow Marshall stability of 3.11 kN (700 lbf).

thick. Rut depths in the sections were typically 6-8 mm (0.25-0.3 in) at the time of the field inspection; maximum observed rut depths were 12 mm (0.5 in). The sand-asphalt base was constructed by using an excellent local sand that has a small amount of clay. The average Marshall stability of the sand-asphalt base varied from 2.5 to 3.0 kN (566 to 675 lbf).

Rutting in the Marianna test sections was found to gradually increase with the number of wheel load repetitions (Figure 2). The 3.0-kN (674-lbf) Marshall stability sections had an average rut depth of 6 mm (0.25 in) in the traffic lane compared to 11 mm (0.43 in) for the 2.5-kN (566-lbf) Marshall stability sections (Figure 3). Therefore, for conditions existing at the Marianna test road, increasing the Marshall stability from 2.5 kN to 3.0 kN resulted in a significant reduction in rut depth in the traffic lane. This large difference in observed rut depths between the low- and high-stability sections was not reflected in measured present serviceability index (PSI) values.

The Marianna test road was constructed over a very stiff subgrade that had a modulus of elasticity that was greater than the reported modulus of the sand-asphalt base (1). The presence of the stiff subgrade undoubtedly influenced the observed results and must be considered in extrapolating these results to other pavements. Rutting in the high-stability sections was inversely proportional to base thickness; however, in low-stability sections it was constant (Figure 3) or else increased with base thickness.

The performance of sand-asphalt and limerock bases has been compared at the Lake Wales test road (2,3). The sand-asphalt and limerock base sections both had a 40-mm (1.5-in) and 75-mm (3-in) AC surfacing and a 75- to 250-mm (3- to 10-in) thick base. After approximately 1.75 million equivalent 80-kN (18-kip) axle loads, the sections that had limerock bases all were in good condition, although some longitudinal cracking was observed in the thinner sections. The sand-asphalt base sections that had a 75-mm surfacing and 200-mm (8-in) surfacing were not cracked, whereas the sections that had a 40-mm surfacing were cracked. Moderate transverse cracking was observed in the sand-asphalt sections that had 100- to 150-mm (4- to 6-in) bases for both 40- and 75-mm AC surfaces.

In the Lake Wales test road, after 1.75 million 80-kN axle loadings, the limerock base sections were performing better structurally than those constructed with sand-asphalt. Perhaps one factor that partially accounted for the performance difference was the use of blow sand in the sand-asphalt. These blow sands are considered inferior to the more angular sands found in the northern part of the state. These sands were, however, blended with equal amounts of crushed-stone screenings, which resulted in mean Marshall stabilities that varied from 1.5 to 2.3 kN (340 to 528 lbf).

Georgia

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 is 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 1. Sand can be used in AC 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 5.5-7 percent in sand-asphalt mixes. In the Albany and Bainbridge areas, screenings are generally blended with the sand, and an asphalt content of 7-7.5 percent is usually required. Type 1 sand-asphalt (SA-1) requires a minimum 50-blow Marshall stability of 1.55 kN (350 lbf), and type 2 sand-asphalt (SA-2) requires a minimum stability of 3.1 kN (700 lbf). Both sand-asphalt mixes require a maximum air voids content of 15 percent, a flow of 5-15, a 24-h immersion compression retention of 70 percent, and 95 percent unstripped aggregate. The sand equivalent required is 25, although if blending is performed, the sand equivalent of the natural sand could be as low as 20. Some problems with clay balling have been reported with sands that have sand equivalencies in the vicinity of 20-22 when a drum mixer is used. In conventional asphalt plants, the clay balls are screened out and have not caused any problems. Gradation specifications for the sand require that 100 percent pass the 300-μm (No. 50) sieve and between 2 and 20 percent pass the 75-μm (No. 200) sieve.

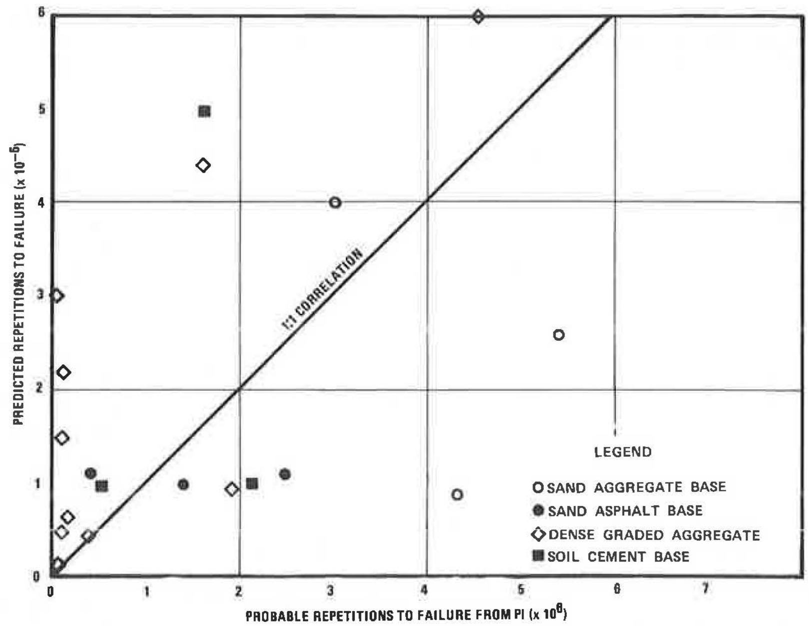
Experience in Georgia and elsewhere has shown that a dirty sand that has approximately 4-7 percent clay is probably best, provided the clay breaks down and does not form balls during mixing.

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

Maryland

The Maryland Department of Transportation typically has placed 130-180 mm (5-7 in) of AC over a 50-mm (6-in) sand-asphalt base. An asphalt content of typically 4.2-4.8 percent is used, and good performance is observed for this type construction. Specifications allow the use of either a natural sand, screenings, or sand-aggregate blends. The only gradation requirement is that not more than 12 percent pass the 75-μm sieve. Also, the sand-asphalt base mixes are required to have a 50-blow Marshall stability of not less than 1.1 kN (250 lbf) and flows of less than 16. The sand-asphalt base is placed at a density of 95 percent of the 50-blow Marshall value.

Figure 4. Relationship between observed and predicted pavement failure for selected Maryland pavements.



To overcome problems experienced with separation between 75-mm (3-in) layers, Maryland has begun either to use one 150-mm (6-in) layer or to increase the asphalt content by approximately 0.5 percent (4). The sand-asphalt is mixed at a temperature of about 149°C (300°F), and rolling begins at about 77°-132°C (260°-270°F). A detailed description of construction of thick and thin lifts is given elsewhere (4, 5).

Sand-asphalt base mixes have been frequently used for subdivision streets, parking lots, and other private work in Maryland. Usually a 40-mm (1.5-in) AC surfacing is placed over a 130-mm (5-in) sand-asphalt base that has about 4 percent asphalt cement. To minimize the quantity of asphalt cement, a dirty sand is used that has loose gradation requirements. The plant is run on the cold side at 120°-135°C (250°-275°F), and a mixing time of 30-35 s is used. This low asphalt content sand-asphalt is compacted immediately after placement by using a rubber-tired roller followed about 60 m (200 ft) by a steel-wheel roller. For this sand-asphalt base, 150 mm is assumed to be equivalent to 100 mm (4 in) of crushed-stone black base.

Maryland Base-Course Study

Stromberg (6, 7) studied the performance of 31 pavements in Maryland that have various base types, including sand-asphalt, soil-cement, sand-aggregate, crushed stone, gravel, and water-bound macadam. Traffic loadings were on the order of 1-2 million 80-kN (18-kip) equivalent axle loads. The soil-cement pavement sections typically consisted of a 150-mm soil-cement base placed beneath 150 mm of AC. The sand-asphalt sections consisted of 75-90 mm (3-3.5 in) of AC above a 130- to 180-mm (5- to 7-in) sand-asphalt base. Extraction tests indicated that the sand-asphalt bases had an average asphalt content of 4-4.5 percent, 3.7-4.3 percent fines, and void contents in the range of 13.8-14.6 percent. The soil-cement base had an average unconfined compressive strength of 11 400 kPa (1653 lbf/in^2).

Two of the three sand-asphalt base pavements were found to perform extremely well (Figure 4), and the performance of the third section was below that predicted. In general, the sections that had sand-asphalt and soil-cement bases exhibited good performance when compared with the dense-graded aggregate base pavements. Predicted pavement life was generally greater than observed. After 1 million repetitions, the sand-asphalt base sections had 15 mm (0.15 in) of rutting, which was approximately the same as in the dense-graded aggregate-base sections. Probably the relatively small amount of rutting in the sand-asphalt bases was primarily due to using an average asphalt content of only 4-4.5 percent and a relatively high degree of compaction.

South Carolina

The South Carolina Department of Highways uses sand-asphalt extensively for bases in the Coastal Plain. On Interstate work, a structural section has been used consisting of a 50-mm (2-in) AC surfacing, 100-mm (4-in) AC binder, and 150-mm (6-in) sand-asphalt base. Thicker sections were previously used. Only A-4 soils or better are used in the top 460 mm (18 in) of the subgrade for Interstate work. On primary roadways, a section often used consists of a 40- to 60-mm (1.5- to 2.5-in) AC surfacing, 60-mm (2.5-in) AC binder, and a 150-mm sand-asphalt base.

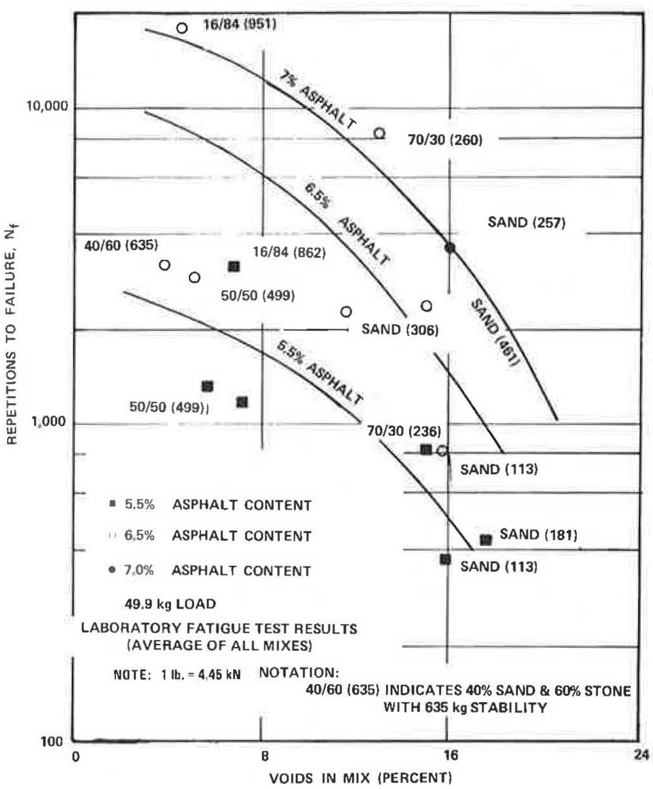
Sand-asphalt is used as a thin surface overlay on existing secondary roads; the overlay thickness typically varies from 19-20 mm (0.75-0.8 in). Sand-asphalt is seldom used for new construction on lightly traveled roads in South Carolina.

Many pavements constructed with sand-asphalt bases (such as I-20) have performed satisfactorily. One section on I-20 was observed to be in excellent condition after 1.2 million equivalent 80-kN axle loads (one direction). Cracking was not observed in this or similar sections, although rut depths measured with a 1.2-m (4-ft) straightedge were typically 6-10 mm (0.25-0.4 in). This section consists of 200 mm (8 in) of AC overlaying a 200-mm sand-asphalt base. No problems with ponding of water have been reported on I-20, which has a cross slope of 1.67 percent.

An AC-20 viscosity grade asphalt cement usually is used in sand-asphalt mixes that have asphalt contents that vary from 4.2 to 4.8 percent. Substitution of local sands for the finer portions of surfacing and binder mixes is also permitted in South Carolina. For sand-asphalt base mixes, essentially the only gradation specification requires 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 1.3-kN (300-lbf) Marshall stability mix is used for most work, a stability of 2.2 kN (500 lbf) has been used on some primary and Interstate construction. In some instances, sand-asphalt mixes have been used that consist entirely of crushed-stone screenings and have a sand equivalent greater than 35. The sand-asphalt is mixed at temperatures from 121°C to 163°C (250°F to 325°F) with a maximum reduction in temperature of 11°C (20°F) at the time of rolling.

No specification requirements are placed on either density or rolling procedures. South Carolina has experienced rutting problems in some sections; reported rut depths in the worst case were 25-40 mm (1-1.5 in) 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 250 mm (10 in) in thickness.

Figure 5. Effect of air voids and asphalt content on fatigue life of sand-asphalt and sand-stone blend mixes.



LABORATORY FATIGUE AND RUTTING TESTS

Fatigue and rutting tests were performed on a wide range of sand-asphalt and sand-stone base-course mixes. The materials used, test procedures, and equipment have been described in detail elsewhere (5). Bituminous base materials tested included both pure sand-asphalt mixes and also sand-stone blends that have stone contents that vary 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 until failure at the center of a rectangular beam specimen supported on a rubber subgrade. The repeated-load triaxial test used to evaluate the rutting properties of the mix consisted of subjecting a cylindrical specimen to 100 000 repetitions of axial load. An axial repeated deviator stress of 170 kPa (25 lbf/in^2) and a confining pressure of 34 kPa (5 lbf/in^2) was used for this study (5).

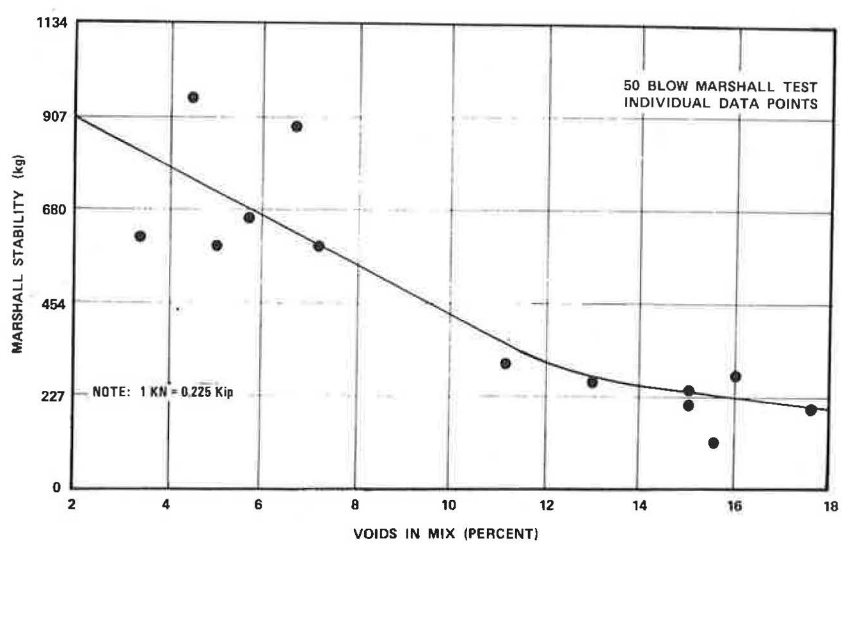
Fatigue Test Results

Recent research has shown that, in general, fatigue curves given in terms of tensile strain cannot be directly compared (5, 8, 9). The constant-load method of interpreting the fatigue test results, therefore, was used in this study (5). The constant-load method of interpretation consists 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 constant-load method gives a reliable comparison when the fatigue test simulates field support and loading conditions with reasonable accuracy (5, 9).

Larger asphalt contents and lower air voids in the sand and sand-stone blend mixes were found to increase fatigue life significantly, as illustrated in Figure 5.

A general trend was found between air voids in the mix and Marshall stability for the sand-asphalt and sand-stone blend base mixes tested (Figure 6). Therefore,

Figure 6. Relationship between Marshall stability and voids in mix for sand-asphalt and sand-stone blends tested.



the data from Figure 5 can be replotted by using 50-blow Marshall stability rather than void content, as illustrated in Figure 7. Increased fatigue life with increasing Marshall stability and asphalt content occurred for Marshall stabilities between 0.9 to 8.9 kN (200 to 2000 lbf), which was the range of stabilities tested.

These test results clearly show that the fatigue life of a sand-asphalt or sand-stone blend mix is directly related to asphalt content and air voids that have also been found for conventional mixes (7, 8). For the sand and sand-stone mixes studied, other undefined variables also appeared to influence the test results. Although Marshall stability is probably not a fundamental independent variable, as air voids appears to be, Marshall stability (or air voids) together with asphalt content can be used in design for estimating the fatigue resistance of sand-asphalt and sand-stone blends.

Rutting Test Results

Rutting test results for the sand and sand-stone asphalt mixes are summarized in Figures 8 and 9. Laboratory test results are presented in terms of the theoretical rut depth that would occur in the base of a typical pavement section that has a sand-asphalt or sand-stone blend base. Comparisons of rut depths are made for full-depth bituminous pavements that have a 90-mm (3.5-in) AC surfacing and a 180-mm (7-in) sand-asphalt or sand-stone blend base. This pavement section is assumed to be constructed over a reasonably good subgrade that has a modulus of elasticity of approximately 28 000 kPa (4000 lbf/in^2). The pavement section is assumed to be located in the coastal plain area of the southeast, where the mean pavement temperature for rutting is close to 35°C (95°F). The rut depths given are for 1.2×10^6 equivalent 80-kN (18-kip) single axle loads.

By comparing predicted rut depths rather than the measured plastic strains, a better feeling is developed for the effect of the mix variables on the actual relative magnitude of rutting that is likely to develop in a typical pavement section. The theoretical approach used to calculate the rut depth has been described in detail elsewhere (5).

With an increase in asphalt content, rut depth in the sand-asphalt and sand-stone blends tested rose at a slightly increasing rate (Figure 8). For an increase in asphalt content from 5.5 to 6.5 percent, the rut depth increased by 40-70 percent, which is similar to the rate for a typical AC black-base mix.

Figure 8 shows that the addition of crushed stone to a sand-asphalt mix is effective in reducing rutting of the mix. The reduction in rut depth due to increased stone content is probably due to increased internal friction of the mineral skeleton that results 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.

The influence on rut depth of Marshall stability and asphalt content for Altamaha sand-stone blends is shown in Figure 9 as solid lines. 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 indicates that Marshall stability can be used as a rough guide for evaluating the relative beneficial effect on rutting of blending crushed stone with sand or blending two sands together.

When all the mixes shown in Figure 9 are considered, more scatter in data occurs than for just the Altamaha mixes that were prepared from the same materials. This indicates that other less well-defined characteristics of the mix, such as grain size, angularity, and gradation of the sand, also have important effects on rutting.

The theoretical method proposed by Barksdale and others (5) 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 relative susceptibility of a sand-asphalt base to rutting can, however, be obtained from the generalized design relationship given in Figure 10 (5). 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 and subgrade. The design section on which this figure is based was previously given in this section.

Figure 10 was developed for a mean pavement temperature with respect to rutting of 35°C (95°F), which was found to exist in the coastal plain areas of the southeast. Since the magnitude of rutting is influenced by other factors (in addition to voids or Marshall stability), this figure should only be used as a general guide. Neither of the proposed methods fully considers the relationship of constant or increasing rut depth with base thickness observed at the Marianna test road for low-stability sections (Figure 3).

ALLOWABLE RUT DEPTH

The allowable rut depth that 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 on the cross slope of the pavement and the transverse width of the rut. For rolled asphalt construction in England, Lister and Addis (10) have found that rut depths greater than approximately 13 mm (0.5 in) result in the ponding of water on pavements that have a 2.5 percent cross slope. They also found that the optimum time for overlaying a rolled asphalt pavement corresponds to a rut depth of approximately 10 mm (0.4 in) measured with a 1.8-m (6-ft) straightedge. The 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 19 mm (0.75 in) is generally defined as pavement failure.

The PSI value of sections at the American Association of State Highway Officials (AASHO) Road Test were found by Lister and Addis (10) to be inversely proportional to rut depth. The relationship appears to be conservative for pavements that have sand-asphalt bases (5).

Field inspections indicate that the rutting developed in sand-asphalt base pavements is relatively wide, which possibly accounts for the higher than expected PSI ratings. To take into consideration the width of the rut, Verstraeten and others (11) have developed rut criteria for use in Belgium based on the transverse slope of the rut. For highways in Switzerland, Huschek (12) proposed a 4-mm (0.15-in) limiting water film on the surface. To satisfy this criterion, Huschek indicated that the rut depth must be less than 18 mm (0.7 in) for a 2.5 percent cross slope, which agrees with the criteria for rut depth proposed by Verstraeten and others (11). Use of sand-asphalt mixes requires that relatively large rut depth be permitted. For now, an allowable average design rut depth of 10 mm (0.4 in) is recommended for primary and Interstate pavements and 15 mm (0.6 in) for secondary roads constructed by using high asphalt contents or sand-asphalt mixes. This level of rutting is in agreement with the finding of Lister and Addis (10). Rut depths of this magnitude have been observed on test sections in Florida (2) and Interstate pavements in South Carolina (5). No problems due to this level of rutting were reported either

Figure 7. General effect of Marshall stability and asphalt content on fatigue life: all sand-asphalt and sand-stone mixes.

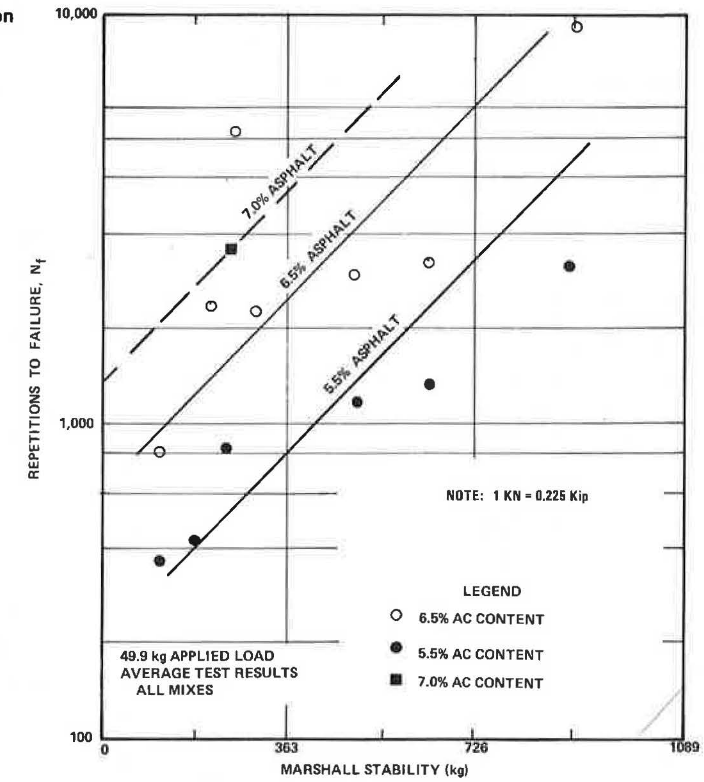


Figure 8. Relationship between stone content and rut depth for varying asphalt contents.

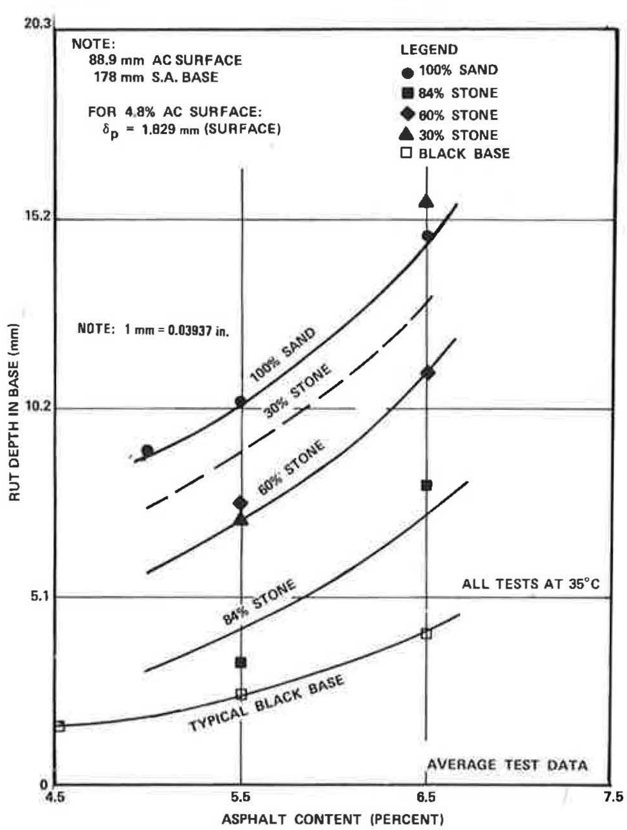
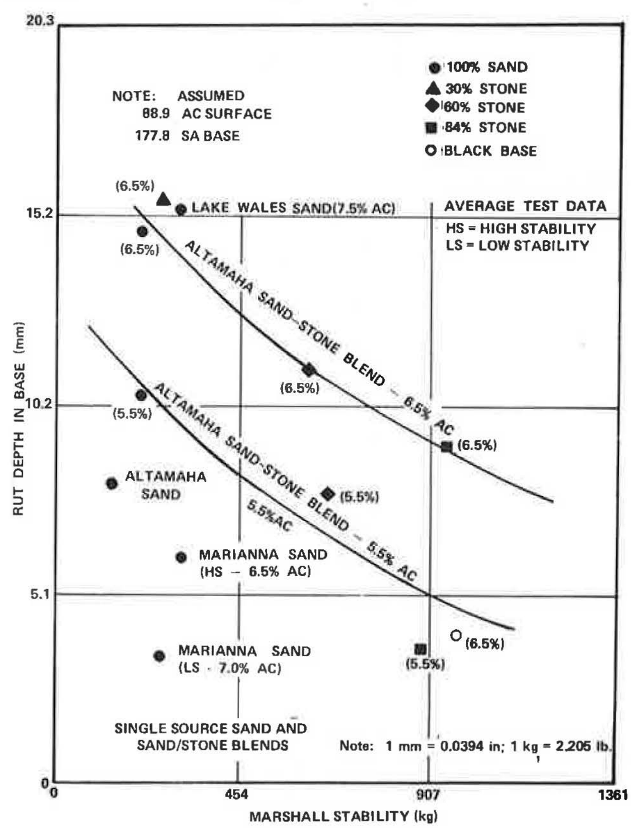


Figure 9. Effect of Marshall stability and asphalt content on rut depth.



in these test roads or at numerous pavements visited during this study that have similar levels of rutting.

STRUCTURAL THICKNESS DESIGN

At the current time, the required structural section can be most readily determined by using the AASHO Interim Guide (13) and a PSI value of at least 2.5 (5). Of course, other more mechanistic design methods based on the fundamental fatigue and rutting modes of distress can be used.

A conservative value of the laboratory test results should be used in estimating the soil support value for use in the Interim Guide. An analysis of the results of a field performance study in Maryland and also past experience indicates that the soil support value based on CBR results is often too high (7). The soil support values given in Table 2, based on the American Association of State Highway and Transportation Officials (AASHTO) soil classification, can be used as a general guide in

Figure 10. Design relationship for estimating preliminary rut depths in sand-asphalt and sand-stone blend bases: 266.7 mm structural section supported by a fair subgrade.

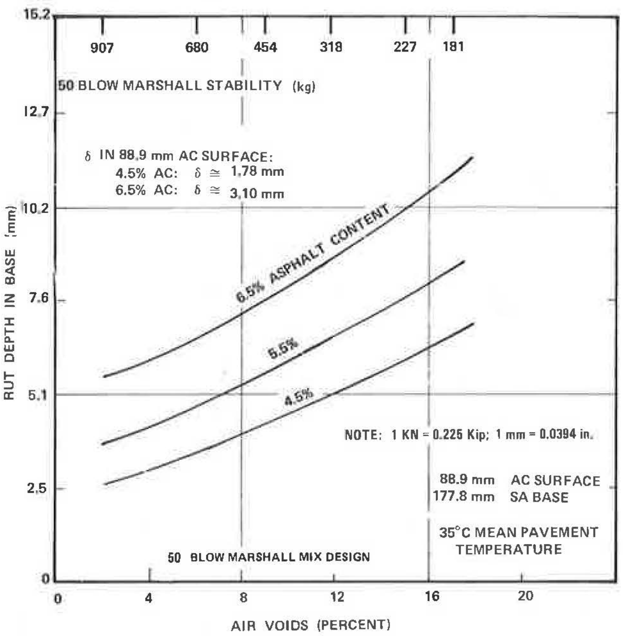


Table 2. Limiting soil support values based on AASHTO soil classification.

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
A-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 3. Recommended AASHTO Interim Guide structural coefficients for thickness design.

			General Requirements			
Structural Layer	Class	Structural Coefficient	Asphalt Content (%)	Air Voids (%)	Marshall Stability[a] (kN)	Other
Surface and binder course (weighted average)						
AC	1	0.48	>6.0	2-4	>6.67	
	2	0.44	≥4.8	2-6	>5.34	
	3	0.35	<4.4	2-8	>3.11	
Sand asphalt	1	0.35	>5.8	<14	>2.45	
	2	0.27	<4.8	<18	>1.78	
Base course						
Crushed stone, untreated	1	0.14				Well graded; 38 mm or greater top size; 3-8 percent fines; 100 percent T180 compaction
AC	1	0.34	>5.8	2-4	>5.34	
	2	0.28	<4.8	<8	>5.34	
Sand asphalt	1	0.25	>5.8	14	>2.67	
	2	0.17	<4.5	18	>1.56	
Sand cement	1	0.24				>4138 kPa, 7-day compressive strength
	2	0.18				>2759 kPa, 7-day compressive strength
Inverted structural section—experimental[b]						
Unstabilized sand base		0.10-0.12				Clean, medium to coarse sand, <4-8 percent fines
Unstabilized sand-crushed stone blend		0.16				

Note: 1 kN = 225 lbf; 1 mm = 0.04 in; 1 kPa = 0.15 lbf/in².
[a] Given Marshall stabilities are for a 50-blow mix design.
[b] Structural section consisting of unstabilized clean sand or crushed stone placed between a sand-cement base and AC surface course. Use structural coefficients for sand-cement base and AC surface course given above.

establishing upper limiting values.

Recommended structural coefficients for use in the AASHO Interim Guide for surface and base courses are given in Table 3 for construction by using sand-asphalt and sand-cement pavement sections. The actual value of the structural coefficients can vary greatly depending on the quality of materials used, level of stabilization, construction specifications, and the quality control program followed during construction. In general, the higher-quality 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 content (7).

DISCUSSION

Sand-asphalt and sand-stone blend AC can be successfully used as base courses, surfacings, and leveling courses. The 50-blow Marshall mix design method supplemented by the findings presented in this paper gives a practical procedure for designing sand-asphalt base mixes. At the current time, two different approaches were found to be followed in the design of sand-asphalt base mixes in the four states visited. Florida designs a sand-asphalt mix that typically has 6.5-7.5 percent asphalt content and a 50-blow Marshall stability greater than 2.2 kN (500 lbf) and often greater than 3.1 kN (700 lbf). This type of sand-asphalt base mix is usually placed beneath AC surfacings, typically 75 mm (3 in) in thickness and used for moderate to heavy traffic-loading conditions. On the other hand, Maryland and South Carolina use a mix that has typically 4 to 5 percent asphalt content. This type mix is generally placed beneath 125-150 mm (5-6 in) of AC and used under moderate to heavy traffic-loading conditions. Georgia follows a design between these two extremes.

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 laboratory fatigue tests indicate that the fatigue resistance of a sand-asphalt mix can be increased by a factor of approximately four by increasing the asphalt content from 4.5 to 5.5 or 6 percent. Likewise, an increase in Marshall stability from 1.5 kN (350 lbf) to 3.1 kN should increase the laboratory fatigue life by a factor of about three. Based on observed field performance and laboratory fatigue tests, the recommendation is made that, for at least moderate to heavy traffic conditions and AC surfacings 75-100 mm (3-4 in) in thickness, the higher-quality sand-asphalt base construction, which has Marshall stabilities greater than 2.2-3.1 kN, should be used.

For thicker AC surface courses or light traffic conditions, lower stability or asphalt-content mixes can be used successfully. For this 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 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.

Blight and others (14) have found that soluble salt contents greater than 2 percent can cause deterioration of an AC 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. Three samples of sand from tidal fluctuation areas in rivers were tested for soluble salt content. Very low salt contents were found present in these sands (5). Based on the absence of observed problems in the field and these limited results, high soluble salt content is probably not a major problem, although some sands undoubtedly have excessive soluble salt concentrations present (14).

In general, the sand equivalent has been found in this study and others (15, 16) not to be a good indicator of the quality of a sand for use in sand-asphalt. Clay balling in some materials may become a problem when the sand equivalent is less than 22-24 when these materials are used in a drum dryer. Sands with high sand equivalent values 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 (17).

Both experience and the laboratory tests indicate that approximately 4-7 percent clay (as determined by the elutriation test) is actually desirable in a sand-asphalt mix. Hence, 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 (15) and summarized elsewhere (5) can also be used as a general guide for sand-asphalt mixes.

Finally, the use of sand-stone blend AC base-course mixes with up to 75 percent stone content offers an excellent way on some projects to reduce the overall cost of the mix while at the same time obtaining a high-quality AC base. These mixes can be designed to have good fatigue properties and reasonably low asphalt contents in the range of 4.5 to 5.5 percent. At the same time, such mixes should experience on the order of 25 percent less rutting than a pure sand-asphalt (Figure 8).

GENERAL CONCLUSIONS

Field inspections conducted in four southeastern states indicate that sand-asphalt and sand-stone blend asphalt mixes can be successfully used as base courses under both light and heavy traffic conditions. Rutting in pavements constructed by using a sand-asphalt base is typically between 8 and 15 mm (0.3 and 0.6 in). As a result, more consideration must be given to rutting in the mix design of sand-asphalt bases compared with conventional mixes. An allowable rut depth of 10 mm (0.4 in) is recommended on primary and Interstate pavements and 15 mm (0.6 in) on secondary roadways.

The 50-blow Marshall method can be used for mix design of sand-asphalt bases. Results of dynamic tests are described for the evaluation of fatigue and rutting of sand and sand-stone asphalt mixes. Variables that influence the fatigue and rutting performance of these sand-asphalt mixes are discussed.

Use of sand-stone blend AC base-course mixes offers an excellent way on some projects to reduce the overall cost of the mix, while also obtaining a high-quality AC base. Sand-stone blend mixes should experience on the order of 25 percent less rutting than a pure sand-asphalt.

ACKNOWLEDGMENT

I would like to express my sincere appreciation to Jim Niehoff and Jess Schroeder for performing the laboratory tests. This work was carried out under a grant from the Georgia Marine Science Center, University of Georgia, which was funded by the Office of Sea Grant, National Oceanic and Atmospheric Administration, U.S. Department of Commerce. Finally, appropriate acknowledgment is given to Vicki Clopton for carefully typing the manuscript.

REFERENCES

1. J. C. Goodknight and L. L. Smith. Experimental Flexible Pavement Materials Report. Florida Department of Transportation, Gainesville, Dec. 1966, 35 pp.
2. Summary Data Reports for Study P-1-63, Flexible Pavement Design. Office of Materials and Research, Florida Department of Transportation, Gainesville, 1976.
3. H. G. Godwin and R. L. McNamara. Summary Data Reports for Study P-1-63, Flexible Pavement Design. Office of Materials and Research, Florida Department of Transportation, Gainesville, Res. Rept. 196, Aug. 1976.
4. O. E. Briscoe. Deep Lift Construction on Maryland's State Roads. Paper presented at the 6th Annual Maryland Asphalt Paving Conference, College Park, MD, Feb. 1968.
5. R. D. Barksdale, J. W. Niehoff, and J. A. Schroeder. Final Report on Utilization of Local Sands in Highway Construction. Georgia Marine Science Center, Univ. of Georgia, Rept. 79-4, 1979, 195 pp.
6. F. J. Stromberg. Investigation of Base Courses for Flexible Pavements. Bureau of Research, Maryland State Highway Administration, Final Rept., Sept. 1972.
7. R. D. Barksdale. Performance of Asphalt Concrete Pavements. Journal of Transportation Engineering, Proc., ASCE, TE1, Jan. 1977, pp. 55-73.
8. R. D. Barksdale and J. H. Miller. Development of Equipment and Testing Techniques for Evaluating Fatigue and Rutting Characteristics of Asphalt Concrete Mixes. School of Civil Engineering, Georgia Institute of Technology, Atlanta, 1975.
9. R. D. Barksdale. Practical Application of Fatigue and Rutting Tests on Bituminous Base Mixes. Paper presented at the 1978 Annual AAPT Meeting, Lake Buena Vista, FL, Feb. 1978.
10. N. W. Lister and R. R. Addis. Field Observations of Rutting in Practical Implications. TRB, Transportation Research Record 640, 1977, pp. 28-34.
11. J. Verstraeten, J. E. Romain, and V. Veverka. The Belgium Road Research Center's Overall Approach to Asphalt Pavement Structural Design. Proc., Fourth International Conference on Structural Design of Asphalt Pavements, Ann Arbor, MI, Vol. 1, Aug. 1977, pp. 298-324.
12. S. Huschek. Evaluation of Rutting Due to Viscous Flow in Asphalt Pavements. Proc., Fourth International Conference on Structural Design of Asphalt Pavements, Ann Arbor, MI, Vol. 1, Aug. 1977, pp. 297-508.
13. AASHO Interim Guide for Design of Pavement Structures. AASHO, Washington, DC, 1972.
14. G. E. Blight, J. A. Stewart, and P. F. Theron. Effects of Soluble Salt on Performance of Asphalt. Proc., Second Conference on Asphalt Pavements for Southern Africa, Durban, South Africa, 1974, pp. 3-1—3-13.
15. C. R. Freeme. The Selection of Sands Suitable for Gap-Graded Mixtures. National Institute for Road Research, Pretoria, South Africa, Rept. RB/2/75, 1975.
16. C. P. Marais. Tentative Mix-Design Criteria for Gap-Graded Bituminous Surfaces. TRB, Transportation Research Record 515, 1974, pp. 132-145.
17. S. M. Acott. The Development and Mix-Design of Gap-Graded Asphalt. National Institute for Road Research, Pretoria, South Africa, Rept. RB/1/75, 1975.

Performance of Sand-Asphalt and Limerock Pavements in Florida

Charles F. Potts, Byron E. Ruth, and Lawrence L. Smith

This paper presents a summary of three test roads that were constructed between 1964 and 1971 by the Florida Department of Transportation. The test sections were designed and constructed to be included as a part of the state's Satellite Test Road Program. The sections were designed to provide variations in surface and base-course thicknesses, type of base materials, and stability levels of sand-asphalt hot mix. The base courses evaluated included limerock, sand-asphalt hot mix, and shell. The individual test sections have been monitored to determine their structural behavior, condition, and serviceability. Test parameters for the constructed pavements were analyzed as a basis of comparison to test data on performance collected over several years.

The performance of flexible pavements in Florida has been investigated more intensely during the past 10 years. Numerous test roads have been constructed to evaluate design and construction material variables. This paper presents a summary of three test roads that were constructed between 1964 and 1971. These test roads were designed to provide variations in surface and base-course thicknesses, type of base-course materials, and stability of sand-asphalt hot mix (SAHM). Base-course materials include limerock, SAHM, and shell. The quality of the aggregates would probably be considered as poor in comparison to the harder, more durable crushed stone and gravels used in other states.

These test roads have been monitored to determine their structural behavior, condition, and serviceability. This information was extracted from data summaries and reports prepared by the Florida Department of Transportation (1, 2). Test parameters for the constructed pavements were analyzed for comparison to performance test data that were collected over several years. Additional data were selected from reports that evaluated the fatigue fracture and dynamic properties of specimens recently cut from some of the existing test road sections (3, 4).

The significance of the test road monitoring programs and laboratory evaluation tests is evident when the performance achieved by using marginal aggregates is considered. Both limerock and SAHM bases can con-

Figure 1. Typical limerock base aggregate gradation.

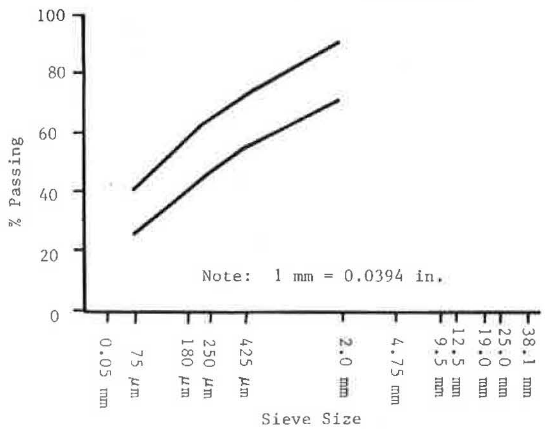


Figure 2. Typical SAHM base aggregate gradation.

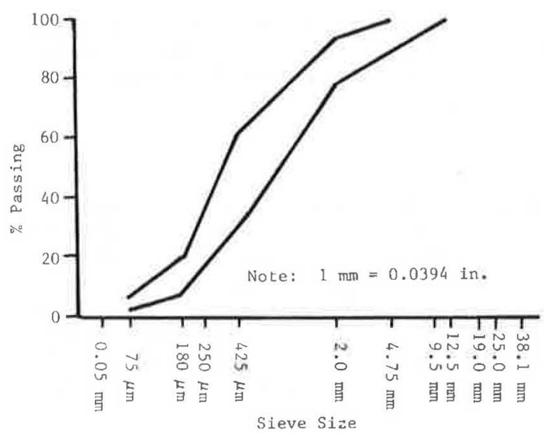


Table 1. Design characteristics—Marianna test road SAHM base thickness and stability.

Base Thickness (cm)	Section Stability High (543 kg)	Section Stability Low (362 kg)
10.2	3	11
10.2-15.2 tapered	2, 4	10, 12, 14
15.2	5	13
15.2-20.3 tapered	1, 6	9
20.3	7, 8	15, 16

Note: 1 cm = 0.394 in; 1 kg = 2.2 lb.

sistently provide excellent serviceability when proper design and construction controls are implemented.

GENERAL CHARACTERISTICS OF LIMEROCK BASES

Florida limerock is generally categorized as Miami oolite in the southern portion of the state and as Ocala limerock in the northern portion. Some hard limestones can be found in the state, but the majority of the rock that is mined is considerably softer, lower quality, and characteristically is called limerock. Bulk specific gravity values generally range between 2.15 and 2.50 and water absorption values are typically 3-7 percent, ranging to more than 15 percent in extremely poor-quality limerocks. Specifications for limerock base materials are given below, and the ranges in typical aggregate gradations are presented in Figure 1.

Carbonates—Ocala limerock (for limerock base), 95 percent minimum; Miami limerock, 70 percent minimum;

Organic matter—0.5 percent maximum;

Chemical change—limerock that shows a significant tendency to air slake or to undergo chemical change under exposure to the weather will not be acceptable;

Liquid limit and plasticity—limerock shall be nonplastic and have a liquid limit (LL) < 35; and

Gradation—97 percent minimum passing the 8.9-cm (3.5-in) sieve and graded uniformly down to dust; the fine material shall consist entirely of dust of fracture.

Limerock bases are generally compacted to not less than 98 percent of AASHTO T180. Pavement thicknesses vary from a 10.2-cm (4-in) limerock base with a 2.5-cm (1-in) asphalt surface to 25.4-cm (10-in) bases with up to 10.2 cm (4 in) of asphalt leveling and surface course. The structural properties of limerock bases are very susceptible to water. However, this is seldom a problem where pavements have adequate surface drainage and were constructed over sandy subgrade soils that are prevalent in Florida.

Another unique aspect of limerock bases is the increase in stiffness that occurs with age and is attributed to a form of cementing action between aggregate particles. Plate bearing values often increase by more than 50 percent within a few years after construction. For example, typical plate bearing values of 172-241 MPa (25-35 E3) will increase to more than 276 MPa (40 E3).

GENERAL CHARACTERISTICS OF SAHM BASES

SAHM bases are usually considered to be inferior to the more conventional base-course materials. This belief stems from experience obtained with local sands that do not provide adequate stability and are difficult to compact in the field because of rutting by compaction equipment. Compacted densities may also be low when subgrade soils do not provide an adequate working platform. However, experience in Florida with SAHM bases has been reasonably good. This is primarily due to the blending of crushed limerock screenings, shell, or other crushed material with local sands to improve the properties of the SAHM. The specifications for SAHM aggregates and mixtures are presented below. The range in aggregate gradations for SAHM bases is illustrated in Figure 2.

Sand—Local sand shall be nonplastic with hard, durable grains free from deleterious substances and shall not contain more than 7 percent clay.

Blended aggregate--Local sand blended with other materials (e.g., crushed shell, rock screenings, mineral filler, or other material) shall not exceed 12.5-mm (0.49-in) maximum size nor contain in excess of 12 percent passing the 75-μm (No. 200) sieve.

Mixture requirements—Hubbard-Field stability, 362 kg (498 lb) minimum unless otherwise specified; mineral filler content, as required for stability, 12 percent maximum; mineral aggregate, 91-96 percent by weight of mix; and asphalt cement (AC-20), 4-9 percent by weight of mix.

Compaction requirements—based on equipment and specified rolling procedures; density requirements are not specified.

Table 2. Summary of SAHM base test results.

Test	High-Stability Sections (1-8) Tests (N)	Mean	SD	Coefficient of Variance (%)	Low-Stability Sections (9-16) Tests (N)	Mean	SD	Coefficient of Variance (%)
Field density (Mg/m³)	23	2.00	0.02	1	24	1.99	0.38	2
Field CBR	8	20.1	14.3	71	8	26.2	11.3	43
Plate bearing value (MPa)	8	82.50	13.20	16	8	65.20	18.25	28
Marshall stability (kg)	19	305	57.95	19	24	256	40.98	16
Hubbard-Field stability (kg)	19	704.50	77.49	11	24	560.60	72.88	13
Extraction results (% passing)								
12.5 mm	8	100	0		2	100		
9.5 mm	8	99	0.5		2	100		
4.75 mm	8	98	0.6		2	100		
2.0 mm	8	95	0.7		2	96		
425 µm	8	31	0.7		2	44		
180 µm	8	8	0		2	13		
75 µm	8	4	0		2	4		
Bitumen (%)	8	6.4	0.1		2	6.9		

Note: 1 Mg/m³ = 62.43 lb/ft³; 1 MPa = 145 lbf/in²; 1 kg = 2.2 lb; 1 mm = 0.039 in.

Table 3. Design characteristics—Palm Beach test road base materials and thickness.

Base Type	Base Thickness (cm)	Section Number
SAHM	7.6	1[a]
	11.4	2
	15.2	3
	7.6	7[b]
	11.4	8
	15.2	9
Shell	10.2	10
	15.2	11
	20.3	5
Limerock	10.2	4
	15.2	6

Note: 1 cm = 0.394 in.
[a]Base stability (Hubbard-Field) = 362 kg (796 lb) (design).
[b]Base stability (Hubbard-Field) = 543 kg (1195 lb) (design).

TEST ROAD PROJECTS

Marianna, US-90

The Marianna test road project was designed for evaluation of the stability and thickness of SAHM bases. The project is located in the westbound lanes of US-90 near Marianna, in the panhandle of Florida. Construction of the project was completed during the summer of 1964. The design characteristics of the test road are summarized below and in Table 1.

Surface type and thickness—type 1 asphalt concrete (AC): 7.6 cm (3.0 in), includes 5.1 cm (2.0 in) of AC binder;

Base type and thickness—SAHM: 10.2 cm (4.0 in), 10.2-15.2 cm (4.0-6.0 in) tapered, 15.2 cm, 15.2-20.3 cm (6.0-8.0 in) tapered, and 20.3 cm;

Base stability (Hubbard-Field)—362-543 kg (798-1197 lb) design; and

Subgrade strength (all sections)—limerock bearing ratio (LBR) 77, actual mean value; plate bearing value, 82 MPa (11 890 lbf/in²).

Compacted subgrade soils produced LBR values of 77. The mean Hubbard-Field stability values for the constructed SAHM base gave mean values of 704 kg (1557 lb) for the low-stability sections, which exceeded the design requirements. Table 2 presents a summary of test results for the SAHM bases.

Traffic has increased steadily from an average daily traffic (ADT) count of 2700 in 1964 to 4900 in 1979. As of 1978, approximately 1.68 million 80-kN (18-kip) axle loads have been applied to the test sections in the westbound traffic lane and about 1.45 million in the westbound passing lane.

Palm Beach

An experimental test section, 2.8 km (1.74 miles) long, was designed and constructed in 1970 as a portion of the SR-704 extension between FL-7 and Royal Palm Beach. The purpose of constructing the experimental test road was to evaluate the performance and strength equivalencies of various types of bases. Test sections were constructed by using limerock, shell, and SAHM bases of different thicknesses and stabilities as summarized below and in Table 3.

Surface type and thickness—type 2 AC, 3.8 cm (1.5 in); Marshall stability, 357-448 kg (787-988 lb); mean, 389 kg (858 lb);

Base type and thickness—SAHM: 7.6, 11.4, and 15.2 cm (3.0, 4.5, and 6.0 in); shell: 10.2, 15.2, and 20.3 cm (4.0, 6.0, and 8.0 in); and limerock: 10.2 and 15.2 cm; and

Subgrade strength (all sections)—LBR 40 (as constructed mean LBR 46); plate bearing value, 137 MPa (19 865 lbf/in²); and maximum density, 103.4 percent.

Tables 4 and 5 present the summary of test results for the different test sections constructed.

At the time of construction, the field test samples indicated that the original design values of 362 and 543 kg (800 and 1200 lb) Hubbard-Field stabilities were not being achieved. An in-depth evaluation revealed that the sandy shell component was of a different gradation than that used in the original design. This caused the reduction in stability. In order to not jeapordize the evaluation of the sections, it was decided to continue to use this design so that a differential in stabilities could be maintained. The only effect this had on the total study was the reduction in the stability level. This reduction would subsequently result in problems in obtaining adequate compaction in the field.

The air void contents for the SAHM ranged between 9.7 and 15.6 percent in 1977, after six years of traffic. Limerock (oolitic in origin) and shell bases were compacted to about 100 percent of AASHTO T180. Plate bearing values for the limerock bases were more than 30 percent greater than the values for the shell.

Table 4. Summary of SAHM base test results.

Test	Section Number	Tests (N)	Mean	SD	Coefficient of Variance (%)	95 Percent Confidence Limits Lower	Upper
Field density, nuclear (Mg/m^3)	1	3	1.94	0.01	0.6	1.91	1.97
	2	3	1.94	0.01	0.2	1.93	1.95
	3	3	1.93	0.01	0.6	1.91	1.97
	7	3	1.95	0.02	1.0	1.91	2.00
	8	3	1.94	0.01	0.2	1.94	1.96
	9	3	1.96	0.01	0.7	1.93	2.00
Core density, AASHTO T-166 (Mg/m^3)	1	10	1.96	0.02	0.9	1.95	1.97
	2	10	1.97	0.02	0.7	1.97	1.99
	3	10	1.98	0.01	0.5	1.93	1.99
	7	10	1.99	0.03	1.5	1.93	2.02
	8	9	2.01	0.02	0.8	2.00	2.20
	9	10	2.02	0.02	0.9	2.02	2.04
Field CBR[a]	1	3	25	2.8	11.3	18.1	32.3
	2	3	33	3.0	9.3	25.1	40.3
	3	3	34	5.0	14.7	21.6	46.5
	7	3	29	3.1	10.5	21.7	37.0
	8	3	22	1.2	5.4	19.2	25.1
	9	3	27	1.2	4.7	23.7	29.9
Plate bearing value[a] (MPa)	1	3	123	7.48	6.1	104	141
	2	3	104	5.21	5.0	92	117
	3	3	88	6.09	6.9	73	103
	7	3	118	12.97	11.0	86	151
	8	3	113	10.86	9.6	86	140
	9	3	100	0.90	0.9	98	102
Marshall stability (kg)							
Low	1	5	81.4	6.52	8.0	73.3	89.6
	2	7	90.0	16.48	18.3	74.7	105.4
	3	8	105.4	23.19	22.0	65.2	94.6
High	7	8	191.0	19.10	10.0	175.0	207.0
	8	8	167.4	26.62	15.9	145.2	189.6
	9	10	155.2	17.69	11.4	142.5	167.9
Hubbard-Field stability (kg)							
Low	1	7	210.4	30.50	14.5	182.4	238.9
	2	8	204.1	38.40	18.8	171.9	236.2
	3	8	206.3	33.01	16.0	178.7	233.9
High	7	8	347.5	38.61	11.1	314.9	379.6
	8	8	311.3	31.13	10.0	285.1	337.6
	9	8	330.3	12.88	3.9	319.9	337.1
Core thickness (cm)							
7.6	1	10	7.6	0.51	6.7	7.6	8.4
11.4	2	10	11.9	0.51	4.3	11.6	12.2
15.2	3	10	14.7	0.49	3.4	14.2	15.0
7.6	7	10	8.1	0.38	4.7	7.9	8.6
11.4	8	10	11.2	0.37	3.3	10.9	11.4
15.2	9	10	15.2	0.81	5.3	14.7	15.7
Typical extraction results (% passing)							
12.5 mm	1, 2, 3	19	100				
9.5 mm		19	99	0.5	0.5	98.6	99.1
4.75 mm		19	93	1.4	1.5	92.9	94.1
2.0 mm		19	86	2.0	2.3	85.5	87.2
425 μm		19	70	1.8	2.6	69.4	71.0
180 μm		19	20	1.8	9.4	18.8	20.3
75 μm		19	2	0.6	24.6	2.0	2.6
Bitumen[b] (%)		19	7.7	0.3	4.0	7.5	7.8
Typical extraction results (% passing)							
12.5 mm	7, 8, 9	18	100				
9.5 mm		18	98	0.9	0.9	97.6	98.5
4.75 mm		18	89	2.3	2.6	88.0	90.0
2.0 mm		18	78	1.8	2.3	77.2	79.0
425 μm		18	62	2.5	4.1	61.1	63.6
180 μm		18	19	2.3	12.4	17.5	19.8
75 μm		18	3	0.8	29.6	2.4	3.2
Bitumen[c] (%)		18	8.2	0.2	2.7	8.1	8.3

Note: 1 Mg/m^3 = 62.43 lb/ft^3; 1 MPa = 145 lbf/in^2; 1 kg = 2.2 lb; 1 cm = 0.394 in; 1 mm = 0.039 in.
[a]These tests for the SAHM base are dependent on temperature and rate of loading.
[b]Design asphalt content: 8.0 percent.
[c]Design asphalt content: 8.5 percent.

Table 5. Summary of limerock and shell base test results.

Base	Test	Section Number	Tests (N)	Mean	SD	Coefficient of Variance (%)	95 Percent Confidence Limits Lower	Upper
Limerock	Field density (Mg/m^3)	4, 6	12	2.06[a]	0.02	1.2	2.05	2.08
	Percent of maximum density		10	100.1	1.40	1.4	99.1	101.1
	Plate bearing value (MPa)		6	240	33.88	14.1	205	276
	Laboratory LBR		10	193	39.8	20.6	164.9	221.8
Shell	Field density (Mg/m^3)	5, 10, 11	18	1.92[b]	0.04	2.2	1.90	1.94
	Percent of maximum density		18	99.3	1.61	1.6	98.5	100.1
	Plate bearing value (MPa)		9	183	44.78	24.5	148	217
	Laboratory LBR		15	101	9.6	9.5	95.7	106.3

Note: 1 Mg/m^3 = 62.43 lb/ft^3; 1 MPa = 145 lbf/in^2.
[a]100.1 percent AASHTO T-180.
[b]99.3 percent AASHTO T-180.

Traffic in terms of ADT was approximately 1700 in 1979. The total 80-kN (18-kip) axle loads are about 120 000, but the distribution between lanes is biased because trucks loaded with fill material travel on the eastbound lane and return empty on the westbound lane.

Lake Wales

The Lake Wales test road was specifically designed for evaluation of limerock and SAHM base materials. Construction of the four-lane facility was completed in January 1971, and since that time it has accommodated about 2.5 million 80-kN axle load repetitions in the northbound traffic lane and about 0.6 million in the passing lane.

The various base and surface thickness combinations for the test sections are presented below and in Table 6.

Surface type and thickness—type 1 AC, 3.8 and 7.6 cm (1.4 and 3.0 in);

Base type and thickness—SAHM: 7.6, 10.2, 15.2, 20.3, and 25.4 cm (3.0, 4.0, 6.0, 8.0, and 10.0 in); limerock: 7.6, 10.2, 15.2, 20.3, and 25.4 cm; and

Subgrade strength (all sections)—LBR 40.

Tables 7 and 8 give the aggregate gradation for samples of limerock base and SAHM base obtained after compaction in the field. The SAHM was designed with a 50-50 blend of local sand and crushed stone screenings and 7.7 percent asphalt content. The limerock base was compacted to between 97 and 101.5 percent of AASHTO T180. Details pertaining to field test data are presented in the following discussion on evaluation of performance.

EVALUATION OF TEST ROADS

Test roads are generally evaluated on a yearly basis, although the exact time interval varies because of work loads and equipment availability. Construction test records and pavement evaluations taken immediately after construction form the basis for documentation and comparison to future evaluations, to observed pavement distress, and to changes in pavement serviceability. Pavement condition and serviceability surveys most often include 89-kN (20-kip) Benkelman beam static deflections (rebound), in inside and outside wheelpaths of both passing and traffic lanes, surface texture measurements, rutting, and the square meters of cracking and patching per 92.8 m^2 (1000 ft^2) of pavement. The present serviceability index (PSI) is determined during each evaluation according to the equation:

$$PSI = 5.0 - 3.947\ [\log(1 + \overline{SV})] - \text{texture} - 0.155\ \text{rutting} - 0.007\ [\log(C + P)] \quad (1)$$

Table 6. Design characteristics—Lake Wales test road base materials and thickness.

Base Type	Base Thickness (cm)	Section Number: 3.8-cm Surface Thickness	Section Number: 7.6-cm Surface Thickness
Limerock	7.6	3A	3B
	10.2	2B	2A
	15.2	4B	4A
	20.3	5A	5B
	25.4	1B	1A
SAHM	7.6	6B	6A
	10.2	8B	8A
	15.2	7A	7B
	20.3	9A	9B
	25.4	10B	10A

Note: 1 cm = 0.394 in.

Table 7. Lakes Wales limerock data.

Test	Section Number	Mean	SD	Coefficient of Variance (%)	95 Percent Confidence Limits: Lower	95 Percent Confidence Limits: Upper
Mechanical analysis (% passing)	1A and 1B through 5A and 5B					
2.0 mm		81.0	4.7	5.8	80.1	82.2
425 μm		62.6	4.3	6.9	61.7	63.6
180 μm		55.0	4.2	7.6	53.9	55.7
75 μm		35.0	3.8	10.9	34.9	36.2
Field moisture content (%)		10.1	1.3	12.8	9.9	10.4
Optimum moisture content (%)		11.1	0.48	4.4	11.0	11.2

Note: 1 mm = 0.039 in.

Table 8. Lake Wales SAHM data.

Test	Section Number	Design: 50 Percent Local Sand	Design: 50 Percent Crushed Stone Screenings	Field Data: Mean	Field Data: SD	Field Data: Coefficient of Variance (%)	Field Data: 95 Percent Confidence Limits: Lower	Field Data: 95 Percent Confidence Limits: Upper
Extraction, 98 tests (% passing)	6A and 6B through 10A and 10B							
12.5 mm			100	100	0.1	0.1	100.0	100.0
9.5 mm			100	100	0.1	0.1	100.0	100.0
4.75 mm		100	98	99	0.7	0.7	99.1	99.3
2.0 mm		100	88	89	4.5	5.1	88.1	90.0
425 μm		79	37	54	6.8	12.5	53.1	55.8
180 μm		13	17	10	1.7	16.1	10.0	10.7
75 μm		2	11	3	1.9	56.7	3.0	3.7
Bitumen (%)		7.7		7.7	0.5	6.1	7.6	7.8

Note: 1 mm = 0.039 in.

where

$\overline{SV}$ = slope variance, the average of four passes of the chloe profilometer in the outside wheelpath (or by Mays meter and developed correlation).
texture = texture of the surface of the road as measured in centimeters by the Texas Text-Ur-Meter [every 30.6 m (100 ft) with a minimum of five readings].
rutting = rutting measured in centimeters by rut depth gauge [every 30.6 m (100 ft) with a minimum of five readings].
C + P = cracking and patching measured in square meters per 92.8 m^2 of the pavement surface.

A summary of data collected up to 1978 for the

Figure 3. Marianna deflection comparison (westbound traffic lane).

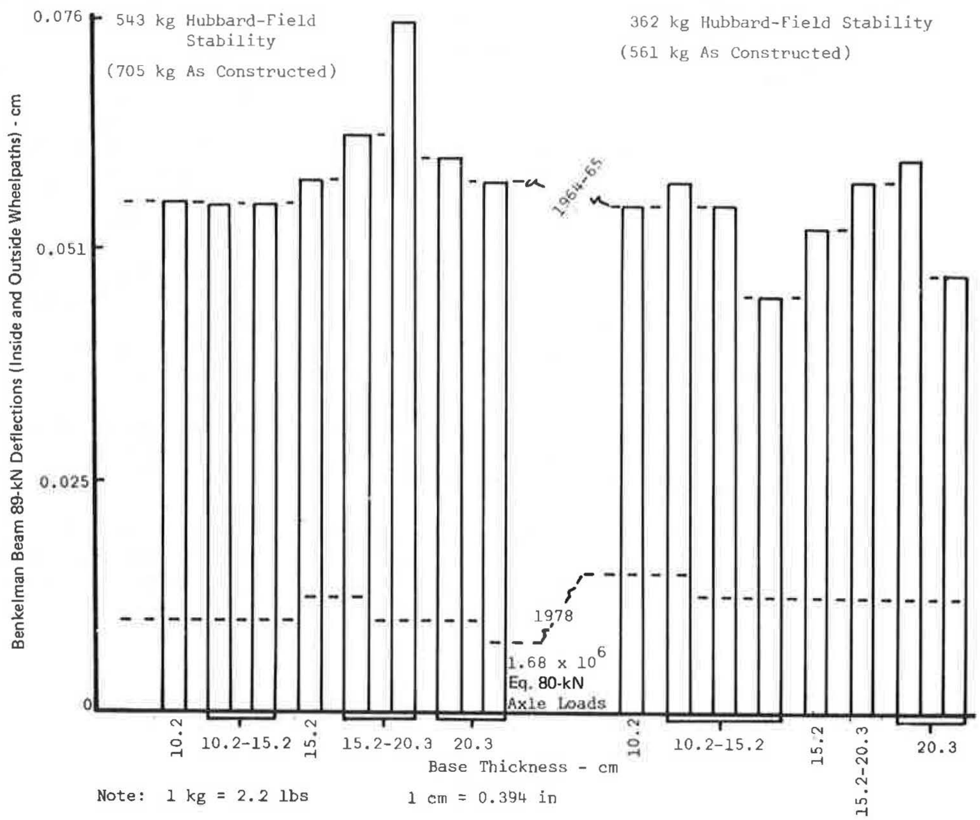
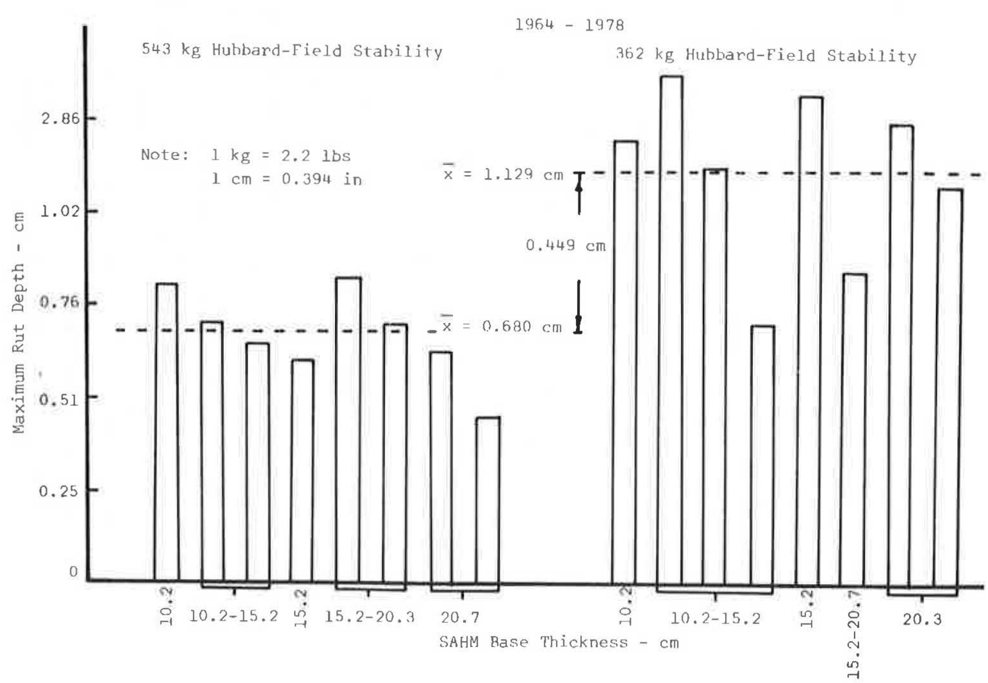


Figure 4. Marianna rut depth comparison (westbound traffic lane).



Marianna, Palm Beach, and Lake Wales test roads has been previously published by the Florida Department of Transportation in a condensed report (5).

Marianna Test Road

The Benkelman beam deflection data presented in Figure 3 illustrate that deflections, taken immediately after construction, for the 543-kg (1200-lb) Hubbard-Field stability SAHM base varied between 0.056 and 0.076 cm (0.022 and 0.030 in) and slightly exceeded the deflection for the 362-kg (800-lb) stability SAHM base. This variation is not excessive and, from a practical viewpoint, the SAHM stability and deflections can be considered to be uniform between most test sections. Deflection measurements in 1978 indicate greater uniformity among all test sections. However, the high-stability SAHM does appear to provide slightly lower deflections than the lower-stability SAHM—0.010 cm (0.004 in) as compared to 0.013 cm (0.005 in). In general, the deflection response of all test sections is similar and is indicative of the uniformity and good bearing values obtained by the subgrade (LBR 77).

The effect of stability on rut depth is significant. It is illustrated in Figure 4. On the average, rut depths increased 0.46 cm (0.18 in), from 0.69 to 1.14 cm (0.27-0.45 in), from high-stability to lower-stability test sections. This degree of rutting had little effect on the reduction in PSI as shown in Figure 5. Calculations indicate that slope variance and texture contributed substantially to reduce PSI values from the 4.5 range after construction to below 3.0 in 1978. High-stability SAHM base pavements were primarily affected by surface texture (up to 0.50 reduction in PSI) with a small contribution from slope variance. The lower PSI values for low-stability SAHM pavements are attributed directly to the lower-stability base. A detailed analysis of serviceability data was not possible because of resurfacing at two different times prior to 1978.

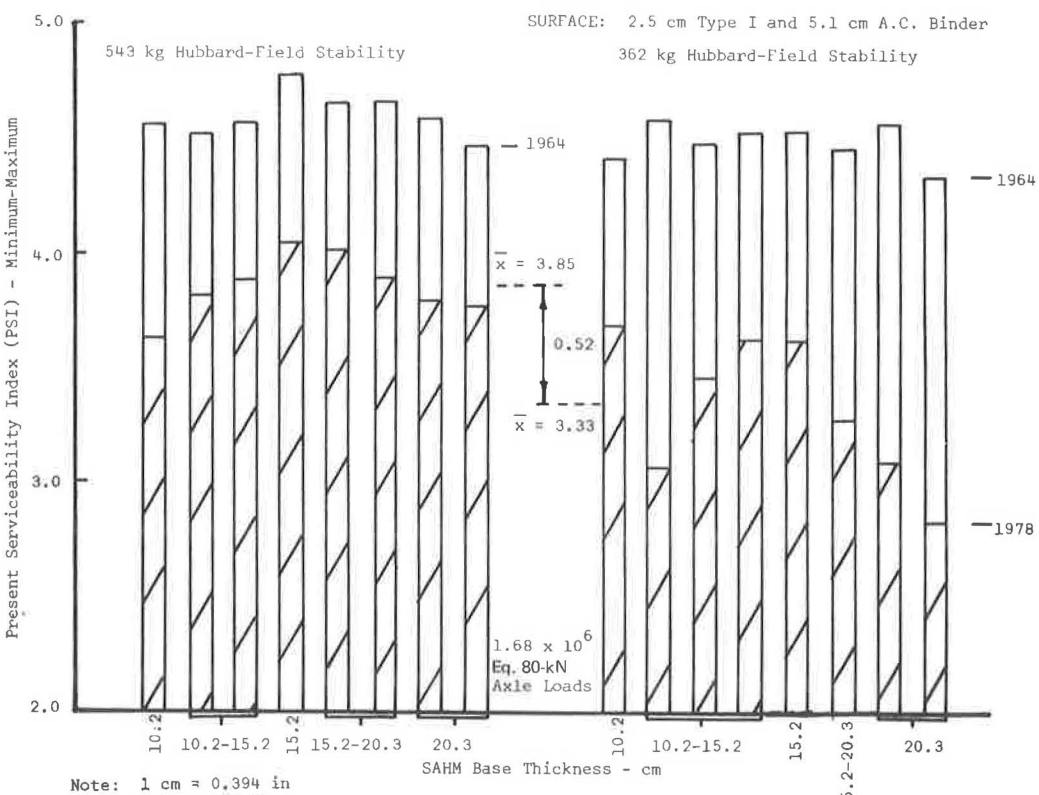
Palm Beach Test Road

Some cracking has been observed in the eastbound lane of all test sections, but no cracking has been detected in the westbound lane after eight years of service. This condition exists because of the predominance of loaded trucks that travel eastbound. The results of pavement condition surveys provided only a subtle indicator of differences among the test sections. Higher traffic volumes would have probably accentuated these differences in performance.

Figure 6 illustrates that the limerock base sections produced lower deflections after construction than did either the SAHM base or shell base test sections. However, data collected up to 1978 indicated that deflections in both SAHM and limerock bases were essentially the same. The shell bases gave deflections that were about 0.005 cm (0.002 in) greater.

The reduction in pavement deflections with age may be partially due to densification, cementation of limerock materials, and asphalt hardening in both 3.8-cm (1.5-in) type 2 surface course and the SAHM. Abson recovered asphalts gave 60°C (140°F) viscosities, ranging from 2.5 to 5.0 E3 Pa•s. This constitutes an approximate tenfold increase in viscosity. Flexural fatigue tests at 25°C (77°F) on beams cut from the low-stability and high-stability SAHM pavement sections gave average stiffness values of 510 and 772 MPa

Figure 5. Marianna PSI comparison (westbound traffic lane).



Note: 1 cm = 0.394 in
1 kg - 2.2 lb

Figure 6. Palm Beach deflection comparison (eastbound and westbound lanes).

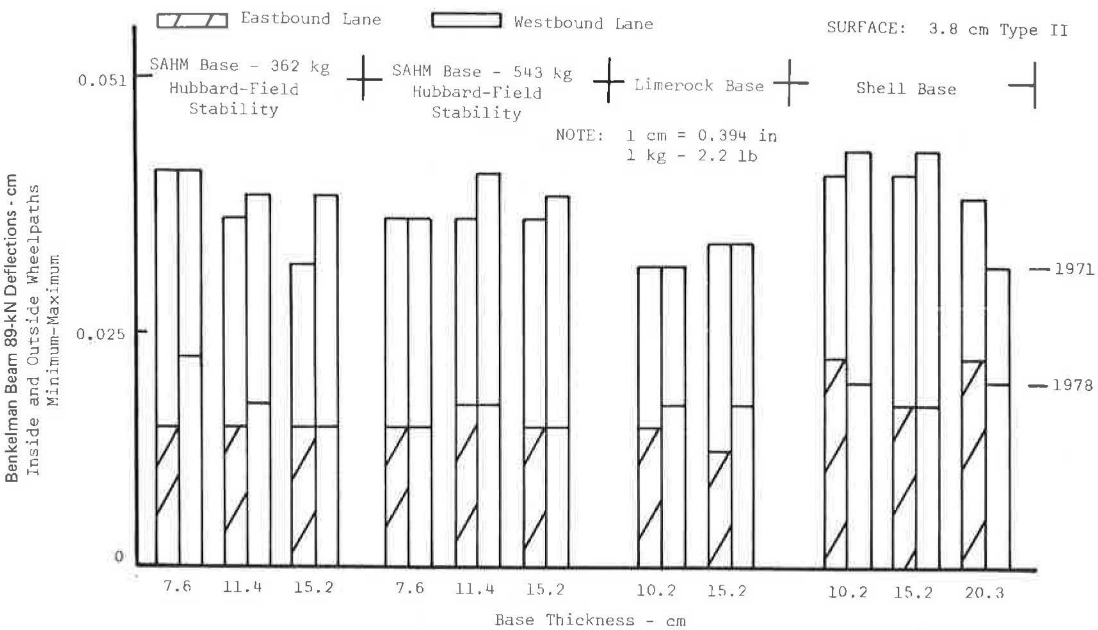
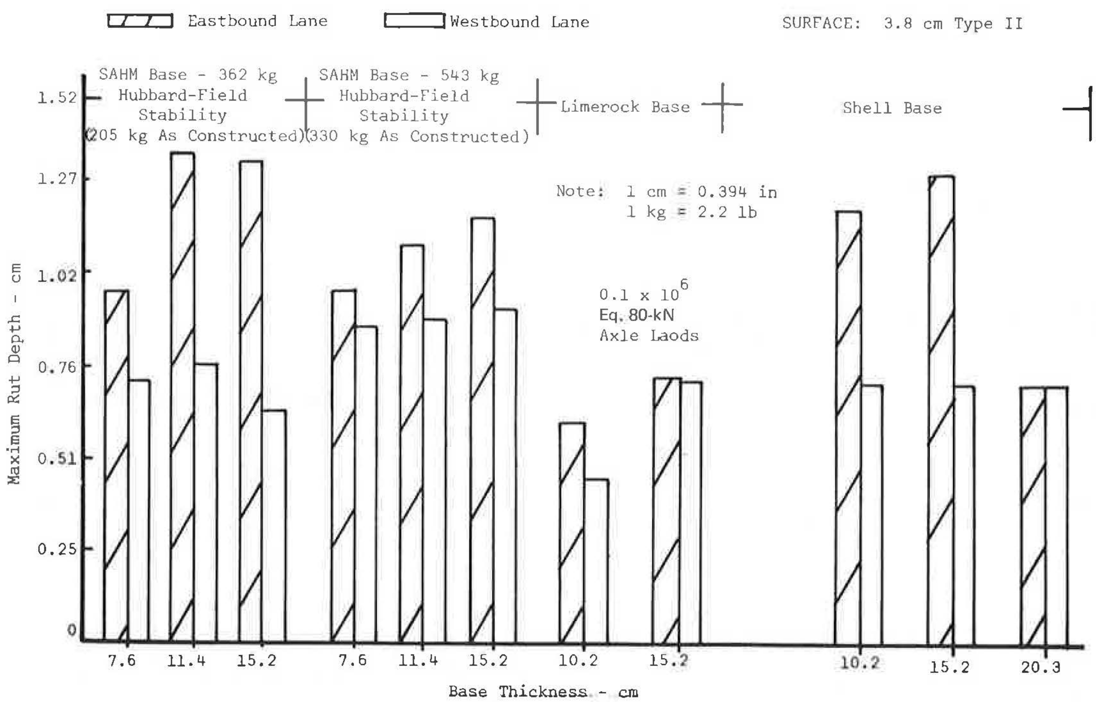


Figure 7. Palm Beach rut depth comparison (eastbound and westbound lanes).



(74 000 and 112 000 lbf/in^2), respectively. This increase in stiffness was not detected by the Benkelman beam tests.

Pavement rutting was most pronounced in the SAHM and shell sections. Figure 7 illustrates the differences in rut depth between test sections. Limerock bases had less rutting [0.46-0.74 cm (0.18-0.29 in)] than SAHM and shell bases in the eastbound lane. The greater degree of rutting is not related to densification of the type 2 surface or the SAHM. Rut depth due to combined surface and SAHM densification was determined to vary from 0.13 to 0.28 cm (0.05 to 0.11 in). Assuming that these rut depth calculations are correct, it seems probable that the majority of rutting has developed from lateral or vertical displacement of the asphalt pavement in the vicinity of the wheelpaths.

The serviceability of the pavement sections had not changed drastically over eight years. Figure 8 shows that all but two sections with SAHM base have maintained a PSI in excess of 4.0. This is attributed to the improved riding qualities attained from construction that uses the SAHM. Other than rutting, there has not

Figure 8. Palm Beach PSI comparison (eastbound and westbound lanes).

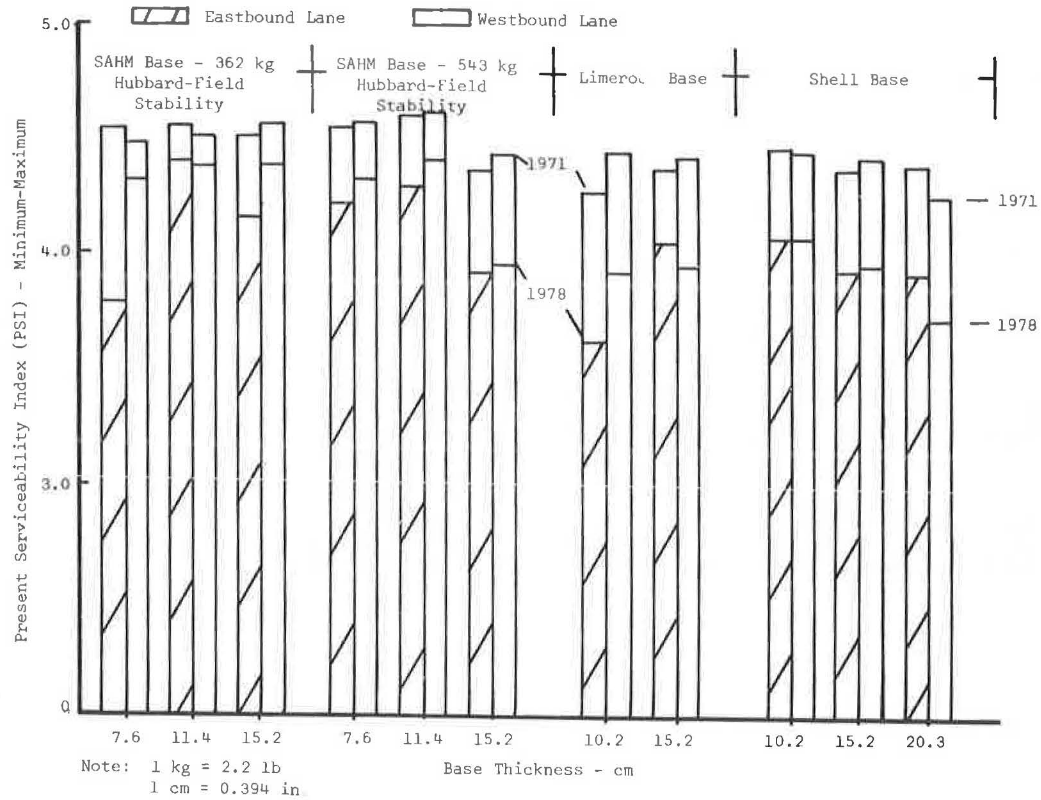
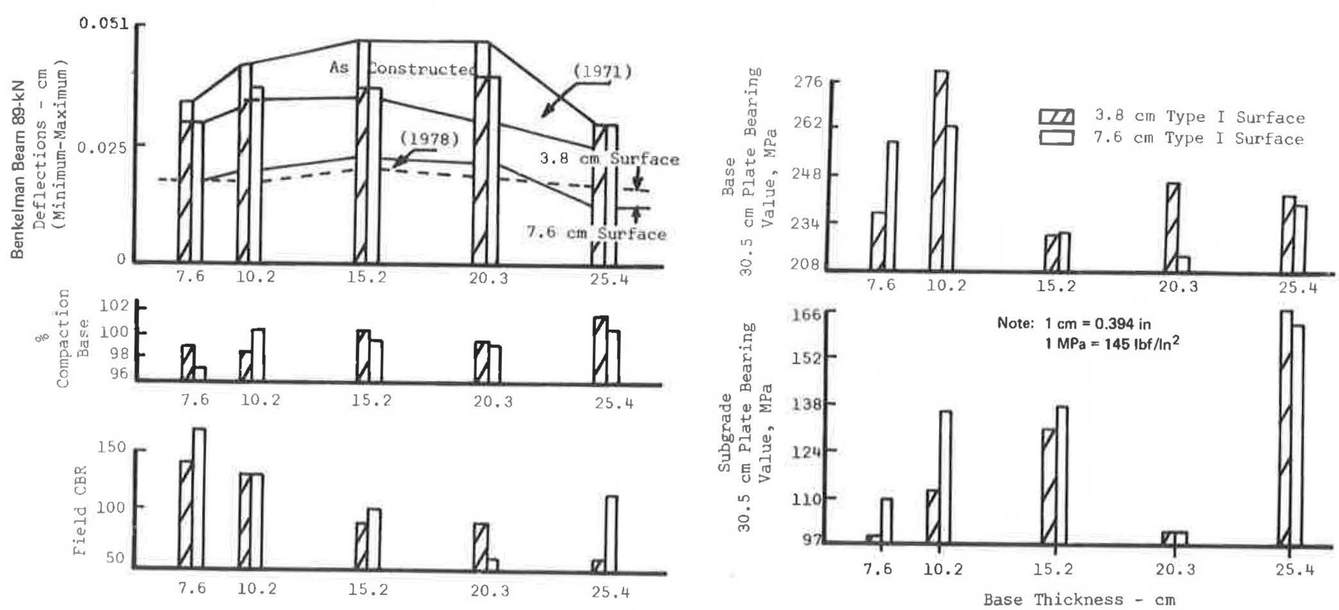


Figure 9. Lake Wales field data—limerock base.



been any indication that differences in performance between the test sections will be observed in the future.

Lake Wales Test Road

The Lake Wales test road has provided detailed performance comparisons between variations in surface thickness, base thickness, and base materials. Variations in test values for the constructed limerock base sections are illustrated in Figure 9. Subgrade bearing values for the 25.4-cm (10-in) limerock sections were greater than for the other sections, which is reflected in the low Benkelman beam deflections. Although the limerock base was compacted to 98 percent or more of AASHTO T180, the slight differences appear to correspond with the subgrade bearing values. However, field California bearing ratio (CBR) and plate bearing values for the limerock base do not seem to correlate with the percentage compaction or subgrade bearing values. Both subgrade bearing values and Benkelman

Figure 10. Lake Wales rut depth data—limerock base.

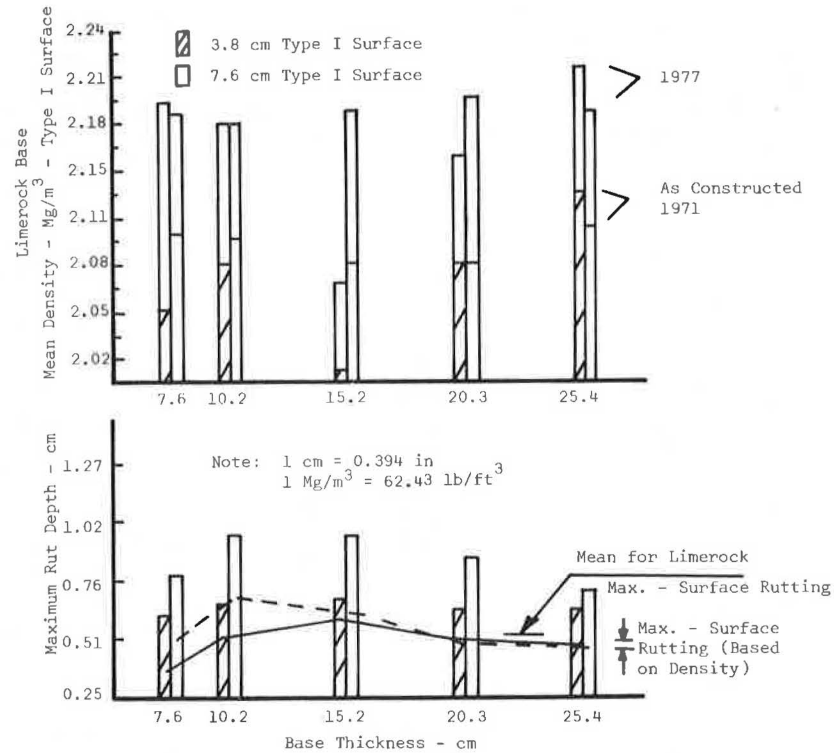
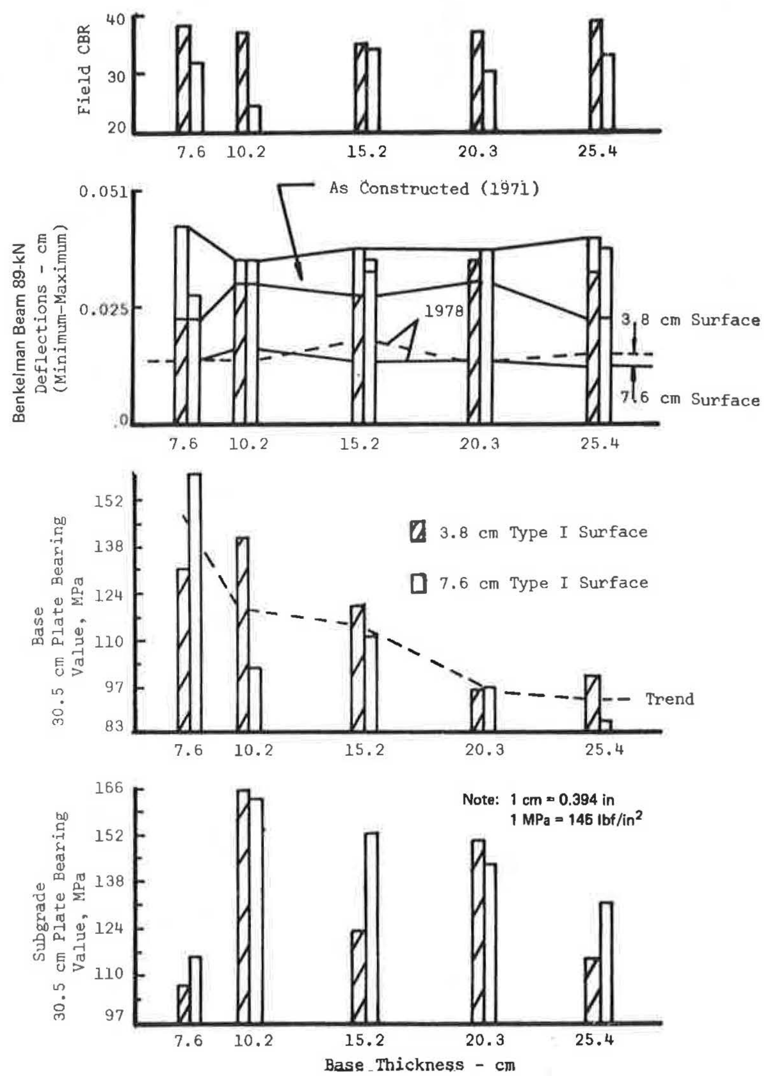


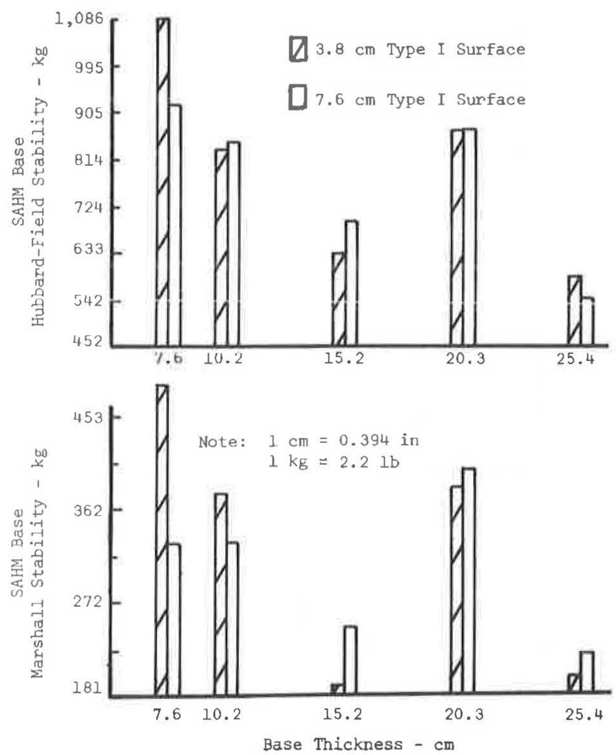
Figure 11. Lake Wales field data—SAHM base.



beam deflections for the limerock sections are similar to those for the Palm Beach test road.

Differences in rut depth for 3.8-cm (1.5-in) and 7.6-cm (3-in) surfaces over limerock bases are shown in Figure 10. Compacted density of the surface course, base course, and relative layer thickness are major

Figure 12. Lake Wales SAHM stability.



pavement variables in the amount of rut depth. The 7.6-cm surface produced slightly greater rutting.

Data for the SAHM base sections are presented in Figures 11 and 12. Benkelman beam deflections are fairly consistent for all sections, even though some variation is evident in plate bearing values. Hubbard-Field and Marshall stability values reflect similar trends with different SAHM base thicknesses. The trend line (illustrated in Figure 11) shows a reduction in plate bearing values for increasing SAHM base thickness. This trend is a result of the increased compressibility and creep flow of the SAHM when subjected to the slowly applied loading conditions of the plate bearing test.

The rut depth and density of the surface course are shown in Figure 13. The rut depth is about 1.12 cm (0.44 in) and 1.40 cm (0.55 in) for the SAHM sections with 3.8-cm (1.5-in) and 7.6-cm (3-in) surfaces, respectively, except for the 7.6-cm (3-in) SAHM base. The results of computations based on densifications of the pavement layer are given in Table 9 and illustrated by trend lines in Figures 10 and 13. In general, about 20-25 percent of the rutting occurs in the 3.8-cm surface with the remaining 75-80 percent in the limerock or SAHM base courses. The 7.6-cm surface contributes 35 percent of the total rutting in all pavement test sections.

The PSI comparison in Figure 14 demonstrates that both limerock and SAHM bases provided equally good performance after 2.5 million equivalent 80-kN (18-kip) axle load applications. Again, the slightly lower PSI values for the limerock base sections were attributed to construction methods and the ability to obtain a smaller slope variance on pavement sections with SAHM bases.

SUMMARY AND CONCLUSIONS

High-quality aggregates are not readily available in

Figure 13. Lake Wales rut depth data—SAHM base.

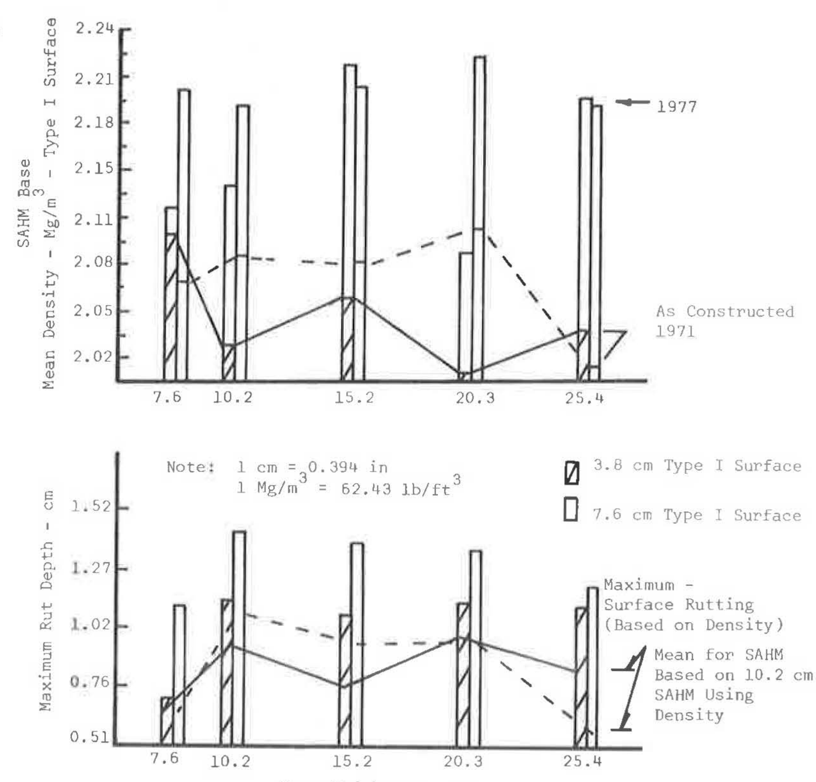
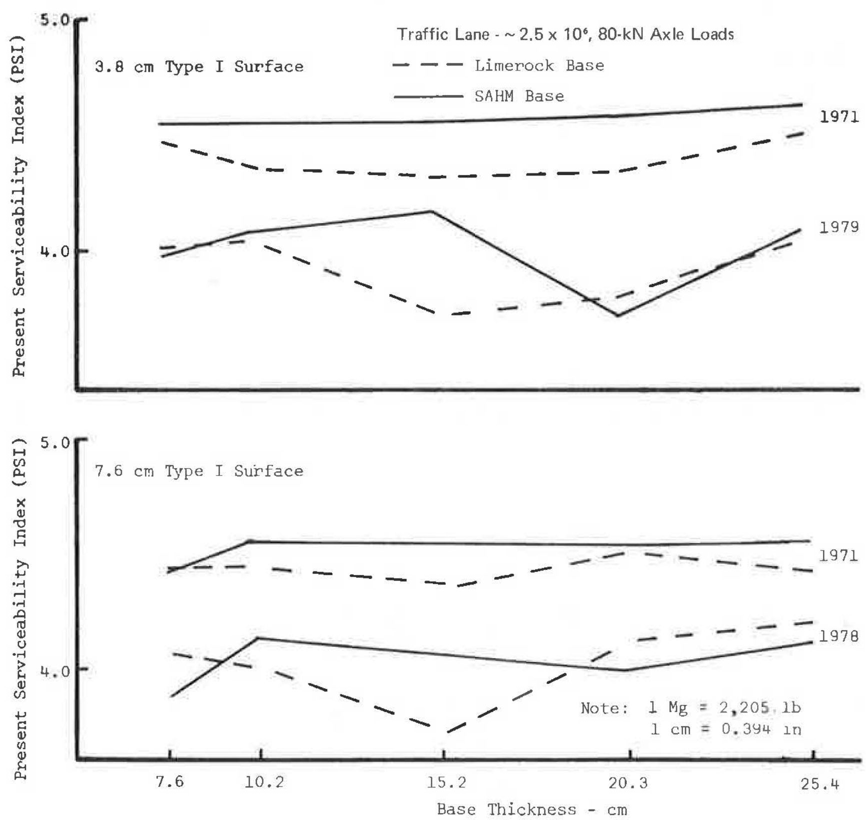


Table 9. Rut depth calculation results.

Base Type	Base Thickness (cm)	Rut Depth (cm): 3.8-cm Type 1 Surface[a]	Residual Due to Rutting of Base[b]	7.6-cm Type 1 Surface[a]	Residual Due to Rutting of Base[b]
Limerock	7.6	0.25	0.35	0.28	0.51
	10.2	0.15	0.51	0.28	0.69
	15.2	0.10	0.58	0.36	0.61
	20.3	0.13	0.51	0.38	0.48
	25.4	0.15	0.48	0.25	0.46
	Mean	0.15	0.48	0.31	0.56
SAHM	7.6	0.05	0.66	0.46	0.66
	10.2	0.20	0.94	0.36	1.09
	15.2	0.33	0.76	0.43	0.96
	20.3	0.15	0.98	0.41	0.96
	25.4	0.28	0.84	0.63	0.58
	Mean	0.20	0.84	0.46	0.86

Note: 1 cm = 0.394 in.
[a]Calculated on basis of density increase to 1977.
[b]Measured maximum rut depth minus surface rut depth.

Figure 14. Lake Wales PSI comparison.



Florida for use in highway pavement construction. However, materials of dubious quality (such as limerock and local sands) can be used successfully when proper design and construction methods are applied. The information presented on these test roads in Florida illustrates that good performance can be achieved with either limerock or SAHM bases. However, it is important to recognize that well-drained soils, adequate surface drainage, and generally good subgrade strengths contribute immeasurably toward long-term serviceability of flexible pavements.

Several important aspects of performance of pavements constructed with limerock or SAHM bases should not be overlooked.

Limerock bases are extremely susceptible to water. Care must be taken to provide adequate drainage and to prevent penetration of water through cracks in the pavement. The long-term strength (stiffness) gain in pavements constructed with limerock bases is attributed to cementing action between aggregate particles. This is obviously beneficial to the structural behavior of the pavements.

The typical concept of using only local sands for SAHM will often cause problems with respect to pavement construction and performance. The blending of limerock screenings with local sands can produce a SAHM that approaches the gradation and quality of a type 2 asphalt surface mixture. Often, 50 percent or more screenings are blended with local sands for this purpose.

Pavement deflections as measured by the Benkelman beam are usually about 0.051 cm (0.020 in) for post-construction and decrease to about 0.025 cm (0.010 in) within a few years. Excessive age hardening of high-viscosity paving asphalts often results in low deflections prior to cracking, with a large increase in deflection after cracking. Indirect tensile tests indicate that the SAHM will provide a greater tolerance to strains before fracture than will a conventional AC mixture.

Pavement rutting is primarily related to traffic densification of surface and base courses. It has been shown that rutting of limerock base courses is less than for SAHM bases. Sixty-five to 80 percent of the rutting occurs in base-course materials. Subgrade conditions and adequacy of compaction can affect the degree of rutting. The stability selected for design of a SAHM and that achieved in construction may influence rutting. A lower-stability SAHM on the Marianna test road indicated an average 65 percent increase in rut depth over that with high-stability bases.

Pavement serviceability is comparable for pavements constructed with either limerock or SAHM bases. Surface cracking on the Lake Wales test road was more prevalent with the thinner surface. However, numerous observations indicate that cracking can be more severe in pavements that have thicker surfaces. Much of this confusion is related to different hardnesses or viscosities of the binder and the ambient temperature for that locale.

REFERENCES

1. H. F. Godwin and R. L. McNamara. Summary Data Reports for Study P-1-63, Flexible Pavement Design. Office of Materials and Research, Florida Department of Transportation, Gainesville, Res. Rept. 196, 1976.
2. H. F. Godwin and C. F. Potts. Experimental Flexible Pavement, Post-Construction and Materials Report—Strength Equivalency Study of Various Base Types (Palm Beach County). Office of Materials and Research, Florida Department of Transportation, Gainesville, Res. Rept. 164, 1972.
3. B. E. Ruth and J. D. Maxfield. Fatigue of Asphalt Concrete. Engineering and Industrial Experiment Station, Univ. of Florida, Gainesville, Final Rept., Project D-54, 1977.
4. B. E. Ruth and A. S. Davis. Fatigue and Fracture of Asphalt Concrete. Engineering and Industrial Experiment Station, Univ. of Florida, Gainesville, Final Rept., Project D-82, 1978.
5. R. L. McNamara. Final Summary Data Reports for Study P-1-63, Flexible Pavement Design (Chiefland, Crestview, Lake Wales, Palm Beach, Marianna). Office of Materials and Research, Florida Department of Transportation, Gainesville, Res. Rept. 196-A, 1979.

Cement Stabilization of Degrading Aggregates

Ira J. Huddleston, Ted S. Vinson, and R. G. Hicks

The suitability of cement as a stabilizing agent for degrading aggregates has been evaluated. Specifically, wet-dry, freeze-thaw, and unconfined compressive strength tests were performed on marine basalt and three gradations of Tyee sandstone from the Siuslaw National Forest, two types of decomposed granite from the Umpqua National Forest, and a moderately weathered granite from the Colville National Forest. Tests were performed on samples at both standard and modified compactive efforts. All of the materials tested had satisfactory durability at relatively low cement contents. Variations in optimum moisture and maximum dry density associated with a change in cement content of 2 percent were insignificant. Samples compacted with a modified compactive effort generally required 1-2 percent less cement to meet durability requirements. Overall, the durability was found to be influenced more by cement content than by compactive effort. The unconfined compressive strength varied with material, cement content, and compactive effort. For a given cement content, the strengths were higher for the specimens compacted with a modified effort than for those compacted with a standard effort. The strengths increased with age for all materials and compactive efforts except the marine basalt and Calahan decomposed granite compacted at the standard effort.

The increasing demand for access to national forests for timber harvesting, recreational, and other purposes has focused attention on the need to provide roads to serve relatively low traffic volumes. These roads are generally constructed and maintained on limited budgets, but they must provide adequate service for many years. Consequently, it is desirable to use high-quality materials in the construction of these roads (i.e., materials that will provide strength and durability to withstand the anticipated environmental and traffic loads).

When high-quality materials are not locally available, three alternatives exist: (a) high-quality materials may be imported, (b) poor-quality materials may be used by lowering design standards, or (c) poor-quality materials may be improved. With respect to the third alternative, the characteristics of the locally available materials may be improved by the addition of stabilizing agents such as lime, cement, or asphalt. The stabilized materials may be used in place of the transported materials.

At the current time, in areas of several Pacific Northwest national forests, quality road-building materials are not available locally and must be transported considerable distances. However, aggregates that do not meet the degradation specifications of the U.S. Forest Service in Region 6 (herein termed degrading aggregates) are locally available in quantity. In recognition of this situation, a study was initiated to investigate the feasibility of using cement as a stabilizing agent for degrading aggregates (1).

MATERIAL CHARACTERISTICS

Five types of degrading aggregates from three national forests were selected for the study. Tyee sandstone and crushed marine basalt were obtained from the Siuslaw National Forest; two distinct types of decomposed granite (termed Goolaway and Calahan, respectively) were obtained from the Umpqua National Forest; poor-quality granite was obtained from the Colville National Forest.

Material processing and aggregate evaluation tests were performed in order to (a) prepare the materials for soil-cement testing and (b) determine their suitability as a roadway material. The material processing operations varied with the material type. For the Tyee sandstone and the Colville granite, processing included crushing, sieving, washing the coarse fraction [retained on a 4.75-mm (No. 4) sieve] and storing for further testing. The remaining materials (Calahan and Goolaway decomposed granites and the marine basalt) did not require crushing but were sieved, washed, and stored as above. Crushing was done with a Braun Chipmunk Crusher, and the materials were sieved and stored in the following sizes: 19 mm (0.75 in), 13 mm (0.5 in), 10 mm (0.375 in), 6 mm (0.25 in), 4.75 mm, and passing 4.75 mm.

Standard tests used by the U.S. Forest Service to determine aggregate quality and characteristics were conducted. The tests included sieve analysis (AASHTO T11/T27), specific gravity and percentage absorption (ASTM D854), Los Angeles (L.A.) abrasion (AASHTO T96), California durability (AASHTO T210), and sand equivalent (AASHTO T176). Results of the aggregate tests are presented in Tables 1 and 2. Also presented in Table 2 are specifications for Pacific Northwest Forest Service base course aggregates.

The data presented in Tables 1 and 2 identify a relatively wide range of material types and qualities. The completely decomposed Goolaway granite contains no material sizes greater than the 4.75-mm sieve. The Calahan decomposed granite has experienced a lower degree of weathering and chemical alteration than the Goolaway granite and contains sizes greater than the 4.75-mm sieve. The Colville granite represents a borderline degrading aggregate since it fails to meet the L. A. abrasion specifications only. The sandstone is felt to be representative of the quality of sandstones found throughout the coast range of Oregon and Washington. The sandstone is lacking in its resistance to abrasion and also in resistance to breakdown in a wet environment (indicated by the California durability test results). Finally, the marine basalt test results are probably typical of the marine basalts found in the coast range. They show good resistance to abrasion and a small amount of detrimental fines but are quite susceptible to breakdown in a wet environment.

Table 1. Grain size distribution of bedrock materials.

Sieve Size	Percentage Passing: Tyee Sandstone[a]	Marine Basalt[b]	Decomposed Granite[b]: Goolaway	Calahan	Colville Granite[b]
38 mm	88	100	100	100	100
25 mm	78		100		
19 mm	73	86	100	96	93
13 mm	61	70	100	85	64
10 mm	49	60	100	77	54
6 mm	37	47	100	69	40
4.75 mm	30	35	100	60	33
2.0 mm	24	25	83	46	23
425 μm	16	11	44	23	11
75 μm	6	6	17	9	2

Note: 1 mm = 0.04 in.
[a]After crushing with Braun Chipmunk Crusher.
[b]As received, no crushing.

TEST PROGRAM

Material Processing

The Goolaway and the Calahan decomposed granites required no crushing. Their gradations, as given in Table 1, are the result of breakdown during excavation (in situ) and transport and handling. Because of the nature of these materials, control of their gradation is very difficult, not only in the field but also in the laboratory. For the purposes of the study it was assumed that changes in the gradations of these materials due to handling, mixing, and compaction would be similar in the laboratory and field operation.

The Tyee sandstone, marine basalt, and Colville granite all required crushing to obtain material sizes smaller than 25 mm (1 in). The gradations given in Table 1 for these materials are those that resulted directly from this crushing operation and do not necessarily represent an optimum or practical gradation for stabilization with cement. For the purposes of the study, the gradations were altered so that they represent, as nearly as can be determined, an optimum gradation.

The rationale used for determining optimum gradations for these materials was based on work done by the Portland Cement Association (PCA) (2, 3). Dense-graded materials (AASHTO A-1-a) require the least amount of cement to meet durability requirements. Note that the amount of cement per unit volume of material stabilized is the product of the cement content by weight and the field dry density. For the range of dry densities and cement contents associated with this study their product is accurately reflected by the cement content alone (i.e., minimum cement contents reflect minimum amounts of cement used in the field). In addition, work by Norling and Packard (3) has shown that the addition of material that is retained on the 4.75-mm sieve to material passing the 4.75-mm sieve decreases the total cement requirement as long as the amount retained on the 4.75-mm sieve does not exceed 50 percent. Norling and Packard combined this finding with a relationship they developed that relates the maximum standard Proctor dry density and the percentage of the material

Table 2. Aggregate evaluation test results.

Test	Tyee Sandstone	Marine Basalt	Decomposed Granite: Goolaway	Calahan	Colville Granite	Forest Service Aggregate Specs
Specific gravity (SSD)	2.32	2.55	2.64	2.56	2.61	
Absorption (%)	8.8	4.3		2.1	0.85	
L. A. abrasion	67.5	34.4		64.4	55	40 maximum
California durability[a]	9	19		60	85	35 minimum
Sand equivalent	48	46		44	82	35 minimum

[a]The results given are for the coarse or the fine fraction depending on which fraction controlled.

Figure 1. Indicated cement contents of soil-cement mixtures containing material retained on the 4.75-mm sieve.

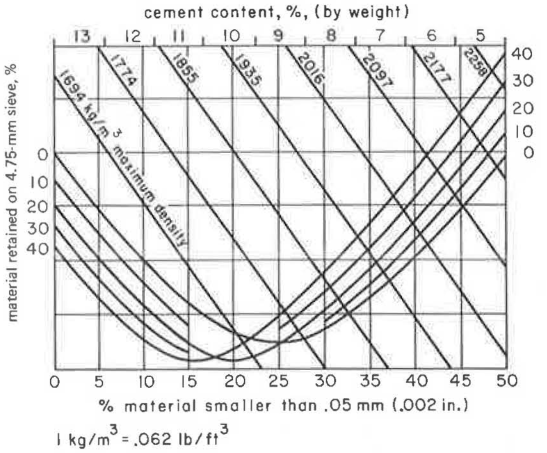


Table 3. Grain size distributions of materials used in test program.

Sieve Size	Percentage Passing: Tyee Sandstone A	B	C	Marine Basalt	Decomposed Granite: Goolaway	Calahan	Colville Granite
25 mm	100	100	100	100	100	100	100
19 mm	92	100	100	100	100	96	100
13 mm	80	93	92	100	100	85	80
10 mm	73	89	87	86	100	77	70
6 mm	65	81	81	67	100	69	58
4.75 mm	60	75	75	55	100	60	50
2.0 mm	48	60	58	40	83	46	36
425 μm	35	39	31	18	44	23	16
75 μm	19	15	6	9	17	9	4

Note: 1 mm = 0.04 in.

smaller than 0.05 mm (0.002 in) to the cement requirement for soils that contain no material retained on the 4.75-mm sieve. The results they obtained are shown in Figure 1. The relationship in Figure 1 indicates that optimum gradations (associated with minimum cement contents) may result from ratios of percentage of material retained on the 4.75-mm sieve to percentage of material smaller than 0.05 mm (0.002 in) of either 40:15 or 20:20.

The Tyee sandstone was blended to a 40:15 ratio, for two reasons:

1. This ratio was the closest to the original gradation and
2. In all likelihood, it represents the gradation most easily obtained under field conditions.

To determine the influence of variations in this gradation on the cement requirement, two additional gradations were selected. These two additional gradations define a range of gradations about the optimum that has approximately the same cement requirement as the 40:15 ratio material.

The gradations for the marine basalt and Colville granite were not obtained from Figure 1. The marine basalt came from stockpiled materials at a commercial rock quarry. It was processed through the rock crusher at the facility. Therefore, it represents a gradation that would be used in the field. Following recommended PCA test procedures, however, it was necessary to screen out material retained on the 13-mm (0.5-in) sieve so that there would not be more than 50 percent retained on the 4.75-mm sieve. The final gradation for the Colville granite was obtained by screening out the sizes retained on the 19-mm (0.75-in) sieve to get a maximum of 50 percent retained on the 4.75-mm sieve. The final adjusted gradations for each material are given in Table 3. The three gradations used for the Tyee sandstone are labeled A, B, and C, where A refers to the optimum 40:15 gradation, and B and C refer to 25:11 and 25:5 gradations, respectively.

Compactive Effort

Standard Proctor (AASHTO T 99-74) compactive effort is used when conducting wet-dry (W/D) and freeze-thaw (F/T) durability tests. Modified Proctor (AASHTO T 180-74) compaction is not normally recommended for durability testing for two reasons:

1. Maximum densities are obtained by using the modified compactive effort at significantly lower water contents than those of the standard compactive effort; this lower water content is not thought to be high enough for proper cement hydration; and
2. The high densities obtained with the modified compactive effort in the laboratory are difficult to obtain in the field in many applications.

However, results of compressive strength tests conducted by the U.S. Forest Service, Region 6 Materials Laboratory, have shown that, for the same cement content, the strengths of specimens compacted with a modified compactive effort are as much as two times greater than those of specimens compacted with a standard compactive effort. These data raise the question of whether or not it would be beneficial and feasible to design for the higher densities and strengths obtained with the modified compactive effort. In consideration of this question, all tests were conducted at both standard and modified Proctor compactive efforts.

Cement Contents

Cement contents for the materials to be tested were determined by following the procedures developed by PCA (2). The PCA method uses gradation and maximum standard Proctor dry density to estimate the required cement content. Tests are performed on samples at cement contents at an estimated optimum value and 2 percent above (high cement content) and below (low cement content) this value. Seven-day cure, unconfined compressive strength (q_u) tests were conducted at the high and low cement contents. Twenty-eight-day-cure q_u tests were conducted at the lowest cement content that satisfied all durability test requirements.

The material mixing process included three stages:

1. The cement was mixed with the material passing the 4.75-mm (No. 4) sieve,
2. The correct amount of water was mixed with the material obtained in step 1, and
3. The material sizes retained on the 4.75-mm sieve were mixed in a saturated surface dry (SSD) condition with the mixture obtained in step 2.

The materials were thoroughly mixed with a trowel at each stage until they were uniform in distribution.

Initially moisture-density tests were conducted at each of the three cement contents for the Goolaway decomposed granite and the A-gradation of the Tyee sandstone. Results from these tests indicated that the variations in optimum moisture and maximum dry density associated with a change in cement content of ± 2 percent were insignificant. Moisture-density tests

Table 4. Summary of durability tests at minimum required cement content.

	Standard Compactive Effort							Modified Compactive Effort						
	Tyee Sandstone			Marine Basalt	Decomposed Granite		Colville Granite	Tyee Sandstone			Marine Basalt	Decomposed Granite		Colville Granite
Test	A	B	C		Goolaway	Calahan		A	B	C		Goolaway	Calahan	
Cement requirement[a] (% by weight)	5	5	5	6	6	5	5	4	4	4	6	4	3	3
Controlling test	F/T	F/T	F/T	W/D	F/T	W/D	W/D	F/T	F/T	F/T	W/D	F/T	W/D	W/D
Moisture content (% by weight)	14.2	14.3	15.6	13.9	13.2	10.4	7.6	11.9	12.2	13.5	12.9	11.1	6.6	6.2
Dry density (kg/m^3)	1846	1817	1793	1836	1846	2058	2115	1966	1905	1875	1952	1913	2187	2163
Soil-cement weight loss (% by weight)	7.2	6.6	9.1	13.4	4.9	12.8	5.7	8.1	3.6	8.7	12.2	10.8	9.3	8.7

Note: 1 kg/m^3 = 0.062 lb/ft^3.

[a]Minimum cement content required to pass durability test.

for the remaining materials were therefore carried out at the intermediate cement contents only.

Durability Testing

The durability testing included standard F/T (AASHTO T 136-76) and W/D (AASHTO T 135-76) tests. Duplicate samples at each cement content were compacted for F/T testing. One sample was used for brushing to determine soil-cement weight loss. The other sample was used to measure moisture and volume changes. Duplicate samples at the intermediate cement content were compacted for W/D testing, one sample each for determining weight loss and moisture and volume changes.

Strength Testing

The unconfined compressive strength of the stabilized material was measured after 7- and 28-day moist curing. On the 7th (or 28th) day of curing the samples were placed in a water bath to soak for four hours. After soaking, the samples were surface dried and the ends were capped with a sulfur-based capping compound. Unconfined compressive strength tests were conducted at a load rate of 138 kPa (20 lbf/in^2) per second. Load and deformation were continuously monitored with a load cell and linear variable differential transformer. The results represent the average of two tests for the 7-day cure specimens and one test for the 28-day cure specimens.

DISCUSSION OF TEST RESULTS

Durability

Table 4 presents a summary of the results at the minimum cement content for each material considered. Moisture and volume change results are not included because no appreciable changes could be detected during the durability testing for any of the materials tested.

Of the seven materials tested, only the marine basalt at the standard compactive effort exceeded the 14 percent weight-loss criteria established by PCA (2). The lack of failures for the remaining materials has two probable explanations.

One explanation is that the procedure used to select the cement contents for testing is an approximate one and cannot guarantee that, in each case, the minimum cement content will be determined. A possibility of error in the procedure is present when applied to the degrading aggregates that were used in this study, because of the use of maximum dry densities instead of void ratios to determine the average cement requirement. The cement requirement is a function more of particle packing, which is best described by the void ratio, than of density, which also varies with the specific gravity of the particles. The degrading aggregates used in this study typically have low specific gravities and, therefore, do not compact to high densities even though they may have relatively low void ratios.

The other explanation is that the W/D tests were conducted only at the intermediate cement content as suggested in the PCA procedure (2) because the F/T tests normally control for materials that have low silt and clay contents. However, the W/D test appeared, in some cases, to control for the materials in this study. It is believed that additional failures would have been experienced had W/D tests been conducted on the materials at the lower cement contents.

Soil-cement weight loss was generally less for the samples compacted with a modified compactive effort than for those compacted with a standard compactive effort. This was true for the sandstones, basalt, two of four cement contents of the Calahan decomposed granite, and three of four cement contents of the Colville granite. The Goolaway decomposed granite and the remaining cement contents for the Calahan decomposed granite and the Colville granite all had higher weight loss for the samples compacted with a modified effort. The differences between the values for the two compactive efforts were less than 1 percent for all cases in which the modified value exceeded the standard and varied from 1 to 7 percent for the opposite case. The minimum required cement content determined for each material was less with a modified compactive effort for all the material tested except the Goolaway decomposed granite and the marine basalt. It is likely that, had the sandstones been tested at 4 percent cement with a standard compactive effort, they would have met the durability requirements.

In general, the effect of the increased compactive effort on the durability of the materials was not great. The durability increased more with cement content than with compactive effort.

Variations in the durability between the three gradations of the Tyee sandstone were small. All three gradations required the same cement content to meet durability requirements for both compactive efforts. Calculated values of soil-cement loss varied only slightly among the three materials. The table below gives the three gradations and their total accumulated soil-cement loss for all cement contents.

Gradation	Compactive Effort	Total Accumulated Soil-Cement Weight Loss (% by weight)
A	Standard	17.8
	Modified	19.6
B	Standard	15.1
	Modified	11.4
C	Standard	21.8
	Modified	24.0

The results indicate that the B-gradation resulted in the least total soil-cement loss, followed by gradations A and C. (Refer to Table 3 for gradation characteristics.) This order is somewhat different than would be predicted. The A-gradation should be the optimum, followed in order by gradations B and C. The differences are not large, however, and a more reasonable conclusion might be that variations in gradation that were made have little effect on durability.

Strength

The strength test results are summarized in Figure 2. The strengths increased with time for all materials except the marine basalt and Calahan decomposed granite at a standard compactive effort. The density for the marine basalt was considerably higher for the 7-day-cure specimen than for the 28-day-cure specimen. This probably explains the higher strength. No explanation is available for the Calahan decomposed granite except that the two strength values are close and possibly represent the occurrence of an abnormally high strength value for the 7-day-cure test and an abnormally low strength value for the 28-day-cure test.

The strengths obtained were higher for specimens at a modified compactive effort at equal cement contents in all cases except for the marine basalt with 8 percent cement. The marine basalt had equal strengths at 8 percent for the modified and standard compaction specimens. The ratios of strengths at modified to standard compaction varied only slightly with cement content and material. The average value of the ratio was 1.45, but the Colville granite had a ratio of 1.80 and the marine basalt, as previously mentioned, had a ratio near 1.0.

The effect of variations of the gradation of the Tyee sandstone on strengths is illustrated in Figure 3. For the test specimens at the standard compactive effort the order of strengths for the three gradations is not well defined. It is apparent, however, that gradations A and B resulted in higher average strengths than the C-gradation. There is a definite order of strengths for the three gradations at the modified compactive effort. As shown, at both 4 and 7 percent cement, the B-gradation resulted in the highest average strength with gradations A and C following, in that order. This order is coincident with the order established by durability tests and suggests that the B-gradation may be the best for stabilization with cement. All three of the gradations used resulted in a satisfactory stabilized product; however, the test results cannot be extrapolated to identify situations in which gradational limits should be imposed. The C-gradation contained a low amount of silt and clay sizes and 25 percent was retained on the 4.75-mm (No. 4) sieve. The B-gradation contained 25 percent material on the 4.75-mm sieve, but it also contained three times as much material passing the 75-μm (No. 200) sieve. This indicates that the C-gradation is close to the lower limit for material sizes smaller than the 75-μm sieve.

COMPARATIVE COST ANALYSIS

To allow field engineers to determine the feasibility of using a cement-treated base (CTB) rather than a good-quality aggregate base, a comparative cost analysis was performed. Prevailing prices in 1979 for CTB and imported good-quality aggregate in the areas of the three national forests selected for the study were used in the analysis. The comparative cost analysis was further based on the following assumptions:

1. The base would be constructed over a section 8 km (5 miles) long and 6 m (20 ft) wide,
2. The CTB section is 150 mm (6 in) in depth,
3. The cement content used was 6 percent,
4. A structurally equivalent aggregate section is 250 mm (10 in) in depth,
5. A haul distance of 40 km (25 miles) is required to obtain good-quality aggregate,
6. Degrading aggregate for use in the CTB is readily available at or near the site,

Figure 2. Effect of curing time on strength.

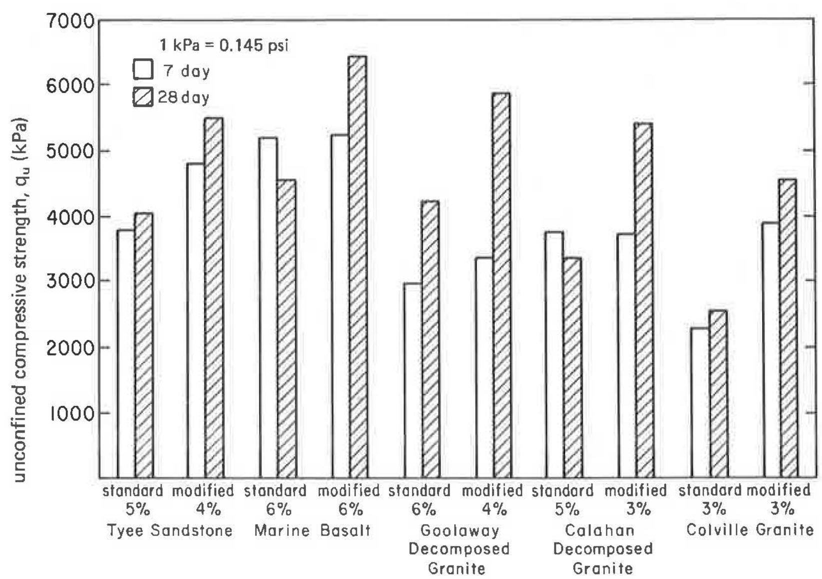


Figure 3. Cement content versus unconfined compressive strength for Tyee sandstone, A, B, and C gradations.

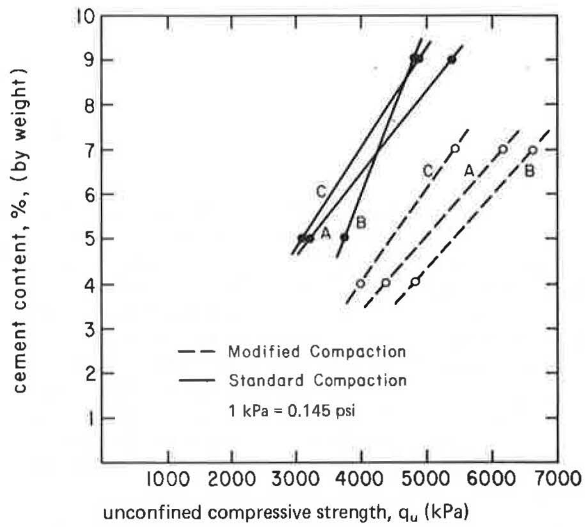


Table 5. Summary of cement-treated base costs.

Item	Unit Price ($/m³)	Total Cost ($)
Ripping, scalping, pit development, and stockpiling	2.11	15 743
Mobilization	0.60	4 498
Plant costs	0.81	6 062
Depreciation and overhead for power unit	3.22	24 054
Grading and excavation	0.71	5 280
Cement (FOB)	7.05	52 703
Haul to job site	2.01	15 058
Placement	1.82	13 591
Subtotal	18.32	136 988
15 percent profit		20 548
Total (cost and profit)[a]		157 536

Note: 1 m³ = 1.31 yd³.

[a]Total = $3.20/m² for 150-mm depth ($0.30/ft² for 6-in depth) and $19 692/km for 6-m width ($31 691/mile for 20-ft width).

7. The degrading aggregate need not be crushed to obtain the required grain size distribution, and
8. A central plant mix operation will be employed to manufacture the CTB.

A summary of the CTB costs is given in Table 5. A summary of the aggregate base costs is given in Table 6. Based on the assumptions made, the CTB is an economically feasible alternative. As the haul distance for the good-quality aggregate increases, the feasibility increases. The costs for the good-quality aggregate base would just equal those of the CTB for a haul distance of approximately 19 km (12 miles). This distance increases to 30 km (19 miles) if the poor-quality aggregate requires crushing and screening to obtain the required grain size distribution.

Table 6. Summary of aggregate base course costs.

Item	Unit Price ($/m³)	Total Cost ($)
Crushing	3.92	48 855
Blasting and ripping	1.70	21 172
Pit development	0.46	5 700
Mobilization-demobilization	0.78	9 771
Load from stockpile	0.47	5 863
Haul (40 km)	9.12	113 998
Process aggregate with grader	0.46	5 700
Rolling	0.35	4 397
Total[a]	17.30	215 456

Note: 1 m³ = 1.31 yd³.

[a]Profit is included in unit prices listed in the table. Total = $4.45/m² for 250-mm depth ($0.41/ft² for 10-in depth) and $26 932/km for 6-m width ($43 343/mile for 20-ft width).

CONCLUSIONS

For the degrading aggregates considered in this study, cement appears to be a suitable stabilizer. All of the materials tested showed satisfactory durability at relatively low cement contents. Minor variations in gradation (variations that do not change the material classification) have little effect on the strength or durability of the stabilized aggregate when compared to the effects due to a change in cement content.

Increasing the compactive effort may lead to a decreased cement requirement and associated economic savings. However, the choice of higher compactive effort requires a project-by-project evaluation and should only be applied where field conditions can ensure higher densities can be obtained. Stabilizing degrading aggregate with cement should be a standard design alternative to good-quality rock in base courses whenever haul distances for the good-quality aggregate become excessive. For the example considered in this study, when the haul distance exceeded 19 km (12 miles), cement stabilization of degrading aggregates was found to be economically justified.

ACKNOWLEDGMENT

The studies described were supported by the U.S. Department of Agriculture, Forest Service, Region 6, Portland, Oregon. The support of the U.S. Forest Service is gratefully acknowledged. Lynne Irwin reviewed the manuscript and offered many helpful suggestions.

REFERENCES

1. I. J. Huddleston. Cement Stabilization of Poor Quality National Forest Bedrock Materials for Road Construction. Oregon State University, Corvallis, M. S. thesis, 1978.
2. Soil-Cement Laboratory Handbook. Portland Cement Association, Skokie, IL, 1971.
3. L. T. Norling and R. G. Packard. Expanded, Short-Cut Test Method for Determining Cement Factors for Sandy Soils. HRB, Highway Research Bull. 198, 1958, pp. 20-31.

Abridgment

Use of Crushed Stone Screenings in Highway Construction

Ignat V. Kalcheff and Charles A. Machemehl, Jr.

Quarried crushed stone is a basic nonmetallic commodity that, according to U.S. Bureau of Mines' statistics for 1977 (1), was produced by 3177 quarries in the United States. The total output was 865 million Mg (954 million tons) valued at \$2.35 billion or an overall average of \$2.72/Mg (\$2.47/ton). The average value has increased by only \$1.54/Mg (\$1.40/ton) since 1927. The material indeed qualifies as a low-cost material.

A portion of total crushed stone production is crushed stone screenings. Crushed stone screenings are the finer fraction of stone products and normally contain particles that are 4-5 mm (0.15-0.19 in) in size and smaller. The particle size distribution, particle shape, and other physical properties may be somewhat different from one geographical location to another and depend primarily on the type of rock quarried, the equipment used for crushing, the ratio of reduction, and the method used for separation of the screenings from the coarse particles. The product at one location, because it has been crushed by the same equipment, is fairly uniform and consistent in physical properties.

The general terminology for stone screenings, stone dust, or crushed fines was established when quarried rock was processed to obtain the coarse aggregate only, and the fine portion of the total product was generated as a waste by-product. With advancement of the technology and with the availability of modern crushing and sizing equipment, crushed stone fine aggregates are being produced intentionally and used successfully wherever fine aggregates are needed.

Crushed stone sands can be produced to certain desired gradations. Manufactured sands are usually produced from crushing operations by using controlled one-size feed aggregates. Stone screenings can be processed in terms of gradation and used as stone sands, provided mix proportions are properly adjusted.

PRODUCTION

Ledge rock is blasted to reduce the size of stone to manageable pieces and then normally crushed by a primary crushing system to reduce the product to sizes less than 125-150 mm (5-6 in). The crushed material is transported directly to secondary crushers, or to a surge-pile and then passed through secondary crushers. Secondary crushers may be classified as one of the following types:

1. Cone and gyratory,
2. Jaw,
3. Roll,
4. Impact,
5. Hammermill, and
6. Cage mill.

Each type has been used, and a certain portion of the resulting product can and usually is crushed for further reduction in size through tertiary or even fourth-state crushing, as described by McLean (2) in detailed comparative analyses of secondary crusher types.

The accumulated fine particles produced during the blasting, transportation, and primary and secondary crushing, if separated on the 4.75-mm (No. 4) mesh sieve, would compose a product that may generally be described as screenings. Screenings may be in the range of 10-20 percent of the total production. During tertiary reduction, the -4.75-mm mesh sieve material ranges from 20 to 60 percent or more of the feed. Tertiary and fourth-state crushing, however, produce fines that have special uses. The cost of these fines is greater due to the added cost in additional crushing, handling, and screening.

Assuming that screenings constitute about 12 percent of total stone production at U.S. quarries, the total amount of screenings produced in the United States during 1977 would have been approximately 104 million Mg (115 million tons). If used only as a subbase for a 3.66-m (12-ft) wide roadway with a thickness of 152 mm (6 in), 86 886 km (54 000 miles) of subbase could have been constructed.

PROPERTIES OF STONE SCREENINGS

Crushed stone screenings, because of the processes involved, may vary in size distribution, but in all cases consist of broken particles that have sharp corners. They may contain weathered rock from the quarry or overburdened material but, generally speaking, do not contain large quantities of plastic fines. In an extreme case, the plasticity index could be around 6 percent, in which case the screenings would be called dirty screenings. The largest quantity of the screenings produced are nonplastic and are the product of crushing rock from sound stone ledges. Studies reported by Shergold (3) indicate that the grading of crushed fines produced from different rock types by use of different types of crushers can be slightly different, but the products are uniform.

Two advantages of stone screenings for highway construction are their uniform grading and crushed faces. The particles provide great shear resistance when compacted. Such products are available and are suitable for mixing with coarse aggregate in a variety of aggregate mixes or for use in upgrading weak soils. They are very effective for stabilization since their nonplastic nature and clean particle surfaces permit full use of the stabilizing agent.

MECHANICAL STABILIZATION OF WEAK SUBGRADES WITH STONE SCREENINGS

Studies reported by Thompson (4) indicate that many typical fine-grained soils do not develop California bearing ratios (CBRs) in excess of 6-8 when compacted at or above the AASHTO T99 optimum water content, which is the minimum required during construction. Load support properties of weak subgrade therefore must be increased.

One procedure that may be employed to increase the CBR of fine-grained soils is to add stone screenings to the soil and compact after thorough mixing and moisture control. Such a remedy is suitable in place of the procedure known as "undercut and backfill" with other materials since 40-60 percent of that which would otherwise be undercut can be used again. The quantity of stone screenings needed will depend on the soils and the desired CBR. Test data reported by Kalcheff (5) provide guidance on how to estimate the quantity needed. For small projects when full laboratory preevaluation cost cannot be justified, one may consider the use of 50 percent screenings.

As one example of a project constructed with soil stabilization with stone screenings, Atlanta's downtown expressway can be cited. In the early 1950s, the Georgia Department of Transportation used 80 percent screenings and 20 percent bank clay—mixed in place as a subbase under the concrete pavement. This expressway, now Interstates 75 and 85, carries in excess of 140 000 vehicles/day. The pavement has performed well, no pumping has been noted, and it continues to be in excellent condition without resurfacing or overlays after almost 30 years.

CEMENT STABILIZATION

Stone screenings have been used combined with portland cement as cement-stabilized bases in many areas. When compacted, such products develop high rigidity with a small amount of portland cement. In comparison to soil-cement stabilization, stone screenings require less cement.

For the most efficient cement use in stabilizing, the screenings should have sufficient fines (material passing 0.075-mm sieve). Normally produced screenings, if not washed and if composed of nondeleterious fines, are best suited for cement stabilization.

USE OF STONE SCREENINGS IN ASPHALTIC CONCRETE

The superiority of crushed sand and stone screenings over uncrushed, smooth-surfaced, rounded aggregates for use in asphaltic concrete has been recognized for a long time. Research studies, such as those by Herrin and Goetz (6), present data from triaxial tests for dense and open-graded mixes and show that the ultimate triaxial strengths are always higher for combinations that use crushed fine aggregates. Research studies in the Asphalt Institute Laboratory by Griffith and Kallas (7) confirm the significant advantages of crushed stone screenings, particularly when larger quantities are used.

A recent study in the National Crushed Stone Association (NCSA) laboratory, reported by Nichols and Kalcheff (8), of asphalt-aggregate mixes under repetitive loading gives more information on the properties of mixtures that contain stone screenings. When the crushed particles are less cubical, the mixes exhibit higher voids in mineral aggregate (VMA) and thus a slightly greater asphalt content is required for meeting the void criteria for mix design—unless more dust of fracture is included as filler. In summary, the findings of the above study in which stone sand was a product of stone screenings were as follows:

1. All-crushed stone mixes had higher Marshall stability and considerably greater resistance to permanent deformation from repeated loads than did identically graded mixes that contained uncrushed sand;
2. Increases in the asphalt content by 0.5 percent above optimum in all-crushed stone mixes had little effect on the permanent deformation properties; mixes that contained uncrushed sand were quite sensitive to increased asphalt contents.
3. The deformation properties of all-crushed stone mixes were less affected by increased temperature during repeated loadings than those mixes that contained uncrushed sand; and
4. The deformation properties of all-crushed stone mixes were less affected by inadequate initial compaction than were those of uncrushed sand mixes, a finding that tends to counter the fact that all-stone mixes are somewhat more difficult to compact.

USE OF STONE SCREENINGS IN PORTLAND CEMENT CONCRETE

As suburbs continue to encroach on local natural sand deposits and as good sources of natural sand become further depleted, it is inevitable that the demand for manufactured sand will continue to increase. When screenings are produced for portland cement concrete, crushers are usually selected to produce bulky particles, and some of the material finer than 75 μm is removed.

Crushed fine aggregate has been used successfully for many years in mass concrete for dams, paving concrete for roadways, structural concrete for buildings, mortar for masonry, and a large number of special construction projects. An example of the extensive laboratory studies reported by Kalcheff (9) is presented in Table 1 for the purpose of giving the reader specifics on concrete properties with stone screenings. The fine aggregates that have shape indices 53 and 55 were stone screenings.

Table 1. Properties of portland cement concrete with crushed and uncrushed fine aggregates.

Shape Index	Fine Aggregate Type	Grading[a]	Passing 0.075-mm Sieve (%)	Water-Cement Ratio	After 28 Days: Compressive Strength (MPa)	After 28 Days: Flexural Strength (MPa)	Shrinkage After 3 Years 0.01 Percent
47	Uncrushed	A	3 ±1	0.55	32.7	5.31	5.2
49	Crushed	A	3 ±1	0.56	35.2	5.69	5.4
		C	11 ±1	0.54	35.4	5.79	4.9
50	Crushed	A	3 ±1	0.57	31.4	5.31	7.0
		C	11 ±1	0.57	31.0	5.48	7.2
51	Crushed	A	3 ±1	0.57	60.8	5.38	5.6
		C	11 ±1	0.57	30.2	5.38	6.0
53	Crushed	A	3 ±1	0.61	28.3	4.83	6.9
		C	11 ±1	0.63	27.2	4.86	6.5
55	Crushed	A	3 ±1	0.67	29.0	4.75	7.1
		C	11 ±1	0.67	29.0	4.90	6.8

Notes: 1 MPa = 145 lbf/in^2.
Portland cement concrete mixes were proportioned with constant 279 kg of cement/m^3 of concrete (5 bags/yd^3) with coarse aggregate size No. 57 (ASTM D448) crushed limestone for 76 ±13 mm (3 ±0.5 in) slump and Vinsol resin for 5.5 ±0 percent entrained air.

[a]No results reported for uncrushed 47, graded only.

USE OF STONE SCREENINGS AS ROADBASE AGGREGATE

Stone screenings are used extensively in roadbase and subbase mixes and are used in combination with coarse aggregate. This type of use reduces the cost of the combined product because the screenings do not have to be separated and reblended. The Bureau of Mines' statistics (1) show that during 1977 more than 340 million Mg (375 million tons) of roadstone and roadbase aggregates were used in the United States. The reasons for that are basically two: (a) these combinations are lower-cost construction materials, and (b) the materials are exceptionally good and suitable for base construction without stabilizing additives.

Screenings have been used in roadbase construction ever since the first broken rock was produced by man. Due to modern technology available for mechanized construction, stone screenings for road construction are being used to an even greater extent. The past, present, and future role of unbound aggregates (a portion of which are stone screenings) in road construction are summed up in the proceedings (10) from a national conference in 1974. The subject of load-deformation characteristics, other fundamental properties, design procedures, production control systems, and quality assurance are extensively discussed in the proceedings.

NCSA staff, with the assistance of NCSA committee members, has prepared a number of manuals on the use of crushed stone products (including stone screenings) for specific purposes, such as construction of parking areas, streets, low-volume roads, highways, shoulders, and airports. Engineers and designers should consider the use of low-cost crushed stone materials for construction. These materials are available today, and the forecast for crushed stone by the year 2000 is on the order of 27 billion Mg (30 billion tons). Stone screenings account for 12 percent of that estimated total; therefore, more than 3 billion Mg (5 billion tons) of stone screenings will be used between now and then.

OTHER USES OF STONE SCREENINGS

Not all the uses of stone screenings for highway construction have been discussed here. The overall subject is very broad. Other applications of stone screenings include bedding materials, fillers, granulars for drain fields, fills, mixtures for de-icing, patches, slurry seals, surface treatment, and overlays.

REFERENCES

1. A. H. Reed. Stone in 1977. In Mineral Industry Surveys. Bureau of Mines, U.S. Department of the Interior, annual prelim. ed., 1977.
2. E. A. McLean. A Comparative Analysis of Secondary Crusher Types. Presented at the 53rd Annual Convention, National Crushed Stone Association, Washington, DC, 1970.
3. F. A. Shergold. The Grading of Crusher Fines. Roads and Road Construction, Vol. 35, No. 410, Feb. 1957, pp. 36-40.
4. M. R. Thompson. Subgrade Stability. TRB, Transportation Research Record 705, 1979, pp. 32-41.
5. I. V. Kalcheff. Mechanical Stabilization of Weak Subgrade Soils with Crushed Stone Products. National Crushed Stone Association, Washington, DC, 1971.
6. M. Herrin and W. H. Goetz. Effect of Aggregate Shape on Stability of Bituminous Mixes. HRB, Proc., Vol. 33, 1954, pp. 293-308.
7. J. M. Griffith and B. F. Kallas. Influence of Fine Aggregates on Asphaltic Concrete Paving Mixtures. HRB, Proc., Vol. 37, 1958, pp. 219-254.
8. F. P. Nichols, Jr., and I. V. Kalcheff. Asphalt-Aggregate Mix Evaluation from Repetitive Compression and Indirect Tensile Splitting Tests. National Crushed Stone Association, Washington, DC, 1979.
9. I. V. Kalcheff. Portland Cement Concrete with Stone Sand: Special Engineering Report. National Crushed Stone Association, Washington, DC, 1977.
10. Utilization of Graded Aggregate Base Materials in Flexible Pavements. Proc., National Crushed Stone Association, National Sand and Gravel Association, and National Slag Association, Oak Brook, IL, 1974.

Sulphur-Asphalt Pavement Technology: A Review of Progress

Thomas W. Kennedy and Ralph Haas

This paper briefly summarizes the current status of sulphur-asphalt pavement technology with emphasis on sulphur-extended asphalts. The various processes that are currently available are discussed and compared, and the various field trials are described. Performance observations and engineering properties are also considered. Finally, the future use, applications, and problems of sulphur-asphalt are reviewed. Based on experience, the use of sulphur-asphalt mixtures can be expected to increase during the next few years. This is especially true of sulphur-extended asphalt mixtures, which have greater applicability and conserve asphalt and produce a corresponding reduction in cost.

The accumulation of surplus sulphur, the need for improved paving mixtures, and the dwindling supply of asphalt and its rapidly increasing cost have provided the incentive to develop new uses for sulphur for the paving industry. One of the largest such applications is the use of sulphur in sulphur-asphalt mixtures.

Two basic approaches have been used. Either sulphur can be added to the mixture or it can replace a portion of the asphalt [sulphur-extended asphalt (SEA)]. Both processes have definite applications, and each has

Table 1. Method for using sulphur in asphalt mixtures.

Basic Method	Example Sources	Features	Example Field Applications	Some Limitations, Actual and Possible
Liquid sulphur addition to hot sand-asphalt mixes	Shell Canada Ltd.	Use of marginal materials (i.e., unstable sands); no compaction requirements	Richmond, British Columbia, 1970 Tilsonburg, Ontario, 1972 Maclean, Saskatchewan, 1974 Sulphur, LA, 1977	Special equipment (i.e., insulated trucks); high quantities of sulphur; questionable economics, except for special situations
	Societe Nationale des Petroles d'Aquitaine	Potential economy; extension of asphalt supply; use of conventional paving equipment	Perimeter road of plant at Lacq in Western France, 1973 Lufkin, TX, 1975	Storage (i.e., costs, formation of H_2S, need for inert cover gas); need for additives to maintain storage stability; extra operators at plant; elemental sulphur vapor at paving site
Preblending of liquid sulphur and asphalt to produce SEA binder	Gulf Canada Ltd.	Potential economy; extension of asphalt supply; use of conventional paving equipment; production of binder, on site, on demand; no additives required	Alberta, 1974, 1977 Ontario, 1975, 1977, 1978, 1979 Michigan, 1977, 1979 Holland, 1978 Louisiana, 1978 Florida, 1979 Minnesota, 1979	Extra operators at plant; elemental sulphur vapor at paving site
	SUDIC	Potential economy; extension of asphalt supply; use of conventional paving equipment; production of binder, on site, on demand	Alberta, 1975, 1977 British Columbia, 1979	Extra operators at plant; elemental sulphur vapor at paving site
Pugmill blending of liquid sulphur and asphalt to produce SEA binder	U.S. Bureau of Mines	Potential economy; extension of asphalt supply; use of conventional paving equipment; no additives required	Nevada, 1977	Elemental sulphur vapor at paving site; uniformity of dispersion; aggregate coating

been used successfully in various field construction projects.

The purpose of this paper is to review these processes and the field trials that have been conducted to date and to illustrate some of the properties of paving mixtures that have sulphur addition.

AVAILABLE PROCESSES

A number of projects have incorporated elemental sulphur in asphalt mixtures. The general objectives were to use sulphur and to obtain improved mechanical properties of the mixtures. Current efforts also have these objectives; they are concerned with achieving improved economy and extending the available supplies of asphalt.

The first application of sulphur in the paving industry was on the Ohio Department of Highways experimental road in Hocking County in Ohio in 1935 (1). At about the same time, a paving-brick filler formulation named Sulmor was developed by Litehiser and Schofield (2). It consisted of asphalt and Thiokol-plasticized sulphur.

Another nearly concurrent effort was initiated by Bacon and Bencowitz in 1936 (3) and patented in 1939 (R. F. Bacon and I. Bencowitz, U.S. Patent 2 182 837, 1939). It included vigorous stirring of as much as 50 percent of elemental sulphur in asphalt at 149°C (300°F) and is known as the Texas Gulf process. Paving mixtures that contained this binder were tested extensively and a small experimental road was constructed.

The major methods currently available for using sulphur in asphalt mixtures, not including such paving materials as Sulphlex, which is currently being investigated by the Southwest Research Institute, have been developed in the mid-1960s to early 1970s. They can be classified as follows:

1. Addition of sulphur to hot sand-asphalt mixtures—Shell Thermopave process,
2. Preblending of sulphur and asphalt to produce SEA—Societe Nationale des Petroles d'Aquitaine (SNPA) process, Gulf Canada process, Sulphur Development Institute of Canada (SUDIC) (Pronk) process for Thermal Asphalt, and
3. Pugmill blending of sulphur and asphalt to produce SEA—U.S. Bureau of Mines process.

The basic approach in the first method is to add hot liquid elemental sulphur, essentially as a filler, to a hot sand-asphalt mix during a second mixing cycle, which occurs after the asphalt has been mixed with the aggregate. In the second method, the basic approach is to disperse hot liquid elemental sulphur in asphalt to create an SEA binder, which is then mixed with aggregate in the same way that asphalt alone is used in conventional mixtures. The third method also attempts to create an SEA binder by the separate addition of the components and pugmill blending of sulphur and asphalt, rather than by preblending. Table 1 summarizes information related to these processes.

Shell Process

The Shell Thermopave process, developed in the 1960s, involved the first substantial use of sulphur in asphalt (Shell International Research Maatschaapij, Ger. Offen. 2 149 676). The basic intent was to use large amounts of surplus sulphur by incorporating it in sand-asphalt mixtures of low stability (i.e., mixtures that contain poorly graded, unstable sands to produce mixtures of high stability).

The process, which has been described in several sources (4-10), essentially consists of the following two consecutive mixing cycles:

1. Mixing sand with asphalt and
2. Mixing sand-asphalt with molten, elemental sulphur.

Temperature of the mixing processes is between 132°C and 149°C (270°F and 300°F) and the final mixture composition has an aggregate-asphalt-sulphur weight ratio of approximately 82-6-12 (10).

The Thermopave mixture production operation requires plant modifications, as well as insulated trucks for hauling to avoid freezing, which occurs at about 116°C (240°F). The mixture can be placed by using either forms or a modified paving machine. No compaction is required.

Mechanical stability of the mixture is very high because the solidified sulphur, which fills the interstitial voids, becomes part of the aggregate structure.

Claims for a process somewhat similar to that of Shell's Thermopave were filed in 1961 by Standard Oil, Chicago (Standard Oil Company, Chicago, U.S. Patent 3 239 361, 1966). The similarity lies in the fact that sulphur is incorporated in the asphalt-aggregate mixture as a post-mix addition technique. However, the Standard Oil process adds the sulphur in finely divided or powdered form and includes the use of carbon (7.5 percent by weight).

More recently, the major emphasis in the Thermopave process has been in the area of maintenance, by using what is known as Thermopatch.

Aquitaine Process

The French SNPA, or Aquitaine, process is essentially a modification of the earlier Texas Gulf process. Instead of using plain elemental sulphur, preplasticized sulphur is used, and additives such as polysulfide (Thiokol LP3) are added before it is blended with the asphalt. Otherwise, the blending or homogenizing, composition, and applications are identical to the Texas Gulf process.

The purpose of the additives used by Aquitaine is basically for stabilization of the sulphur bitumen (S-B) emulsion. Numerous patents in the field of sulphur plasticization have been obtained by Aquitaine during the past 10 years (Societe Nationale des Petroles d'Aquitaine, U.S. Patent 1 303 318); however, generally Aquitaine binders contain only sulphur asphalt.

In 1973, Aquitaine described the application of the S-B emulsion in road paving (11). During the same year, a test strip was constructed outside the refinery, near Lacq, France. The process and the field application were described to the American Chemical Society in Los Angeles in 1974 (12). Additional examples of the use of this process have been described by Gallaway and Saylak (13).

The Aquitaine process offers potential benefits of economy, extension of existing asphalt supply, and use of conventional paving equipment; however, it requires additives in order to ensure stability if storage is required. Moreover, high levels of H_2S are formed in storage and a serious explosion danger can exist. The use of an inert gas (such as nitrogen) may be necessary to reduce these dangers. Apparently the use of additives does not significantly affect the amount of sulphur remaining in a crystalline state; rather, the additives seem to be required purely to maintain stability during storage.

Gulf Canada Process

The Gulf Canada process (14-24) also involves the dispersion of molten, elemental sulphur in asphalt. However, it does not incorporate any additives and uses the asphalt medium itself as a plasticizing agent.

The SEA binder is produced on demand at the paving site by using a sulphur-asphalt module (SAM). Essentially, this unit receives asphalt from a supply line to the asphalt storage tank and molten elemental sulphur from another storage tank, both at temperatures between about 132°C and 149°C (270°F and 300°F) and mixes them into a continuous stream of binder. This SEA binder is then fed into the pugmill. A bypass valve allows for switching to pure asphalt binder with no shutdown of the mixing plant. The SAM unit can be used with either a batch type or continuous type of mix plant and its use has been described in detail by Kennepohl (14).

Other Processes

The Thermal Asphalt process, now handled through SUDIC, produces a preblended SEA binder similar in nature to that produced in the Aquitaine and Gulf Canada processes (25, 26).

Production of an SEA binder is also claimed to be possible through separate addition of sulphur and asphalt to the pugmill, as reported by McBee and Sullivan (27). This process, developed through the U.S. Bureau of Mines, is apparently simpler than the preblending process and avoids patent problems. However, it still requires separate asphalt and sulphur storage plus pumping and metering, can require a longer mixing cycle, does not always result in the uniformity of dispersion achieved with preblending, can result in preferential coating for certain types and gradations of aggregate, and can result in mixtures that have certain properties and durability characteristics that are different from those achieved with preblending. It should be emphasized, however, that these are possible limitations and remain to be quantitatively demonstrated and documented.

FIELD TRIALS

Major field trials of sulphur use in asphalt mixtures during the past decade were started by Shell Canada Ltd. on their Thermopave test roads at Richmond, British Columbia (1970), at Tilsonburg, Ontario (1972), and at McLean, Saskatchewan (1974). These were followed by test roads that used the SNPA process in France (1973), the Gulf Canada process in Alberta (1974), the SUDIC process in Alberta (1975), and the U.S. Bureau of Mines process in Nevada (1977) (Table 1). The period between the mid-1970s and 1979 saw a considerable number of field trials constructed in North America and Europe. Table 2 contains a partial summary.

These field trials demonstrated that SEA binders could be easily produced on site in commercial quantities and that the mixes could be hauled, placed, and compacted with conventional equipment. Extensive, ongoing engineering evaluation of the mixes and test roads is being carried out by both of the previously noted developers of the processes and by the user agencies. A number of the quoted references contain certain interim evaluation information. The major objectives of the field trials have been to

1. Demonstrate full-scale construction feasibility;
2. Provide a basis for longer-term, in-service evaluation of sulphur-asphalt pavements and the effects of the variables that can affect behavior and performance; and
3. Provide a basis for developing a design technology for sulphur-asphalt pavements.

OBSERVATION OF PERFORMANCE

Experience to date has shown that paving mixes that have SEA binder can be routinely produced for regular, full-scale construction and that hauling, placing, and compaction can be accomplished with conventional equipment.

Observations of behavior and performance will, of course, continue for several years and it is premature to make extensive conclusions. Nevertheless, the following interim conclusions can be made.

1. Paving mixes made with SEA binder can vary markedly in stiffness response, depending on the grade of asphalt used and the sulphur-asphalt ratio, as subsequently illustrated. This allows considerable design flexibility in tailoring to particular conditions of traffic, temperature, subgrade support, and materials availability.
2. Thickness reductions due to increased stiffness are theoretically possible in certain situations, but this has not yet been verified by field observations. There are other situations where increased stiffness can in fact be counterproductive (i.e., where a thin slab effect and strain-controlled fatigue exist).
3. Overlay construction seems to have been particularly successful. The 1974 overlay at Windfall, Alberta, shows no significant distress to date.
4. Full-depth construction on granular bases has generally performed quite well, although certain conditions, particularly where soft subgrades are involved, can result in premature distress (28).

ENGINEERING PROPERTIES

The engineering properties of the various sulphur-asphalt binders have been evaluated by various investigators (17, 19). These properties have provided a basis for the design of actual field projects, including the structural design, and have been used to compare these materials to conventional mixtures. Engineering properties have included Marshall and Hveem mixture design properties, resilient moduli, fatigue behavior, low-temperature stiffness, temperature susceptibility, and tensile and compressive strengths.

Table 2. Partial summary of sulphur-asphalt field trials.

Process	Location	Date of Construction	Comments
Shell Canada	British Columbia	1970	Hot-mix, sand-sulphur-asphalt base for urban street construction
Shell Canada	Ontario	1972	Hot-mix, sand-asphalt-sulphur mix in rural highway application
SNPA	France	1973	SEA mix for perimeter road construction at SNPA plant
Gulf Canada	Port Colborne, Ontario	1974	Demonstrate operation of original SAM equipment and feasibility of full-scale production
Gulf Canada	Blue Ridge, Alberta	1974	Five sections of varying thickness; demonstration of full-scale production with conventional equipment and determination of layer equivalency values
Gulf Canada	Windfall, Alberta	1974	Two sections of overlay at two thicknesses, total length 1.6 km; demonstration of sulphur-asphalt overlay and determination of performance parameters such as rutting, crack reflection, and initiation
SNPA	Lufkin, TX	1975	Hot-mix base construction of varying thicknesses
Gulf Canada	Renfrew, Ontario	1975	Seven sections, 2900 m^2 each at 63.5 mm, total length 3.2 km; evaluation of effects of penetration, sulphur-asphalt ratio, binder, and antistripping agents
SUDIC	Calgary, Alberta	1975	Urban street applications of SEA binder hot-mix construction
Gulf Canada	Mellville, Saskatchewan	1976	Demonstration of production of SEA binder for drain mixing; testing for commercial SAM unit
SUDIC	Rocky Mountain House, Alberta	1977	Full-scale construction of several kilometers of SEA pavement plus several test sections of varying thickness
Shell Canada	Sulphur, LA	1977	Demonstration project of sulphur-sand-asphalt hot-mix base construction
U. S. Bureau of Mines	Nevada	1977	Demonstration of full-scale production of SEA mixes by using pugmill blending of sulphur and asphalt for test road
Gulf Canada	Midland, MI	1977	Four sections, 1.6 km total length, to overlay on portland concrete cement
Gulf Canada	Sturgeon Falls, Ontario	1977	Four sections, total length 3.2 km to evaluate SEA binder with soft asphalt for improvement of low-temperature performance
Gulf Canada	Rocky Mountain House, Alberta	1977	Full-scale production run plus 2 km of test sections of varying thickness to determine layer equivalencies
Gulf Canada	Woodstock, Ontario	1978	5000 m^2 at 127 mm; total length 0.8 km; evaluation of heavy traffic effect on SEA pavement with soft asphalt
Gulf Canada	Siddeburen, Holland	1978	2090 m^2 at 330 mm; total length 275 m; demonstration of sulphur-asphalt in the Netherlands
Gulf Canada	Rotterdam, Holland	1978	2500 m^2 at 300 mm; total length 550 m; full-depth SEA on sand base and as overlay under construction practices in Holland; major expressway estimated average daily traffic of 125 000
Gulf Canada	Louisiana	1978	10 800 m^2 at 175 mm; total length 1.6 km; evaluation of SEA in construction with low-quality aggregate (sands) as possible replacement of cement stabilized base construction
Gulf Canada	Gainesville, FL	1979	10 000 m^2 at 75, 125, and 175 mm; total length 1.2 km; demonstration of sulphur-asphalt construction; evaluation of sulphur-asphalt under high-volume traffic; layer equivalency
Gulf Canada	Route-63, MN	1979	Demonstration of SEA performance in overlay as part of longer test project with Petromat and carbon black; 0.8 km; use of drum mixer
Gulf Canada	MI-99, MI	1979	Overlay, 6.4 km long; varying thickness; two sulphur-asphalt ratios; use of drum mixer
Gulf Canada	Route-400, Ontario	1979	Varying thicknesses, from ¼ to 190 mm; on granular base, 1.6-km length; layer equivalencies
SUDIC	British Columbia	1979	SEA in hot-mix construction on highway project

Notes: 1 m^2 = 1.19 yd^2; 1 mm = 0.039 in; 1 km = 0.62 mile; 1 m = 1.09 yd.
Other sulphur-asphalt projects have occurred in Saudi Arabia, West Germany, Illinois, and Maine.

Marshall and Hveem Properties

Sulphur-asphalt mixtures can be designed by conventional methods. In addition, conventional design data and index properties are well understood. Table 3 (22) provides a comparison of average Marshall data for test specimens prepared by ASTM D 1559. The data show the sulphur-asphalt (SA) binder that contained 50 percent sulphur exhibited considerably higher stabilities than conventional mixtures. However, 20 percent asphalt-sulphur produced only a marginal increase. These higher stabilities resulted in no loss in flow properties. Figure 1 (13) illustrates a typical relationship between binder content and stability for conventional and sulphur-asphalt mixtures. Sand mixes (Thermopave) also generally have Marshall stabilities in excess of 9-11 kN (2000-2500 lbf) (10).

Sulphur-extended mixtures also can be expected to exhibit generally higher Hveem stability values. Table 4 and Figure 2 (13) illustrate typical relationships between Hveem stabilities for conventional and sulphur-asphalt mixtures.

Table 3. Stability values for sulphur-asphalt mixtures—Marshall test data.

Binder Type	Binder Percentage by Weight	Voids (%)	Flow (mm)	Voids in Mineral Aggregate (%)	Stability (N)
40-50 Penetration Asphalt					
Asphalt	6.0	3.5	3.6	17.7	12 400
20/80 SA	6.5	3.1	3.0	17.5	12 800
50/50 SA	7.0	3.7	3.5	17.0	20 800
85-100 Penetration Asphalt					
Asphalt	6.0	2.6	3.1	16.3	9 800
20/80 SA	6.5	3.1	2.7	17.3	10 400
50/50 SA	7.0	4.0	3.3	16.9	22 400

Note: 1 mm = 0.039 in; 1 N = 0.224 lbf.

Figure 1. Relationships between Marshall stability and binder content showing effect of sulphur-asphalt ratio.

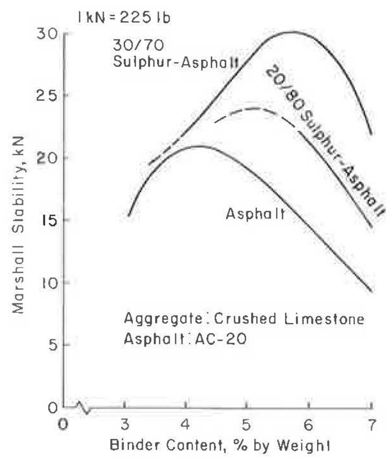
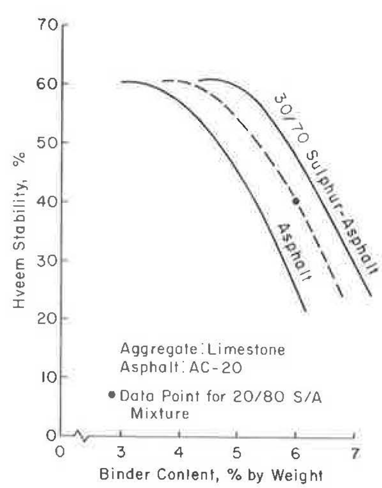


Figure 2. Relationships between Hveem stability and binder content showing effects of sulphur-asphalt ratio.



Resilient Modulus

Generally, the replacement of asphalt with sulphur can be expected to produce higher resilient moduli, but the increase is relatively small up to about 20 percent sulphur. However, for sulphur contents that exceed 50 percent, the resilient moduli increased significantly (17, 19, 22). Figure 3 provides typical resilient moduli for two different penetration grades of asphalt and three levels of sulphur. Other typical values for Texas mixtures are shown in Table 5 (13). Values can range as high as 6 × 10^6 kPa (890 000 lbf/in^2) and may actually be much higher at colder temperatures.

Fatigue Characteristics

The repeated-load indirect tensile test was also used to evaluate the fatigue behavior of sulphur-extended mixtures (17, 19). Typical fatigue life relationships for two penetration grades of asphalt and three levels of sulphur are shown in Figure 4. These results illustrate the effects of temperature and sulphur-asphalt ratio.

Under the stress-controlled loading, increasing the sulphur content to 50 percent provided a substantial increase in the fatigue life; however, there was essentially no improvement for 20 percent sulphur. Lytton (29), however, reported as a result of an analysis that used the VESYS computer program that the fatigue life was actually shortened. This is attributed to the fact that controlled strain conditions were being simulated.

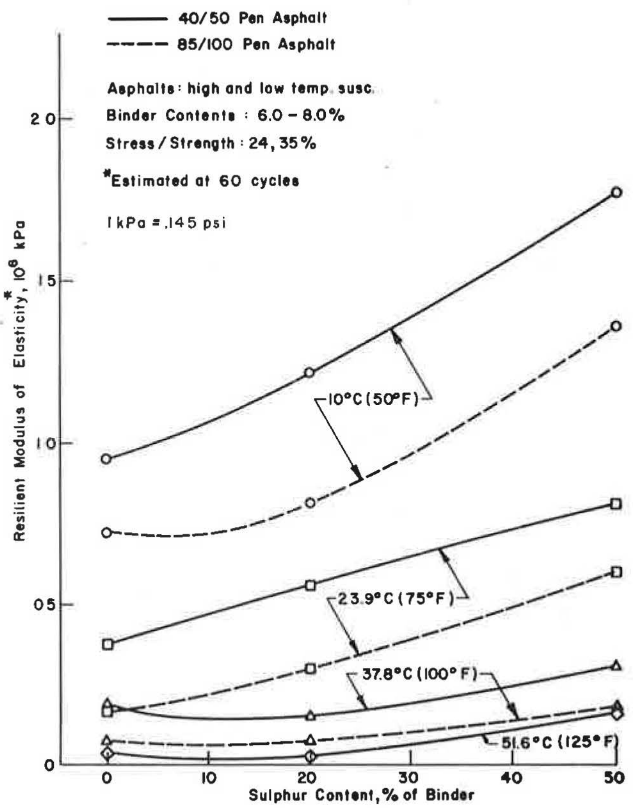
Low-Temperature Stiffness

The addition of sulphur does not have any significant effect on the low-temperature stiffness of the sulphur-

Table 4. Stability values for sulphur-asphalt mixtures—Hveem stability.

Aggregate	Sulphur-Asphalt Mixture	Binder Content (% by weight)	Hveem Stability (%) Laboratory	Hveem Stability (%) Field Data
Limestone	0-100	4	57	
	0-100	5	46	
	0-100	6	26	
	20-80	6	40	
	30-70	5	44	
	30-70	6	37	
	30-70	7	30	
Lufkin sand	30-70	5	30	
	30-70	5.3		34
	30-70	5.5	33	
	30-70	6	31	34
	30-70	6.2		32
	30-70	7	29	29
	30-70	7.5		30
	30-70	8	31	
Lufkin type D	30-70	4.8		40
	30-70	5	44	
	30-70	5.3		39
	30-70	5.5	43	
	30-70	5.65		44
	30-70	6	37	37
	30-70	6.2		36
	30-70	6.5		38
	30-70	7	37	
	30-70	8	37	

Figure 3. Relationships between resilient modulus and sulphur content.



asphalt mixtures (22). The results of an extensive amount of testing over a range of properties indicated that the stiffness modulus of sulphur-asphalt mixtures is affected primarily by the consistency of the asphalt used in the binder and temperature (18). Thus, low-temperature cracking should not be adversely affected by the addition of sulphur to the binder. This has been verified by field observations (22).

Temperature Susceptibility Characteristics

The resilient moduli and low-temperature-stiffness modulus characteristics of SA mixtures can be combined in a single graph, as in Figure 5, to illustrate their temperature susceptibility characteristics over the entire

Table 5. Dynamic (resilient) modulus values for sulphur-asphalt mixtures.

Aggregate	Sulphur-Asphalt Mixture (by weight)	Binder Content (% by weight)	Resilient Modulus (kPa 000s)
Crushed limestone	0-100	5	2790
	0-100	6	2653
	0-100	7	1447
	20-80	6	3169
	30-70	4.5	3541
	30-70	6	6201
	30-70	7	5719
Lufkin type D	30-70	5.5	4134
Lufkin sand	30-70	8	723

Note: 1 kPa = 0.145 lbf/in².

Figure 4. Relationships between fatigue life and temperature for sulphur-asphalt mixtures subjected to a low stress.

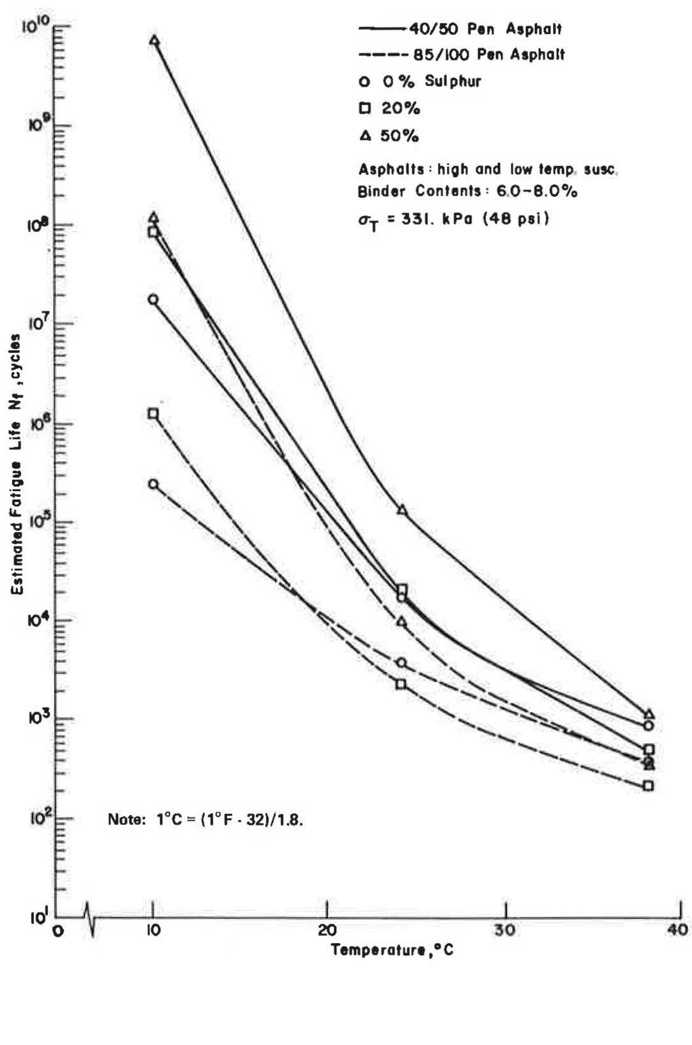


Figure 5. Temperature susceptibility of sulphur-asphalt mixtures.

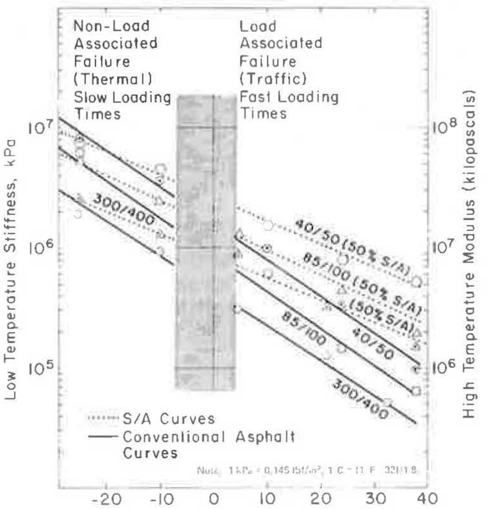
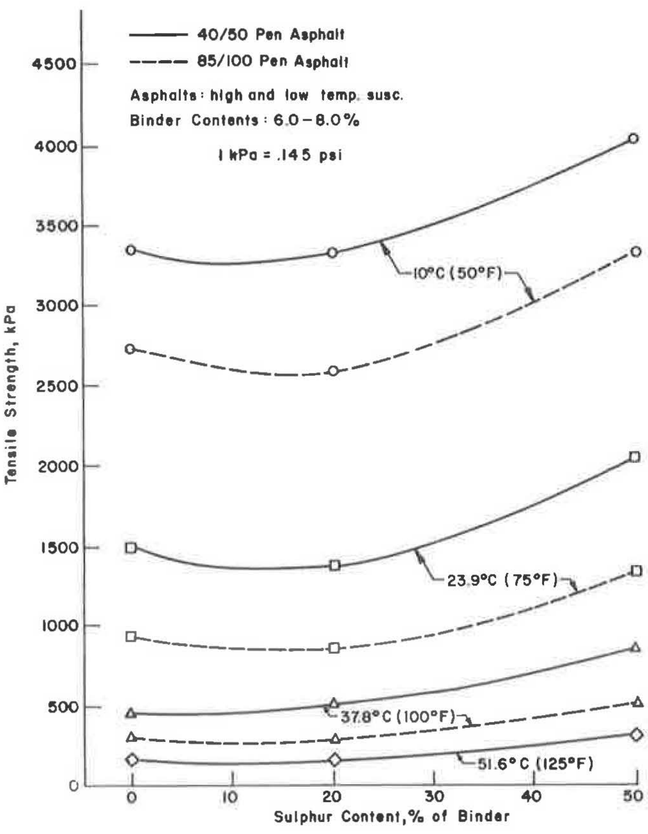


Figure 6. Relationships between tensile strength and sulphur content.



service temperature range (18).

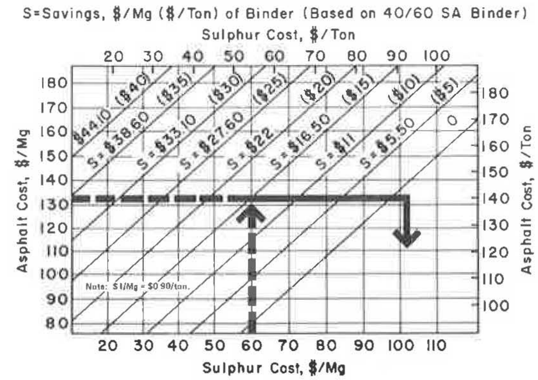
At high temperature, the addition of sulphur to a soft asphalt (e.g., 300-400 penetration) can increase its stiffness in terms of resilient modulus to that of the mixture made with 40-50 penetration asphalt alone. However, as previously noted at low temperatures, the addition of sulphur has no significant effect on stiffness. Thus, it is quite apparent that the effect of the addition of sulphur is very significant at high temperatures (and fast loading times) and insignificant at low temperatures (slow loading times) (22).

Table 6. Compressive strength properties of sulphur-asphalt mixtures.

Aggregate System	Sulphur-Asphalt Mixture	Binder Content (% by weight)	Compressive Strength (kPa)	Tensile Strength (kPa)
Limestone	0-100	5	3307	710
	0-100	6	3169	482
	30-70	5	5340	586
	30-70	6	5519	723
50-50 sand blend	30-70	5[a]	1860	379
	30-70	5.5[a]	4341	503
	30-70	6[a]	2997	427
Lufkin type D	30-70	5.5[a]	1447	379
Lufkin sand	30-70	8[a]	2274	255

Note: 1 kPa = 0.145 lbf/in².
[a] Mixtures used Texaco AC-20; all others used Exxon AC-10.

Figure 7. Binder cost savings with sulphur-extended asphalt binder.



Tensile and Compressive Strengths

Both the tensile and compressive strengths are increased by the addition of sulphur. In the case of tensile strength, the effect of the sulphur is essentially nonexistent until the amount of sulphur exceeds 20 percent (Figure 6). This is essentially the same type of behavior observed for fatigue life and resilient modulus. Additional values of tensile strength are shown in Table 6 for a variety of aggregates, sulphur-asphalt mixtures, and binder contents. Strengths tended to be related to the quality of the aggregate as well as the other two mixture variables. Compressive strengths are also continued in Table 6. As shown, substantial increases in strength generally occurred.

FUTURE POTENTIAL AND QUESTIONS

The use of sulphur in asphalt mixtures should become increasingly attractive from a substitution point of view, per se. For example, as shown in Figure 7, if asphalt is \$132/Mg (\$120/ton) and sulphur is \$60.60/Mg (\$55/ton), there would be a saving of \$16.50/Mg (\$15/ton) of binder used for 40-60 SA ratio. However, it appears that asphalt prices will be well above \$132/Mg (\$120/ton) in many areas of North America in 1980, whereas sulphur prices should not increase as rapidly.

The potential of design flexibility should also continue to make SEA binders more attractive in the future because mixtures can be produced to simultaneously satisfy both low-temperature shrinkage and higher-temperature traffic-loading requirements (17).

If savings in thickness can be realized, these will add to the substitution savings in the binder itself. Both the existing test roads and future planned construction, such as a large-scale overlay project scheduled on I-75 in Florida for late 1979, should allow at least some tentative conclusions to be drawn on this question in the relatively near future.

Extension of asphalt supply through the use of SEA binders should also become increasingly attractive. Spot shortages of asphalt, which have already appeared in 1979 in several areas of North America, can be expected to continue to occur, in association with general petroleum shortages.

The potential of SEA binder mixtures in recycling poses a major question, not so much in the short term but certainly as the first SEA pavements reach the end of their service lives. There seems no reason, however, why these mixtures cannot be recycled effectively, provided direct heating is not used. Also, the potential for using SEA binders in recycling of conventional mixtures should be good.

SUMMARY

Laboratory and field experience has demonstrated that sulphur-asphalt mixtures are more capable of providing improved engineering properties and probably improved field performance than many conventional asphalt mixtures. At this time, emphasis is on the use of SEAs because of their greater applicability and the reduction of asphalt consumption and the corresponding reduction in cost. However, most of the currently available processes can be used and are applicable to specific conditions. Based on current and past experience with sulphur-asphalt mixtures, the use of these materials can be expected to increase during the coming years.

REFERENCES

1. W. W. Duecker. Proc., National Paving and Brick Assn., 1937, p. 60.
2. R. R. Litehiser and H. Z. Schofield. Progress Report on Brick Road Experiments in Ohio. Proc., HRB, Vol. 16, 1936, pp. 182-192.
3. I. Bencowitz. ASTM Preprints No. 95 (9), 539, 1938.
4. R. Hammond, I. J. Deme, and D. McManus. The Use of Sand-Asphalt-Sulphur Mixes for Road Base and Surface Applications. Proc., Canadian Technical Asphalt Assn., Vol. 16, Nov. 1971.
5. I. J. Deme. The Use of Sulphur in Asphalt Paving Mixes. Paper presented at the Chemical Engineering Conference, Symposium on Novel Uses for Sulphur, Vancouver, Sept. 1973.
6. I. J. Deme. Basic Properties of Sand-Asphalt-Sulphur Mixes. Paper presented at the 7th International Road Federation World Meeting, Munich, Oct. 1973.
7. I. J. Deme. Processing of Sand-Asphalt-Sulphur Mixes. Proc., Assn. of Asphalt Paving Technologists, Vol. 43, 1974, pp. 465-490.
8. R. A. Burgess and I. J. Deme. The Development of the Use of Sulphur in Asphalt Paving Mixes. Paper presented at the American Chemical Society National Meeting, Sulphur Use Symposium, Los Angeles, CA, April 1974.
9. J. Fenijn. Elemental Sulfur in Asphalt Paving Mixes. Paper presented to Canadian Sulfur Symposium, Calgary, 1974.
10. D. E. Carey. Sand-Asphalt-Sulphur Hot Mix. Louisiana Department of Transportation, Baton Rouge, Res. Rept. I, June 1977.
11. Societe Nationale des Petroles d'Aquitaine. Properties of Sulfur-Bitumen Binders. Paper presented at the 7th International Road Federation Meeting, Munich, Oct. 1973.
12. P. Vincent. Sulfur Asphalt Concretes. Paper presented at 167th American Chemical Society Meeting, Los Angeles, 1974.
13. B. M. Gallaway and D. Saylak. Sulphur/Asphalt Mixture Design and Construction Details—Lufkin Field Trials. Texas Transportation Institute, Texas A&M Univ., College Station, Jan. 1976.
14. G. J. A. Kennepohl, A. Logan, and D. C. Bean. Sulfur-Asphalt Binders in Paving Mixes. Proc., Canadian Sulfur Symposium, Calgary, 1974.
15. G. J. A. Kennepohl. The Gulf Canada Sulfur-Asphalt Process for Pavements. Paper presented at Symposium on New Uses for Sulfur and Pyrites, Madrid, 1976.
16. G. J. A. Kennepohl, A. Logan, and D. C. Bean. Conventional Paving Mixes with Sulfur-Asphalt Binders. Proc., Assn. of Asphalt Paving Technologists, Vol. 44, 1975.
17. T. W. Kennedy, R. C. Haas, P. Smith, G. J. A. Kennepohl, and E. T. Hignell. Engineering Evaluation of Sulfur-Asphalt Mixtures. TRB, Transportation Research Record 659, 1977, pp. 12-17.
18. F. R. P. Meyer, E. T. Hignell, G. J. A. Kennepohl, and R. C. G. Haas. Temperature Susceptibility Evaluation of Sulfur-Asphalt Mixtures. Proc., Assn. of Asphalt Paving Technologists, Vol. 46, 1977, pp. 452-480.
19. T. W. Kennedy, P. Smith, and R. C. G. Haas; Austin Research Engineers. An Engineering Evaluation of Sulphur-Asphalt Mixtures. Gulf Oil Canada, Rept. GC-1, June 1976.
20. D. D. Zakaib. Sulphur Asphalt Paving Technology. Gulf Canada Ltd., Pittsburgh, 1978.
21. G. J. Kennepohl and L. J. Miller. Sulphur Asphalt Binder Technology for Pavements. American Chemical Society, Washington, DC, 1978.
22. G. J. A. Kennepohl and R. Haas. Experience with Sulphur-Asphalt Paving Binders. Proc., International Conference on Sulphur in Construction, Ottawa, Sept. 1978.
23. R. C. G. Haas, G. J. A. Kennepohl, and D. C. Bean. Field and Laboratory Experience with Sulphur Asphalt Pavements. Paper presented at the International Conference on the Use of By-Products and Waste in Civil Engineering, Paris, 1978.
24. H. J. Fromm and G. J. A. Kennepohl. Sulphur Asphaltic Concrete, Three Ontario Test Roads. Proc., Assn. of Asphalt Paving Technologists, Vol. 48, 1979, pp. 135-162.
25. F. E. Pronk, A. F. Soderberg, and R. T. Frizzell. Sulphur-Modified Asphaltic Concrete. R. M. Hardy and Assoc. Ltd., Proc., 20th Annual Conference, Canadian Technical Asphalt Assn., Victoria, British Columbia, Vol. 20, 1975, pp. 135-194.
26. SUDIC: A Canadian Response to the Sulphur Challenge. British Sulphur Corp. Ltd., London, 1976.
27. W. C. McBee and T. A. Sullivan. Direct Substitution of Sulphur for Asphalt in Paving Materials. Paper presented at the 57th Annual Meeting, TRB, 1978.
28. B. P. Shields. Performance of Pavements with Sulfur Asphalt Binders/Alberta 1974-1976. Transportation and Surface Water Engineering Division, Alberta Research Council, Rept. HTE-77/01, 1977.
29. R. L. Lytton, D. Saylak, and D. E. Pickett. Prediction of Sulphur-Asphalt Pavement Performance with VESYS IIM. Proc., Fourth International Conference on Structural Design of Asphalt Pavements, Vol. 1, 1977, pp. 855-861.