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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.

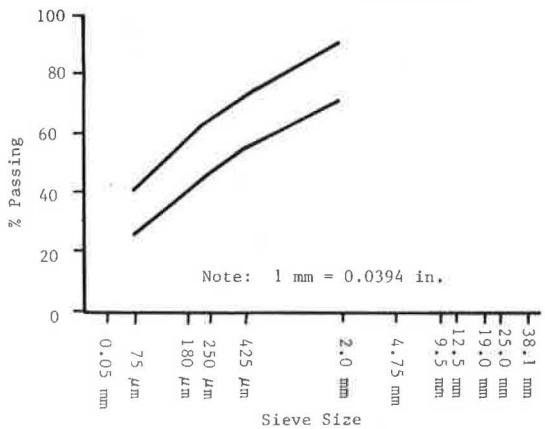


Figure 2. Typical SAHM base aggregate gradation.

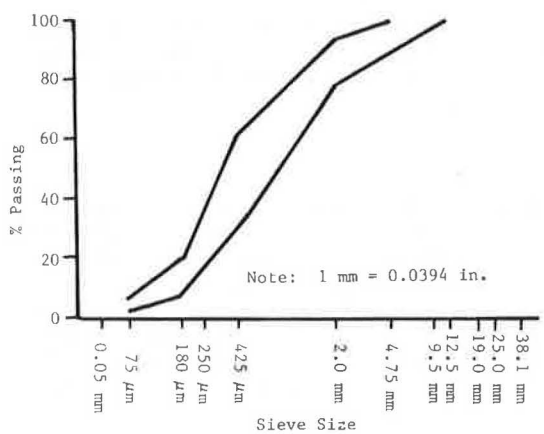


Table 1. Design characteristics—Marianna test road SAHM base thickness and stability.

Base Thickness (cm)	Section Stability	
	High (543 kg)	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/m^3)	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/m^3 = 62.43 lb/ft^3; 1 MPa = 145 lbf/in^2; 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/in^2).

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/in^2); 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/m³)	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/m³)	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/m³ = 62.43 lb/ft³; 1 MPa = 145 lbf/in²; 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/m³)	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/m³)	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/m³ = 62.43 lb/ft³; 1 MPa = 145 lbf/in².
[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.

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.

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 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

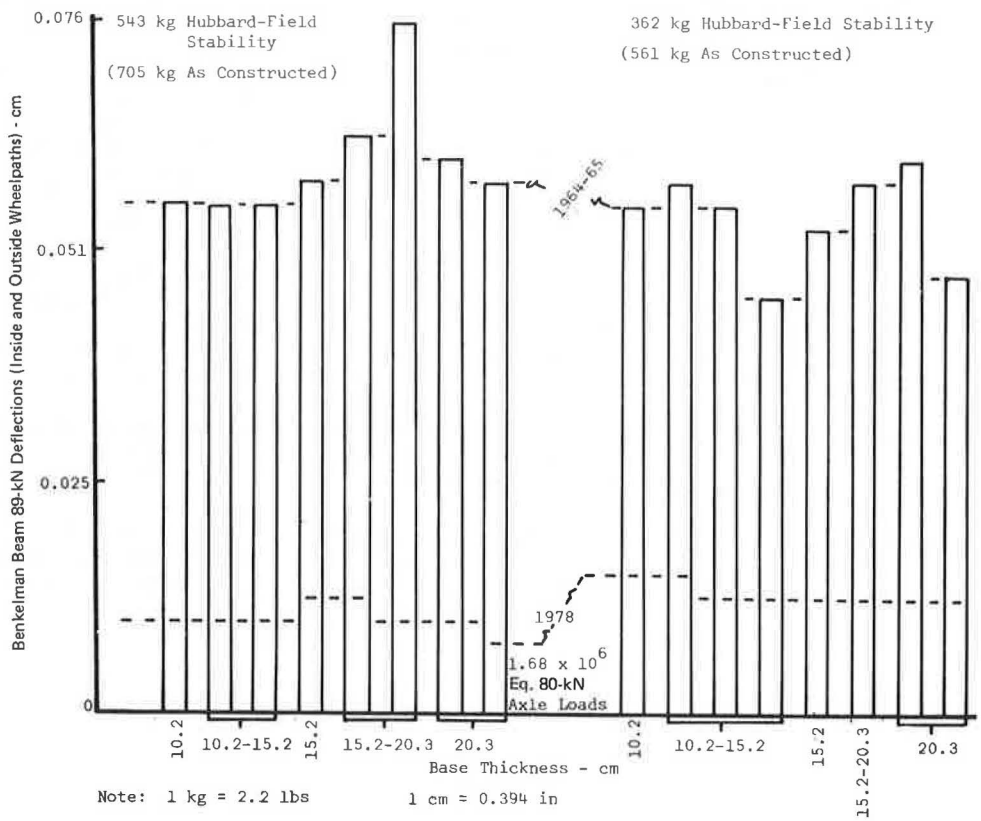


Figure 3. Marianna deflection comparison (westbound traffic lane).

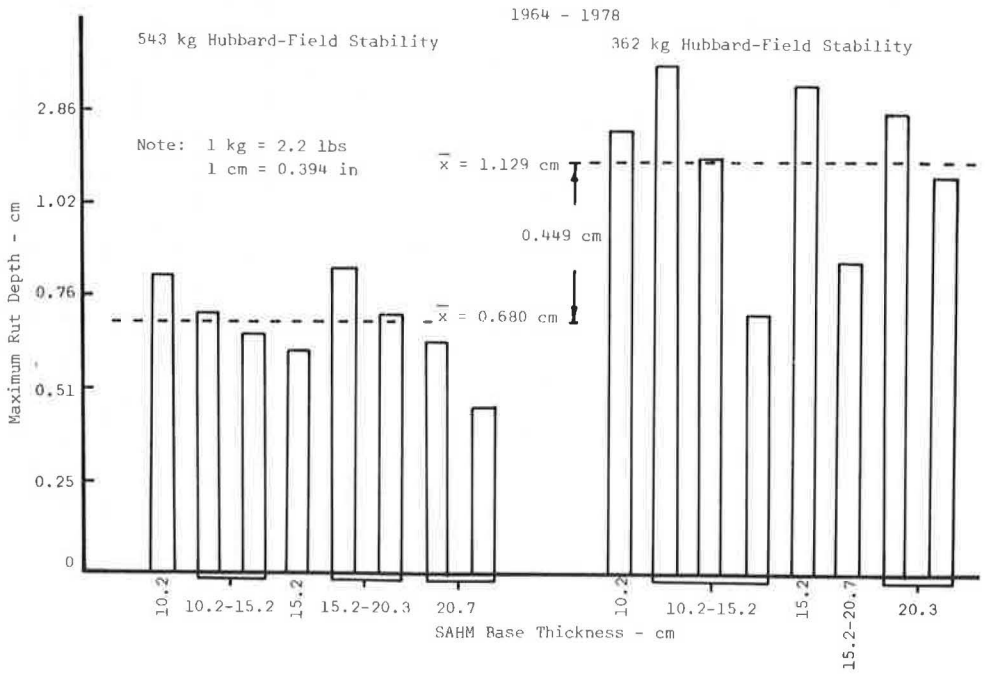


Figure 4. Marianna rut depth comparison (westbound traffic lane).

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).

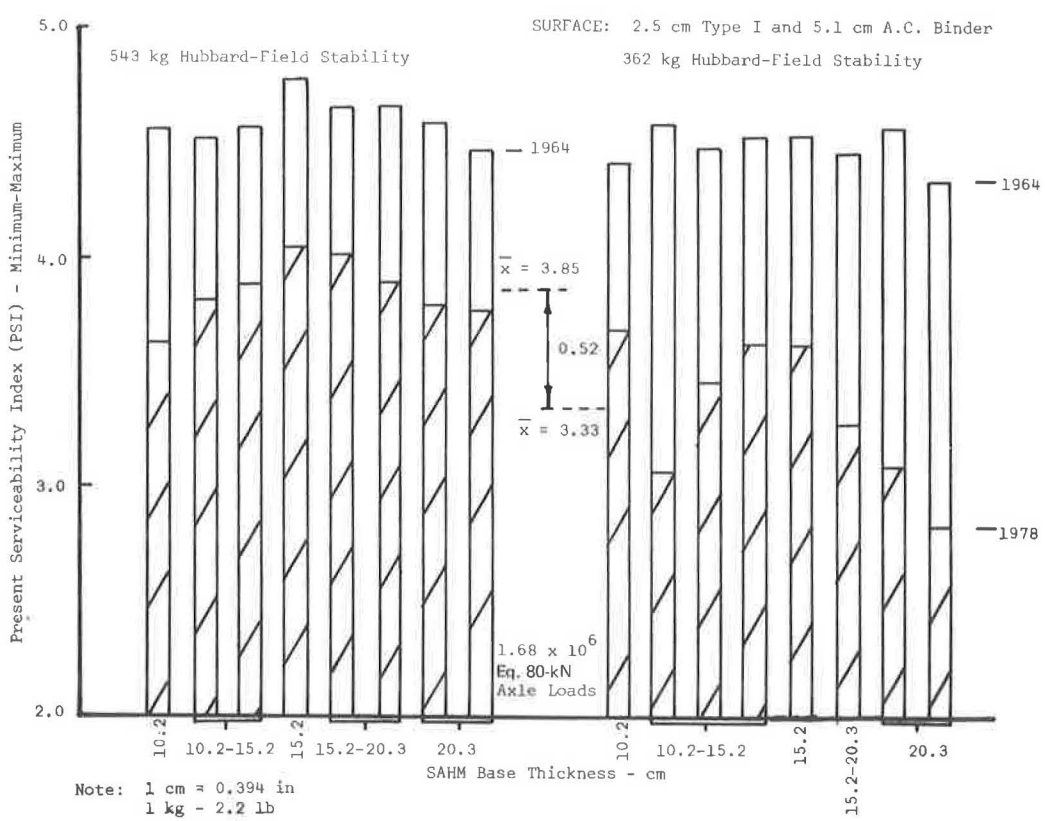


Figure 6. Palm Beach deflection comparison (eastbound and westbound lanes).

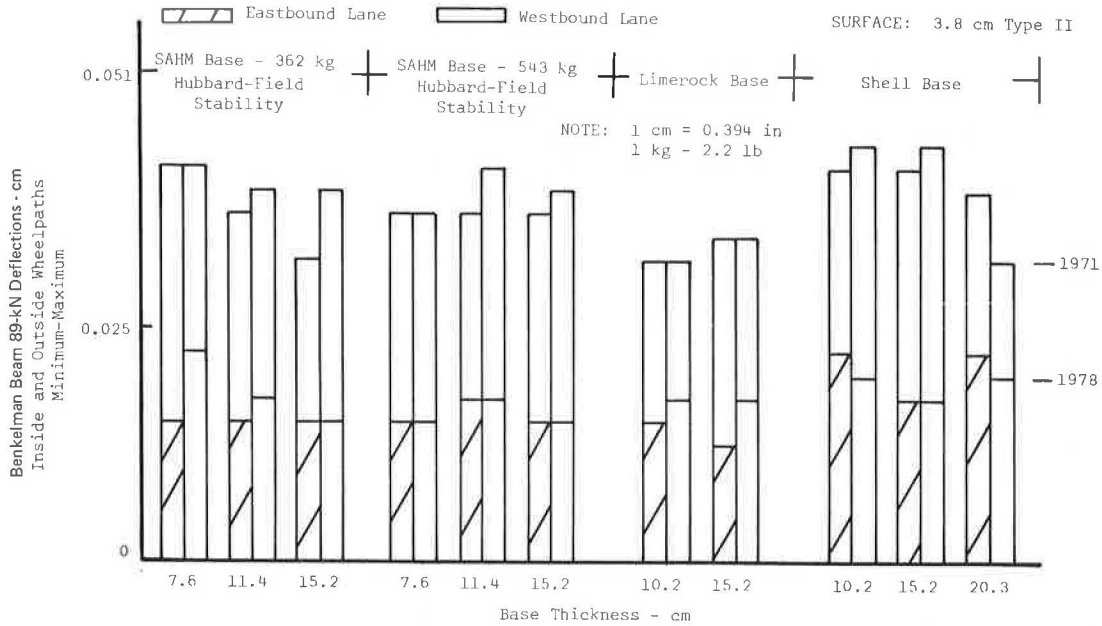
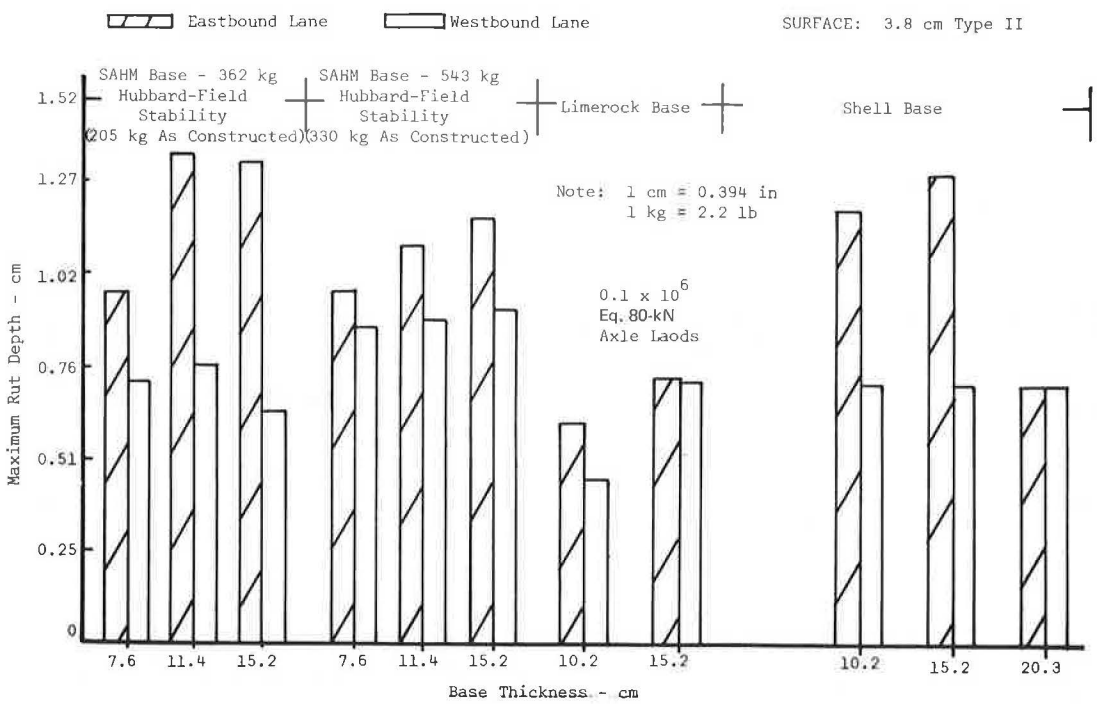


Figure 7. Palm Beach rut depth comparison (eastbound and westbound lanes).



(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).

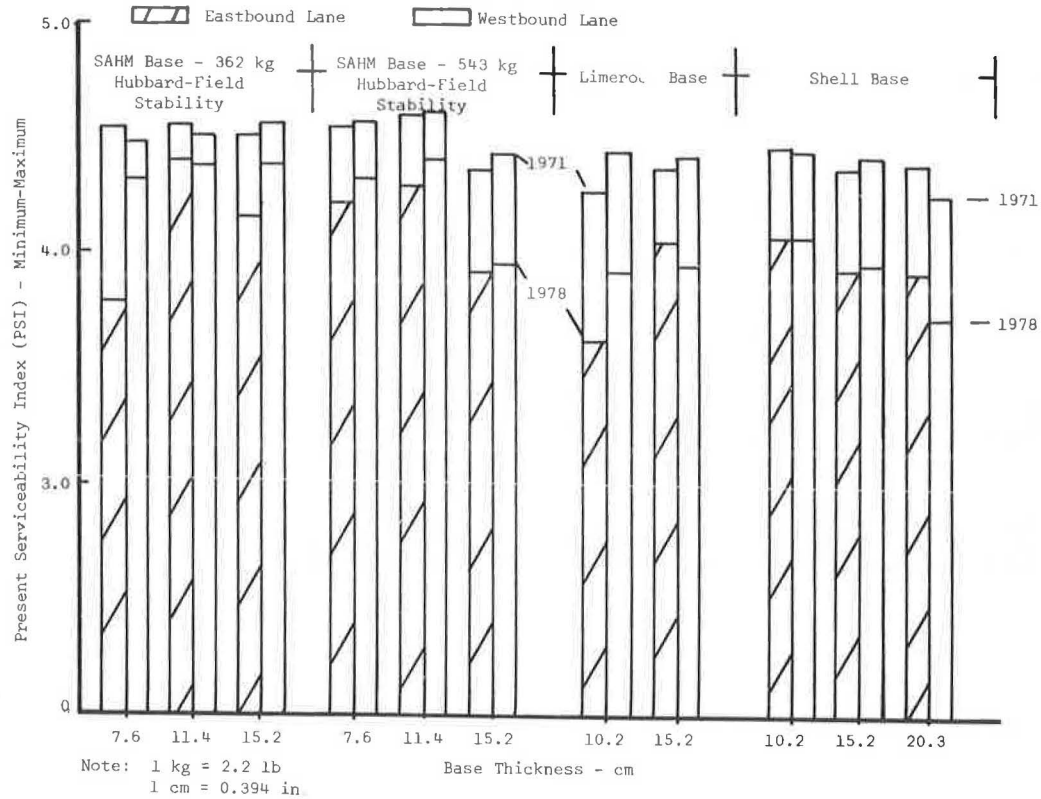
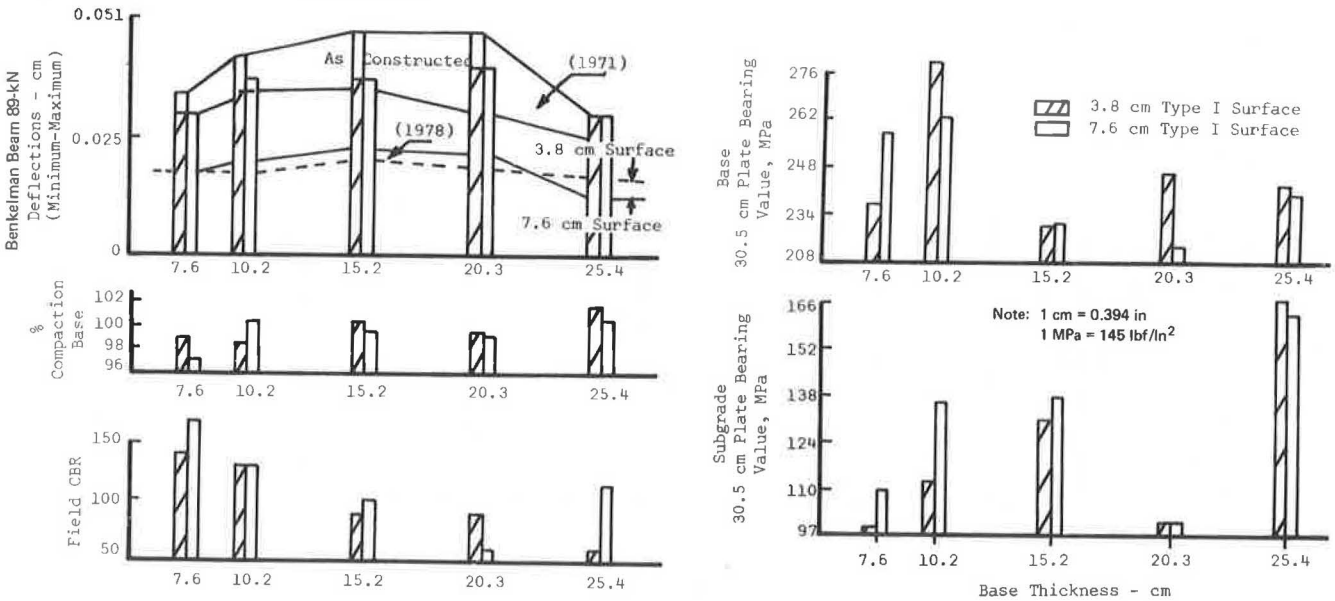


Figure 9. Lake Wales field data—limerock base.



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.

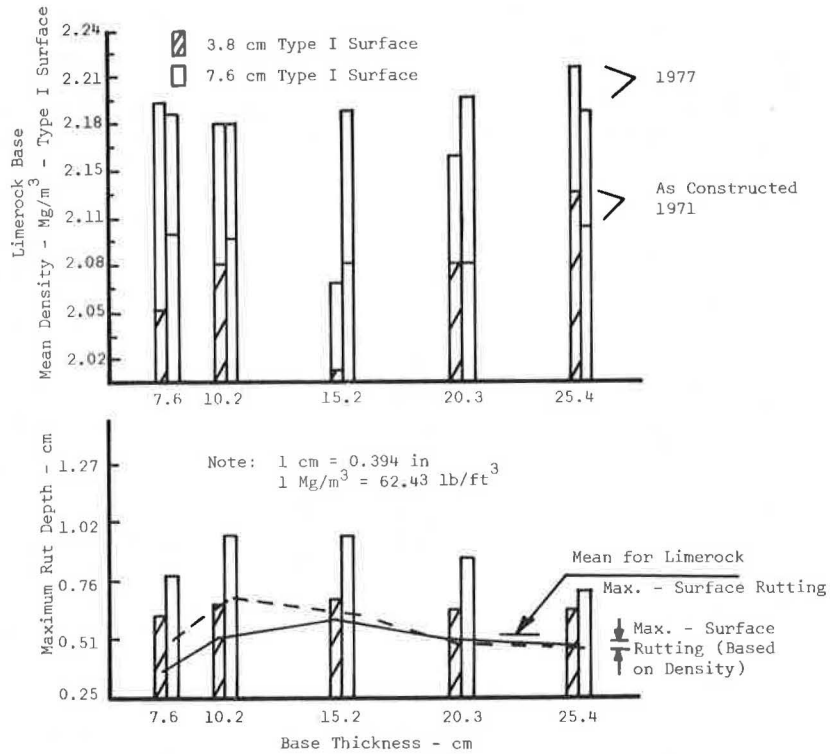
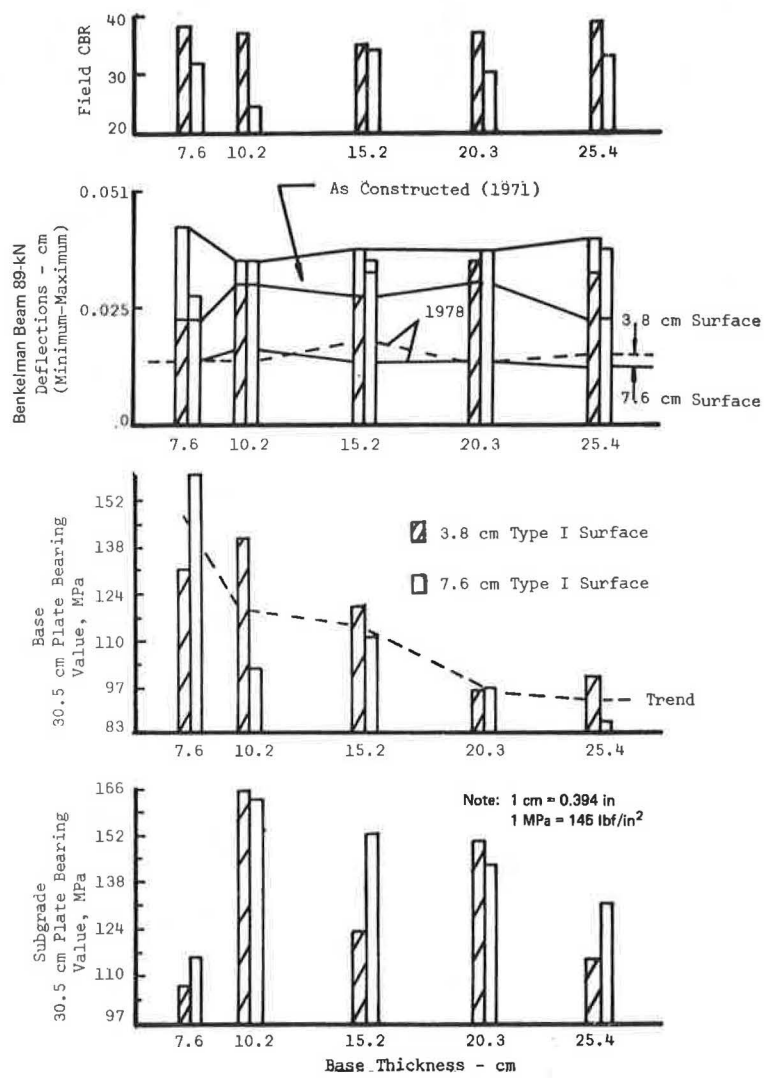


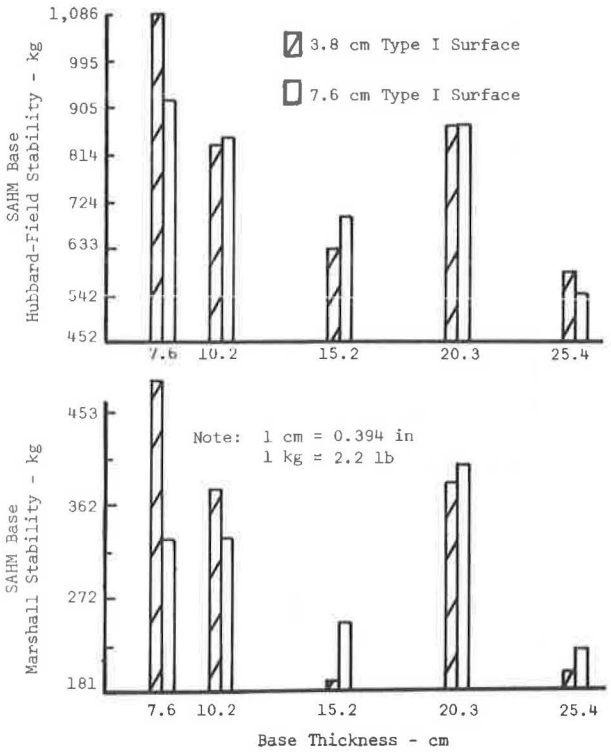
Figure 11. Lake Wales field data—SAHM base.



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

Figure 12. Lake Wales SAHM stability.



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.

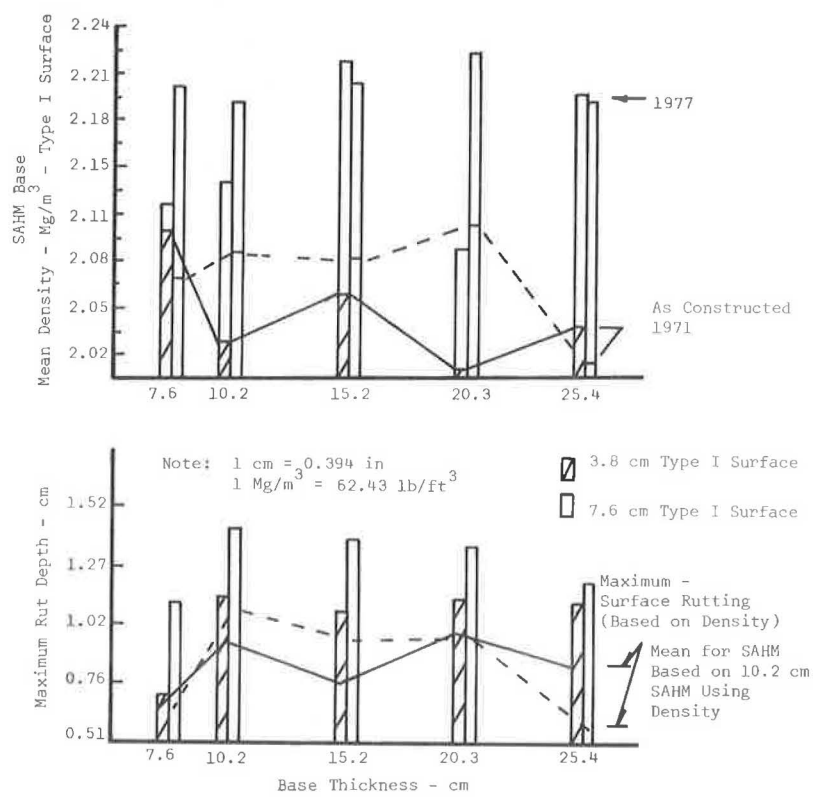
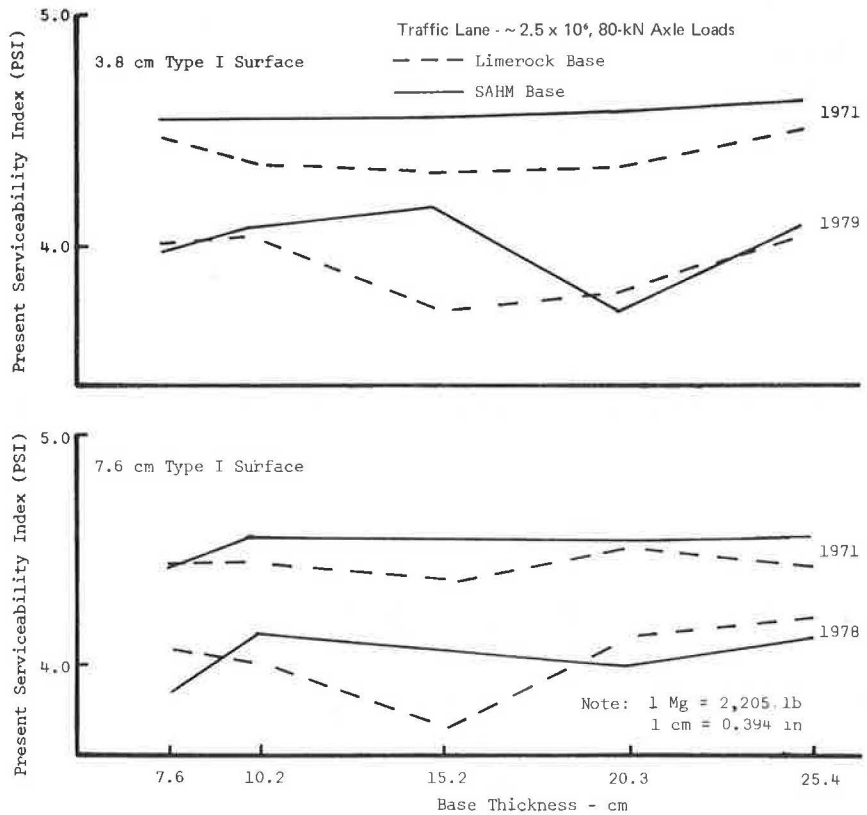


Table 9. Rut depth calculation results.

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 deflec-

tion 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.

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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)