Performance of Crushed-Stone Base Courses

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ABSTRACT

Twelve full-scale, instrumented pavement sections were tested to failure in a special laboratory facility under closely controlled environmental conditions. Seven of the pavement sections were loaded to more than 1 million repetitions and five of these sections to more than 2 million repetitions. A 6.5-kip (29-kN) uniform circular loading was applied to the surface and systematically moved to prevent a punching failure. Pavements tested consisted of five conventional sections having crushed-stone bases, five full-depth asphalt-concrete sections, and two inverted sections. The inverted sections consisted of a crushed-stone base sandwiched between a lower cement-stabilized layer and an upper asphalt-concrete layer. Conventional sections were tested with two thicknesses of base and three base gradations. The crushed-stone base sections were found to give excellent performance when covered with asphalt concrete 3.5 in. (89 mm) thick. Good performance of the engineered crushed-stone base is attributed to (a) a uniform, high degree of density (100 percent of AASHTO T-180), (b) use of a well-graded crushed stone with 1- to 2-in. (25- to 50-mm) top size that has only 4 to 5 percent passing the No. 200 sieve, (c) practically no segregation, and (d) a relatively thin asphalt-concrete surfacing.

The rising cost of petroleum products dictates the use of more materials that are low energy intensive and relatively inexpensive such as unstabilized crushed stone. There is therefore an important need to formally study the performance of granular bases in large-scale test sections taking advantage of recent advances in materials technology and modern instrumentation.

In this study the use of crushed-stone base is evaluated as an alternative to the deep strength asphalt-concrete construction now used by the Georgia Department of Transportation (GDOT) in flexible pavements. Twelve large-scale pavement sections were tested to determine whether engineered crushed stone can be successfully used to replace at least a portion of asphalt concrete in the base course. A summary of the full-depth asphalt-concrete, crushed-stone-base, and inverted sections tested is given in Table 1.

The inverted sections tested are not described in any detail; these results will be described in a subsequent paper. With the exception of test sections 3 and 4, between 0.15 and 4.4 million repetitions were applied to each section (Table 1). All tests were conducted in an enclosed constant-temperature environment at 78 to 80°F (25.6 to 26.7°C) over a period of 2 to 4 months.

TEST FACILITY

The pit in which the tests were performed was 8 ft (2.4 m) wide, 12 ft (3.7 m) long, and 5 ft (1.5 m) deep (Figure 1). To study a maximum number of base variables a different structural section was constructed at each end of the pit to give two tests for each complete filling of the pit. Emphasis was placed during construction of the test sections on achieving uniform material properties and meeting GDOT material specifications.

An air-over-oil cyclic loading system was developed to apply 6.5 to 7.5 kips (29.4 to 33.4 kN) to the pavement in 0.17 sec to simulate a slowly moving, heavy wheel loading. About 70 to 90 load pulses per minute were transmitted to the pavement surface through a water-filled circular rubber bladder. The diameter of the 6.5-kip load applied to the surface was 9.1 in. (231 mm). The resulting peak pressure was about 100 psi (689 kN/m²), uniformly distributed over the pavement surface. Loading was conducted 5 to 6 days a week, 24 hr a day.

Cyclic Loading System

In the hybrid air-over-oil pneumatic loading system, oil was sandwiched between a small [4 in. (102 mm) in diameter] free-floating aluminum piston on the top and a large [12 in. (305 mm) in diameter] aluminum piston on the bottom. Air pressure applied to the top of the small piston was transmitted undiminished to the large lower piston, giving a large force that was applied to the loading bladder resting on the pavement surface. A push rod transmits this force from the lower piston to the loading bladder. To develop the repeated loading, air was cyclically applied to the top of the upper cylinder by using a solenoid valve system and an electronic timer.

The bladder essentially consisted of a thin steel ring covered with a rubber diaphragm on the top and bottom. A constant seating loading was maintained between load applications to prevent a shock loading.

To prevent a localized punching failure from occurring during the test, the repeated loading was applied at a primary load position and six secondary positions located symmetrically around the edge of the primary position (Figure 2). In all tests load was applied in the ratio of five repetitions at the primary position to each repetition applied at any individual secondary position. The basic pattern was to apply 100,000 repetitions at the primary position and 20,000 repetitions to each secondary position; this number, however, was reduced in the early phases of most tests; in a few tests greater numbers of load repetitions were applied in the later phases of testing (1).

The pavement was subjected to a cyclic loading of 6.5 kips up to 2 x 10^6 repetitions. To decrease the time required to cause the failure of strong pavement sections, the loading was increased to 7.5 kips after 2 x 10^6 repetitions. In sections 11 and 12 the load was maintained at 6.5 kips throughout the test. In this series, however, 200,000 load rep-

TABLE 1 Construction and Performance Summary of Pavement Sections Tested

Section Number	Asphalt-Concrete Thickness (in.)	Crushed-Stone Thickness (in.)	Repetitions to Failure (000,000s)	Failure Mode[a]	Comments
1	3.5	12.0	3.0 3.5	Fatigue and rutting	Tested to 2.4 million repetitions; failure extrapolated
2	3.5	8.0	1.0	Rutting	–
3	9.0	None	0.010	Rutting (1 in.)	Bad asphalt: asphalt-concrete content, 5.9 percent; flow, 15.4; stability, 1,870 lb; γ_D = 145.1 pcf
4	6.5	None	0.010	Rutting (1 in.)	
5	9.0	None	0.13	Rutting	Rutting failure primarily in asphalt concrete
6	6.5	None	0.44	Rutting	
7	7.0	None	0.15	Rutting	Direct comparison of crushed-stone and full-depth sections
8	3.5	8.0	0.55	Rutting	
9	3.5	8.0[b]	2.4	Fatigue	δ_p = 0.28 in. at 2.9 million repetitions
10	3.5	8.0[c]	2.9	Fatigue	δ_p = 0.34 in. at 2.9 million repetitions
11	3.5	8.0	3.6	Fatigue and rutting	6.0-in. soil cement subbase[d]
12	3.5	8.0	4.4	Fatigue and rutting	6.0-in. cement-stabilized stone subbase[e]

Note: 1 in. = 25 mm. 1 lb = 0.45 kg. 1 pcf = 16 kg/m^3. 1 psi = 6.89 kN/m^2.
[a]A fatigue failure is defined as class 2 cracking; a rutting failure is defined as a 0.5-in. (12-mm) rut depth.
[b]Coarse-gradation base.
[c]Fine-gradation base.
[d]28-day unconfined compressive strength of 214 psi.
[e]28-day unconfined compressive strength of 1,146 psi.

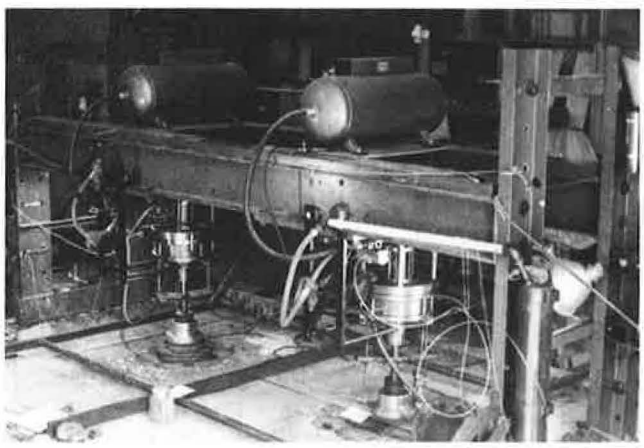

FIGURE 1 General view of pit test facility including load frame, air-over-oil loading system, and loading bladder.

etitions were applied at the primary load position in the latter stages of testing; use of this pattern of loading greatly increased the number of repetitions that could be applied during a given 24-hr day.

Reaction Frame

A reaction frame extended horizontally across the pit in the long direction about 3 ft (0.9 m) above the surface of the pavement (Figure 1). The loading system was attached to the load frame by means of a horizontally oriented thrust plate 1 in. (25 mm) thick and 26 x 33 in. (660 x 838 mm) in size. Rapid positioning by hand of the loading system at seven fixed load locations on the thrust plate was achieved by using seven sets of bolt holes to support the load system. Movement of the load cylinder from one fixed load position to another was easily accomplished by a special carriage that hung from the reaction beams. The carriage rolled along the reaction beam on four wheels and temporarily supported the load cylinder during movement.

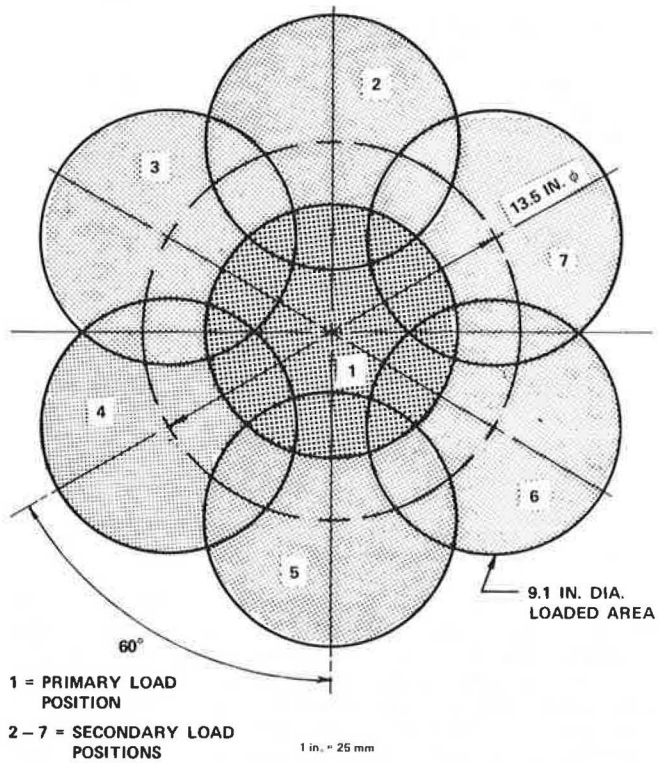

FIGURE 2 Load positions used in test section loading.

Instrumentation

The test sections were extensively instrumented to define the response of the pavement system. Typical-

ly 19 to 24 Bison-type strain sensors were installed to measure both resilient and permanent deformations throughout the pavement section (Figure 3). Small diaphragm-type pressure cells were used to measure vertical stress on the subgrade. Resilient surface deflections were measured by using linear variable differential transducers (LVDTs). Permanent deformations of the pavement surface were measured from a string line by using a metal scale. All instrumentation was carefully calibrated. A detailed description of the instrumentation is given elsewhere (1).

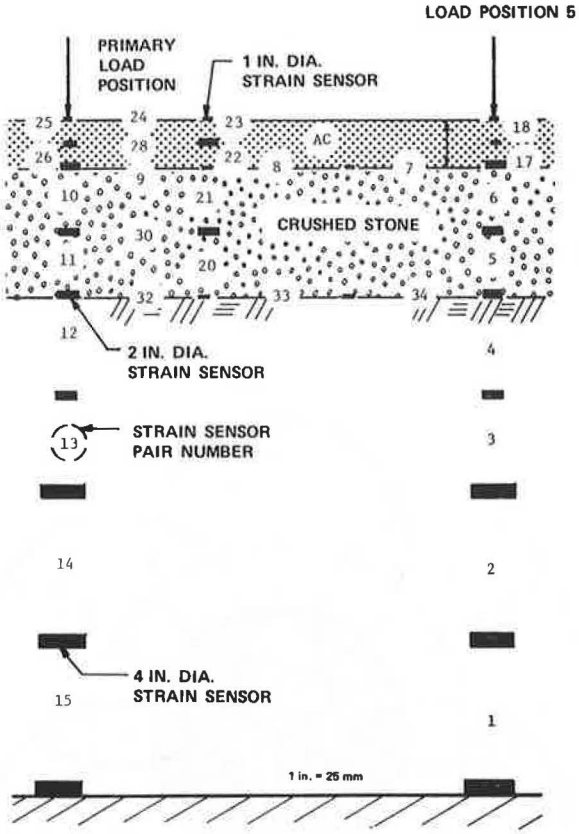

FIGURE 3 Bison strain sensor layout used in section 8, crushed-stone base.

TEST-SECTION CONSTRUCTION AND MATERIAL PROPERTIES

All test sections were constructed by using the same standardized procedures found to give consistent, reproducible results. After being tested to failure, each section was completely removed from the pit and new sections were constructed from the bottom of the subgrade up. Only the silty-sand subgrade soil was reused; after each test the subgrade soil was removed from the pit, stored, remixed, and then placed and recompacted in the pit.

Subgrade

The micaceous silty-sand subgrade was uniformly blended in a small Barber-Green pugmill in small batches. Before blending, the material was weighed, and the water content of each batch was determined by using a Speedy moisture meter. During mixing, the water required to bring the moisture content to optimum was added. The soil subgrade was placed in approximately 2-in. (51-mm) lifts and compacted with five to seven passes of a Jay-12 vibrating base compactor. A Wacker compactor was also sometimes used. A spring-loaded static penetrometer helped in controlling the density of each soil lift; the actual density was determined by using a thin-wall drive-tube sample. The subgrade of all sections had a uniform dynamic cone penetration resistance equivalent to a standard penetration test value of seven to eight blows per foot (23 to 26 blows per meter).

A uniform micaceous silty-sand subgrade about 50 in. (1270 mm) thick was used in all tests. The average maximum dry density (AASHTO T-99) was 105 pcf (16.5 kN/m³), and the optimum water content was about 18.5 percent. The subgrade was compacted in lifts 2 in. thick to an average of 98 percent (σ = 1.5 percent) of AASHTO T-99 density at a moisture content of 20.4 percent (σ = 0.5 percent), which was 2 percent above the optimum value. The micaceous silty sand had an average GDOT volume change of 38.5 percent and a clay content of 20 percent. It was nonplastic and had 85 percent passing the No. 40 sieve and 39 percent passing the No. 200 sieve. The micaceous silty sand had a laboratory resilient modulus of about 900 psi (6200 kN/m²) for a deviator stress greater than about 2.5 psi (17.2 kN/m²), as shown in Figure 4; confining pressure and moisture

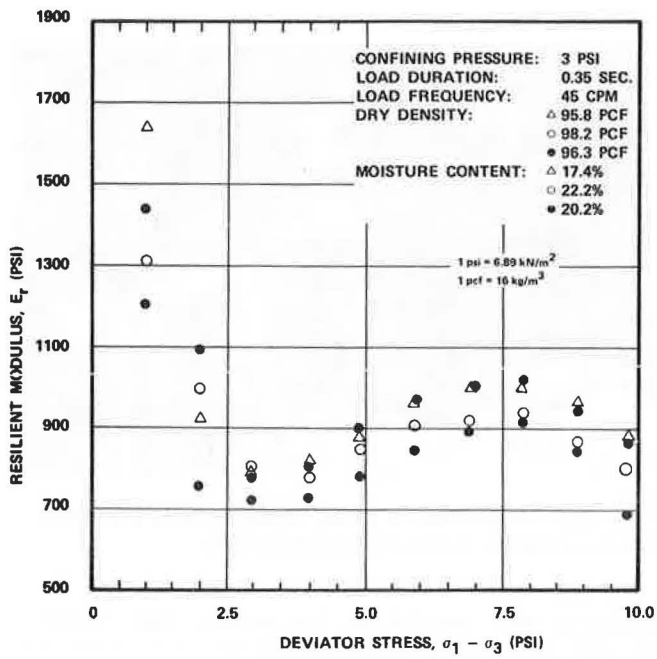

FIGURE 4 Influence of deviator stress and moisture content on resilient modulus of micaceous silty-sand subgrade.

content had only a minor effect on the resilient modulus. A low resilient modulus typically from 1,000 to 3,000 psi (6890 to 20 000 kN/m²) is common in highly micaceous Piedmont soils.

Crushed-Stone Base

The crushed-stone base was constructed by blending together in the pugmill No. 5, No. 57, and No. 810 crushed granite gneiss stone to give the desired gradations. After pugmilling, the stone was bottom-dumped into a special bucket. The bucket was moved by a crane to the desired location and once again bottom-dumped. Use of separate sizes of stone, pug-

milling, and bottom-dumping resulted in a uniform, homogeneous blend having a minimum amount of segregation after placement.

The sand-replacement method was used to determine the density of test sections 1 and 2. This method caused excessive disturbance of the crushed-stone base. The density of all subsequent unstabilized granular bases was therefore determined by a GDOT inspector by using a nuclear density gage. The average density of all sections (except 11 and 12) was 100 percent of AASHTO T-180; little variation was observed between sections. The unstabilized crushed-stone bases used in the inverted sections (11 and 12) were constructed over a rigid cement-stabilized layer. As a result, the density obtained in the crushed stone in these bases was greater than that in the other granular base sections, 105 percent of AASHTO T-180.

Stone gradations used are given in Table 2. The resilient response of the standard-gradation stone base is given in Figure 5 and the plastic response in Figure 6.

Asphalt Concrete

Either a GDOT B or a GDOT modified-B binder was used for the full thickness of the asphalt-concrete layer. The asphalt concrete was transported from the plant to the test facility in an enclosed, heavily insulated plywood box. At the time of delivery to the test facility, the temperature of the asphalt was between 290 and 300°F (143 to 149°C). The asphalt concrete was quickly weighed out, placed in the pit, leveled, and compacted by using a small two-wheel vibrating maintenance roller. The asphalt concrete was placed in lifts about 1.75 in. (44.4 mm) thick and rolled in each direction. A light prime coat of RC-70 was sprayed on the surface of the stone before the asphalt concrete was placed.

The mix designs and extraction test results are shown in Table 3. The stone used was a granite gneiss (obtained from two different quarries) that had the following typical gradation as defined by the extraction tests: 97 to 100 percent passing the 1-in. (25-mm) sieve, 62 to 85 percent passing the 3/8-in. (9.5-mm) sieve, 48 to 60 percent passing the No. 4 (4.75-mm) sieve, and 4 to 8 percent passing the No. 200 (75-μm) sieve.

TEST SECTION FINDINGS

General Comparison

Table 1 gives a general summary of the performance of the sections tested in this study. Sections 3 and 4 failed prematurely by rutting because of a high asphalt content. Both rutting and fatigue failures occurred in the tests. Sections 1, 11, and 12 failed in a combined fatigue and rutting mode. (Section 1 was tested to 2.4 million load repetitions; failure would have occurred at about 3 million to 3.5 million repetitions.) Sections 2 through 8 failed in rutting, and sections 9 and 10 failed in fatigue.

A fatigue failure was defined as the initiation of class 2 cracking. Only fine hairline cracks, which were hard to see, developed in sections considered to have undergone a fatigue failure. Numerous instances of the healing of these cracks were observed throughout the study when the load was placed over them.

A rutting failure was defined as an average rut depth of 0.5 in. (12 mm) measured from a fixed string line; rutting occurring during the first 1,000 repetitions was not included. Of significance is the finding that the sections surviving more than 2 million repetitions failed in fatigue or were close to a fatigue failure. In contrast, sections having relatively short lives failed in rutting. Thus, these test results are a good illustration of the importance of preventing excessive rutting in all layers.

The relatively early failure of the full-depth sections (sections 5-7) in rutting compared with the crushed-stone base sections (sections 1, 2, and 8-10) appears to be caused by (a) application of the heavy loading in a reasonably concentrated pattern, (b) important contributions of rutting from the weak subgrade beneath the relatively thin, full-depth as-

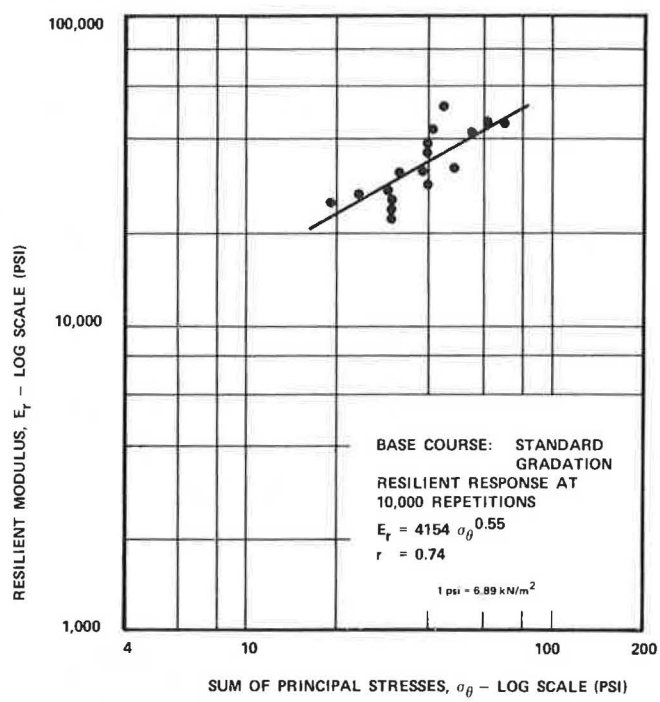

FIGURE 5 Influence of stress state on resilient modulus of crushed-stone base material, standard gradation.

TABLE 2 Crushed-Stone Base-Course Gradations and AASHTO T-180 Maximum Dry Densities

Base	Cumulative Percent Passing by Sieve Size											γ_{max}[b] (pcf)
	1 1/2 in.	1 in.	3/4 in.	3/8 in.	No. 4	No. 8	No. 10	No. 50	No. 60	No. 100	No. 200	
Standard[a]	100	98	83	61	43		31	13			4	137.0
Fine[a]	100	99	92	64	44		31		11		4	139.5
Coarse[a]	98	83	69		40	31	29		10		4	141.5
2-in. Fuller curve	98	82	69		39	31	29		12		4	–

Note: 1 in. = 25 mm. 1 pcf = 16 kg/m³.

[a]The standard-gradation base was used in sections 1, 2, 8, 11, and 12; the fine-graded base in section 10; and the coarse-graded base in section 9.

[b]AASHTO T-180 maximum dry density.

phalt concrete (the subgrade was located closer to the surface than the crushed-stone section and apparently as a result made a larger contribution to rutting), and (c) slightly higher asphalt content of the full-depth sections than that of the crushed-stone base sections (Table 3).

Permanent Deformation

In each test section, the pavement surface beneath the center of the primary load position underwent a continual up-and-down movement of permanent deformation (Figure 7). The surface underwent permanent downward movement when the load was in the primary load position and upward movement when the load was in a secondary position. This cyclic movement was caused by important lateral shear flow of material in all layers back and forth beneath the loaded regions. Net movement of the center (primary load position) was gradually downward as shown in Figure 7.

The distribution of permanent deformation in a full-depth and a crushed-stone base section is compared in Figure 8 after 300,000 load applications. In the asphalt-concrete layers of both sections the permanent deformation was approximately equally distributed between the top and bottom halves of the asphalt-concrete layer. In the full-depth asphalt-concrete section, 67 percent of the total permanent deformation occurred in the asphalt-concrete and 33 percent in the subgrade. In the crushed-stone section, 55 percent of the permanent deformation occurred in the asphalt-concrete surfacing, which was 3.5 in. (89 mm) thick; 10 percent in the crushed stone; and 35 percent in the subgrade. At equal depths more permanent rutting appeared to occur in the upper part of the subgrade beneath the full-depth asphalt-concrete section than in the crushed-stone section.

Going from a 12-in. (305-mm) crushed-stone base to an 8-in. (203-mm) crushed-stone base resulted in an increase in percentage of total rutting in the subgrade from 20 to 39 percent. At the same time, the percentage of rutting in the asphalt surfacing dropped from 59 to 34 percent; rutting in the stone base only increased from 21 to 27 percent of the total. Both this comparison and the previous one indicate that rutting can be relatively small in a properly designed and constructed crushed-stone base under conditions in which water is not a problem in the base. As illustrated in Figure 9, for the crushed-stone bases studied, typically 60 to 70 percent of the rutting occurring in the stone base developed in the upper half of the base. Finally, little difference in rutting was observed within the base between the sections having the coarse- and fine-gradation stone bases (sections 9 and 10) as shown in Figure 9. The gradation of both these sections below the No. 40 sieve was, however, essentially the same; only 4.2 to 4.4 percent passed the No. 200 sieve (Table 2). More total rutting did develop in the fine-gradation base section (section 10) than in the coarse base section. The difference, however, was primarily caused by rutting in the subgrade.

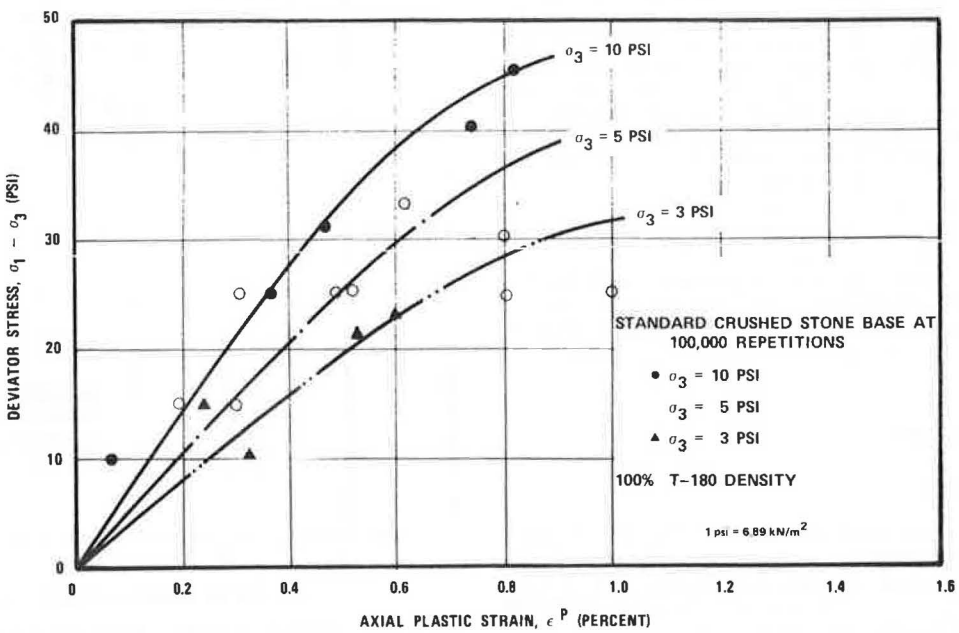

FIGURE 6 Influence of deviator stress and confining pressure on plastic strain in standard-gradation crushed-stone base.

Resilient Response

Resilient strains were measured by using Bison strain coil pairs (Figure 3). Hence the resilient strains given are the average strain occurring between the two coils and not a peak (maximum) value that occurs at a point; the difference, however, between the two values should be relatively small.

The typical variation of resilient strain response with number of load repetitions is given in Figure 10 for section 10; as indicated, the strains typically decreased after 10,000 to 100,000 repetitions. They then underwent an important increase after about 1 million load applications, which indicated reduction in the structural integrity of the pavement.

Comparison of Theory and Observed Response

A comparison of calculated and typical measured resilient response is given in Table 4. The response

TABLE 3 Fifty-Blow Marshall Mix Designs for Test Sections: AC-20 Asphalt Cement

Section	50-Blow Marshall Mix Design[a]					Constructed	
	Optimum Asphalt Concrete (%)	Flow (0.01 in.)	Stability (lb)	Air Voids (%)	γ (pcf)	Asphalt Concrete (%)	γ (pcf)
1,2	4.9	9.6	2,500	4.5	146.0	4.61	143.7
3,4	4.9	9.6	2,500	4.5	146.0	5.9	145.1
5,6	5.2	10.6	2,150	4.5	145.5	5.44[b]	145.3
7,8	5.2	10.6	2,150	4.5	145.5	4.92[c]	145.0
9,10	5.0	10.1	2,950	5.0	148.0	4.90	148.0
11,12	5.2	9.0	2,300	4.0	146.0	–	99.2[c]

Note: 1 in. = 25 mm. 1 lb = 0.45 kg. 1 pcf = 16 kg/m^3.
[a]Resilient modulus of 70,000 psi, 95°F, sections 7 and 8; bending stiffness of 380,000 psi, 80°F, sections 1 and 2; both tests on reheated asphalt concrete.
[b]Section 5 had a measured asphalt content of 5.71 percent and section 6, 5.17 percent; section 7 had 5.60 percent and section 8 had 4.23 percent; the actual difference in asphalt-concrete content was probably less because each pair of sections was constructed at the same time.
[c]Percent maximum from nuclear-density tests.

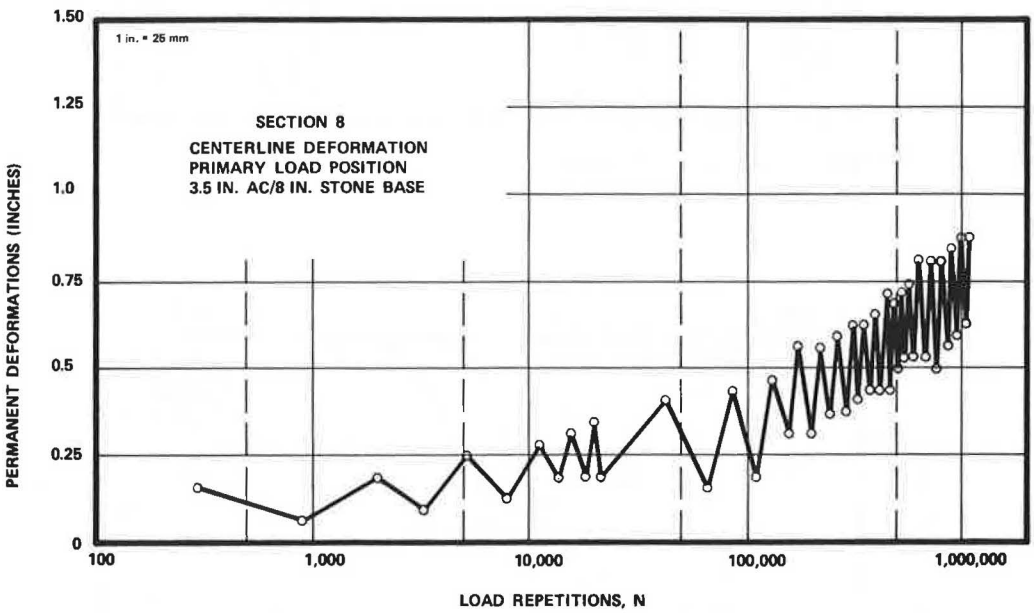

FIGURE 7 Variation of centerline deformation with number of load repetitions, section 8.

was calculated by assuming the layers to be isotropic, linear-elastic, homogeneous, and semiinfinite in horizontal extent (2). Linear-elastic theory of this type is usually used in mechanistic design methods. Because pavements are neither linear elastic nor isotropic, a good match of all measured variables should not be expected.

The laboratory-measured resilient moduli were adjusted to give an approximate best overall fit of the observed strain, deflection, and stress response of the pavement systems. For a reasonably good overall fit of the observed response, the resilient moduli of the subgrade, crushed-stone base, and surfacing were taken to be about 2,800 psi (19.3 MN/m²), 15,000 psi (103 MN/m²), and 4 to 8 x 10⁵ psi (2756 to 5512 MN/m²), respectively.

The moduli values used generally give deflections that are somewhat larger than those measured but give a reasonably good prediction of the tensile strain measured in the bottom of the asphalt-concrete surfacing.

The theoretical vertical stress on the subgrade is considerably smaller than the measured values. Some overmeasurement of vertical stress may certainly have occurred in the investigation. The pressure cells, however, were calibrated in the same subgrade soil, and the effect of dynamic loading was found by calibration to be small. Therefore it is felt that the existing stresses were indeed greater than those predicted by theory. The finding (Table 4) that the measured vertical strains on the subgrade for the full-depth and stone base sections were about 50 percent greater than predicted also tends to indicate larger subgrade stresses.

The resilient modulus used to characterize the subgrade was about three times the value indicated by the repeated-load triaxial tests. Disturbance and remolding effects and the short time for which the laboratory test specimens are subjected to σ_3 may partially account for this important difference. Previous work with the micaceous silty sands of the Piedmont have indicated similar problems with laboratory-evaluated moduli (3).

Fatigue Behavior

Crushed-stone base sections 9 through 12 failed in fatigue or a combined fatigue-rutting failure. The fatigue relationship obtained from these data points

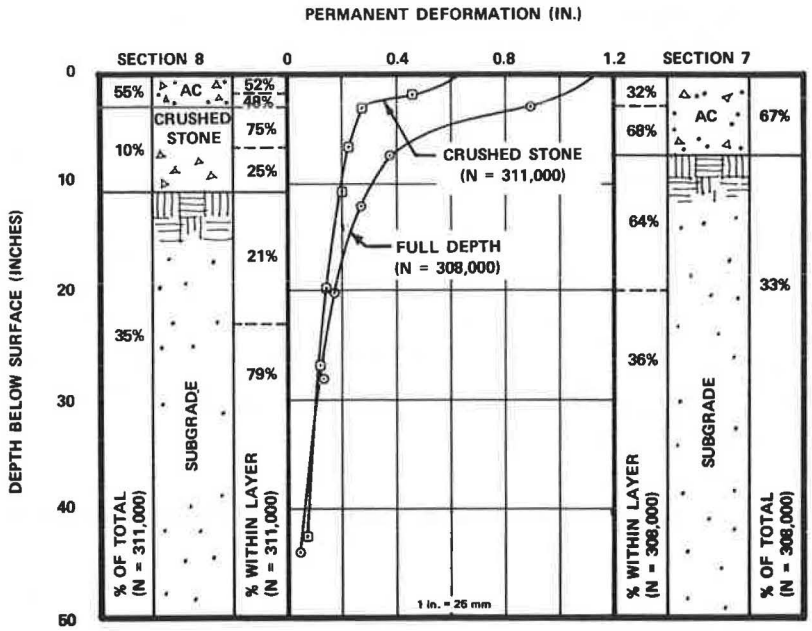

FIGURE 8 Comparison of distribution of permanent deformation in full-depth section 7 and crushed-stone section 8.

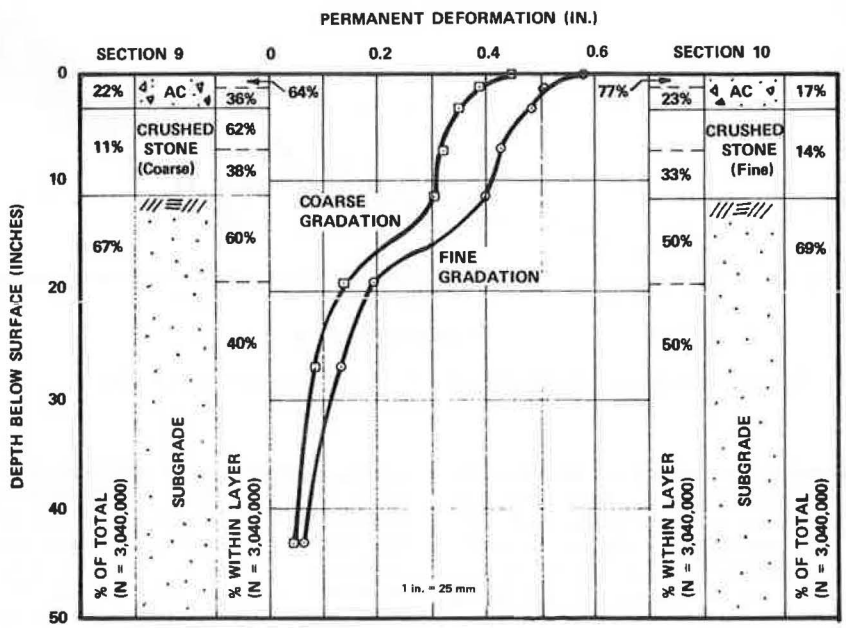

FIGURE 9 Comparison of distribution of permanent deformation in coarse- and fine-gradation crushed-stone base pavements, sections 9 and 10.

for 79°F (26.1°C), $N_f = 0.00108\varepsilon_t^{-2.695}$, was found where N_f is the number of repetitions to cause fatigue failure and ε_t is the tensile strain in the bottom of the asphalt-concrete layer. This fatigue relationship is for a temperature of 79°F, fine hairline cracking, and the assumption that each load application caused the same amount of damage. The relatively concentrated pattern of loading is felt to be more severe than would normally occur for a highway pavement.

The observed fatigue curve, corrected to 70°F (21.1°C), is located above the fatigue curves summarized by Rauhut and Kennedy (4), as shown in Figure 11. The high fatigue curve is probably partly caused by use of a thin bituminous surfacing and a high-quality crushed-stone base. The points fall between the curves developed by Barksdale (5) for a 3-in. and 9.8-in. thickness of asphalt-concrete surfacing.

Influence of Crushed-Stone Base Thickness

Increasing the thickness of the crushed-stone base from 8 in. to 12 in. increased the life of the pavement by a factor of almost 3 (compare sections 1 and

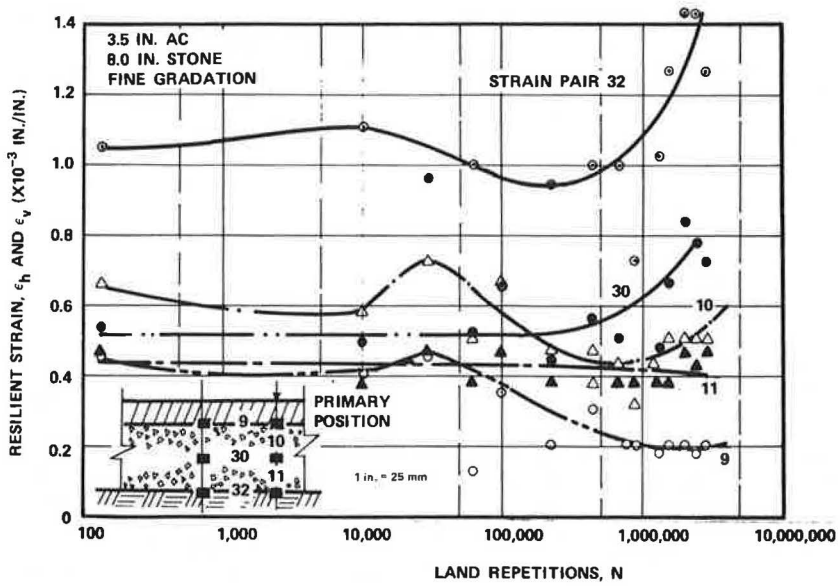

FIGURE 10 Variation of resilient strain with load repetitions, section 10.

TABLE 4 Comparison of Measured and Theoretical Stresses, Strains, and Deflections by Using Final Material Parameters

Pavement Design	Vertical Stress on Subgrade (psi)		Tangential Strain at Bottom of Asphalt Concrete ($\mu\epsilon$)		Surface Displacement (mils)		Vertical Subgrade Strain ($\mu\epsilon$)	
	Typical Measured	Theoretical	Typical Measured	Theoretical	Typical Measured	Theoretical	Typical Measured	Theoretical
Full-depth (9-in.) asphalt concrete	8.7	2.5	308	280	14	17	1,380	852
Asphalt concrete (3.5 in.) on 8-in. crushed-stone base	6.8-11.2	4.0	270-390	352	23	29	2,030	1,450
Asphalt concrete (3.5 in.) on 8.7-in. crushed-stone base on 5.8-in. cement-treated crusher run	3.5	2.1	272	262	17	15	340	463
Asphalt concrete (3.7 in.) on 8.9-in. crushed-stone base on 6.0-in. cement-treated subgrade	3.2	2.5	324	276	26	19	390	649

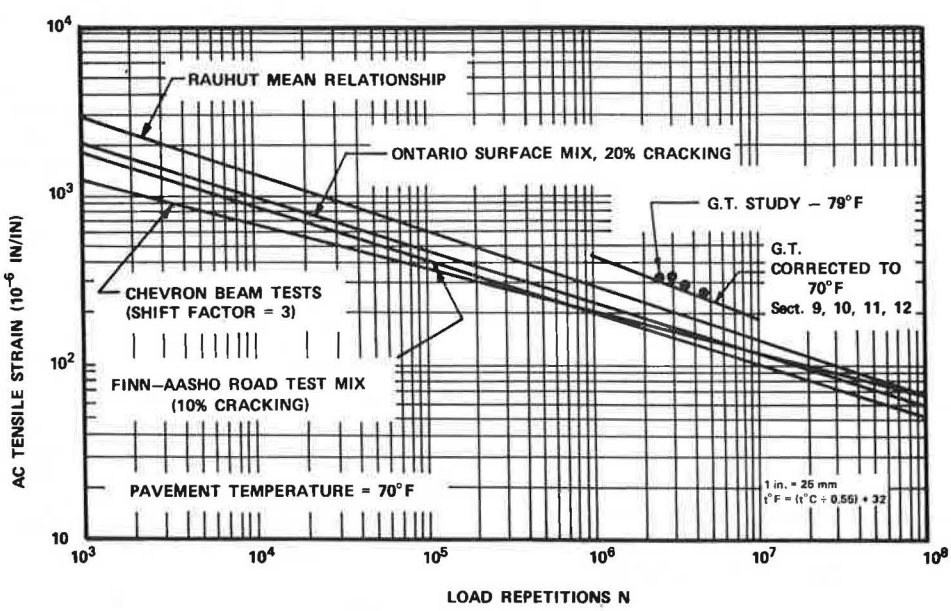

FIGURE 11 Comparison of observed fatigue performance with relationships summarized by Rauhut and Kennedy *(4)*.

2, Table 1). The AASHO Road Test results also indicated a similar significant beneficial effect of a small increase in base-course thickness (6). As the base thickness becomes greater, however, the beneficial effect of increasing thickness probably decreases, as indicated by an analytical study (1).

Influence of Crushed-Stone Base Gradation

Excellent performance was obtained from the granular-base pavements that had both the coarse gradation (section 9) and the fine gradation (section 10). Both these sections failed in fatigue rather than in rutting at a higher number of repetitions than two other 8-in. crushed-stone base pavements (sections 2 and 8). The fatigue life of the fine-gradation base section was about 20 percent greater than the fatigue life of the coarse base section. On the other hand, rutting in the fine-gradation base section was 21 percent greater than in the section having a coarsely graded granular base section. These differences are reasonably minor considering the possible variation.

The somewhat limited test results indicate for the relatively narrow range of gradations tested that gradation has a reasonably minor influence on performance provided the section is compacted to 100 percent of AASHTO T-180 density and little segregation is allowed to occur. All three crushed-stone base gradations, however, had a top size of 1 to 2 in. (25 to 51 mm), 40 to 44 percent passing the No. 4 sieve, and 4 to 5 percent fines.

Influence of Asphalt Content

Permanent deformation in the B and modified-B binder mixes was found to increase dramatically as the asphalt content increased, as shown in Figure 12, which is based on the permanent deformation occurring in the upper 3.5 in. (89 mm) of both full-depth and crushed-stone base sections at 10,000 load repetitions. Because the permanent deformations in most sections were small at 10,000 repetitions, Figure 12 indicates general trends of the influence of asphalt content on rutting.

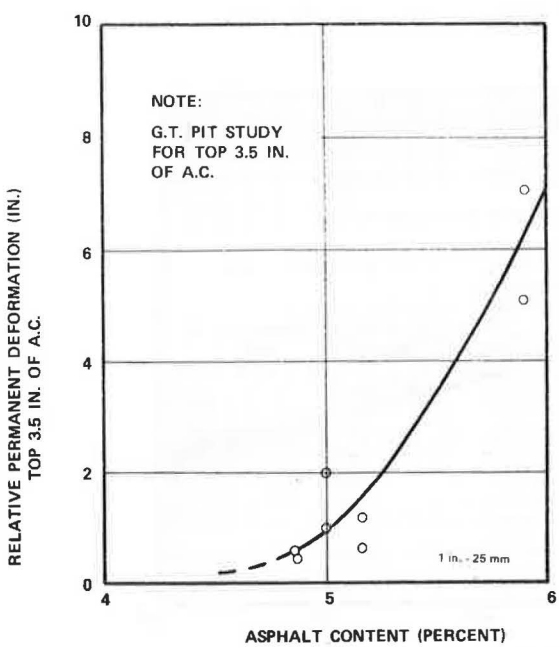

FIGURE 12 Approximate relative rutting in asphalt concrete as function of asphalt content: B and modified-B binder.

The results of these full-scale laboratory studies indicate that the 50-blow Marshall mix design method gives approximately the correct asphalt content. Use of greater asphalt content to increase fatigue life, which is sometimes advocated, does not appear to be justified for heavily loaded sections based on these findings. For heavy traffic and warm summer temperatures, the optimum asphalt content may even be slightly less than the Marshall value.

Permanent Subgrade Deformation

The same resilient micaceous silty-sand subgrade was used beneath all test sections. As previously discussed, the subgrade was removed and recompacted at the same density and moisture content for each test. Figure 13 shows that an increase in base thickness causes a decrease in permanent subgrade deformation. As the base thickness increases, however, the rate at which the deformation decreases becomes less.

A 9-kip (40-kN) dual-wheel load would cause about 1.4 times more rutting than that shown in Figure 13. This extrapolation is based on theory and the plastic strain response obtained from repeated-load triaxial tests. Finally, these studies indicated that the full-depth asphalt-concrete sections were no more effective in reducing subgrade rutting per inch of base than the unstabilized crushed-stone base.

Base-Course Coefficients

The full-scale laboratory tests show that excellent performance can be obtained by using relatively thin asphalt-concrete surfacings and properly constructed crushed-stone bases. The crushed-stone base sections outperformed the full-depth sections in every test series. A higher asphalt content in the full-depth sections probably accounted for most of the poor performance of the full-depth sections.

Based on the observed fatigue and strain response of the pavement, one application of the 6.5-kip circular load used in this study is approximately equivalent to 0.58 applications of a 9-kip dual-wheel load. Now assume an AASHTO Interim Guide (7,8) layer coefficient a_1 of 0.44 for the 3.5-in. asphalt-concrete surfacing and a soil support value of 3.5 (which is greater than would be generally used in Georgia for Piedmont soils). A regional factor of 0.5 assumes that no environmental effects occurred during the study. For this conservative set of assumptions, the average calculated AASHTO Interim Guide crushed-stone base-course coefficient a_2 is 0.19. Based on the results of this study and observed field response (1), a base-course coefficient of 0.18 was recommended for total pavement thicknesses less than 15 in. (381 mm); this is slightly less than the maximum structural thickness used in this study. The engineered crushed-stone base should be compacted to at least 100 percent AASHTO T-180 density and have a gradation approximately similar to that of the stone used in this study. Also, segregation should be minimized during construction and adequate drainage provided.

CONCLUSIONS

The test results show that engineered crushed-stone base sections having relatively thin asphalt-con-

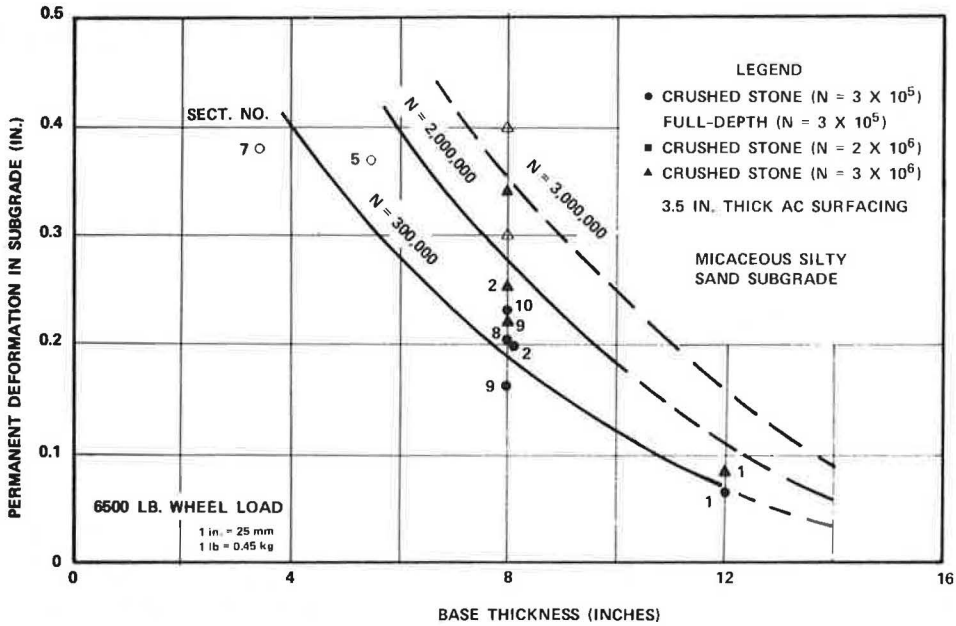

FIGURE 13 Effect of base thickness and number of load repetitions on subgrade rutting: crushed-stone base and full-depth asphalt-concrete test results.

crete surfacings can successfully withstand large numbers of heavy loadings. Full-scale field tests such as those at Lake Wales, Florida; Stockbridge, Georgia; and in North Carolina support this finding (1,9,10). The results of this study show that rutting in a properly constructed crushed-stone base can be less than that in either the asphalt-concrete surfacing or a silty-sand subgrade at a temperature of 79°F; at higher temperatures even greater amounts of rutting would occur in the asphalt concrete.

The good performance of the engineered crushed-stone base is attributed to (a) a uniform, high degree of density (100 percent of AASHTO T-180), (b) use on a well-graded crushed stone with 1- to 2-in. top size that has only 4 to 5 percent passing the No. 200 sieve, (c) practically no segregation, and (d) a relatively thin asphalt-concrete surfacing.

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