MISCELLANEOUS PAPER S-73-4

STUDY OF BEHAVIOR OF BITUMINOUS-STABILIZED PAVEMENT LAYERS

by

C. D. Burns, R. H. Ledbetter, R. W. Grau

March 1973

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
Vicksburg, Mississippi

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FOREWORD

The investigation reported herein was sponsored by the Directorate of Military Construction, Office, Chief of Engineers, and was conducted under Project 6-1, Task 08, Work Unit 004, "Behavior of Bituminous-Stabilized Pavement Layers." The responsibility for conducting the study was assigned to the Soils and Pavements Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss. Field tests were conducted from June through August 1970.

The investigation was conducted under the general direction of Messrs. J. P. Sale, Chief, and R. G. Ahlvin, Assistant Chief, Soils and Pavements Laboratory. Engineers of the Soils and Pavements Laboratory who were actively engaged in the planning, testing, analyzing, and reporting phases of this study were Messrs. R. L. Hutchinson, C. D. Burns, H. H. Ulery, Jr., W. N. Brabston, R. W. Grau, and R. H. Ledbetter. Mr. J. E. Watkins was lead technician. This report was prepared by Messrs. Burns, Ledbetter, and Grau.

Directors of the WES during the conduct of this study and the preparation of the report were COL Levi A. Brown, CE, and COL Ernest D. Peixotto, CE. Technical Director was Mr. F. R. Brown.
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CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

<table>
<thead>
<tr>
<th>Multiply</th>
<th>By</th>
<th>To Obtain</th>
</tr>
</thead>
<tbody>
<tr>
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<td>kilograms</td>
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<tr>
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<td>gallons (U. S. liquid)</td>
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<td>pounds (mass) per cubic foot</td>
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<td>kilograms per cubic meter</td>
</tr>
<tr>
<td>Fahrenheit degrees</td>
<td>5/9</td>
<td>Celsius or Kelvin degrees*</td>
</tr>
</tbody>
</table>

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: \( C = \frac{5}{9}(F - 32) \). To obtain Kelvin (K) readings, use: \( K = \frac{5}{9}(F - 32) + 273.15 \).
SUMMARY

The investigation reported herein was conducted to (a) compare the performance of bituminous-stabilized base and subbase materials with that of unbound granular materials as used in the original multiple-wheel, heavy gear load (MWHGL) test section and (b) determine the difference in performance between a high quality bituminous base constructed of crushed aggregate and a bituminous base constructed of a lower quality uncrushed material.

A test section was constructed within the existing MWHGL test section at the U. S. Army Engineer Waterways Experiment Station (WES), utilizing the existing 4.0-CBR clay subgrade. The test section consisted of four test items. Items 1 and 2 were constructed to a thickness of 15 in., and items 3 and 4 to a total thickness of 24 in. In item 1, the granular base and subbase used in the original construction were replaced by a bituminous-stabilized base constructed of the uncrushed gravelly-sand subbase material used in the original MWHGL test section. Cement filler of 6.5 percent was used with the aggregate to improve the gradation. Item 2 was identical with item 1, except for the 12-in. base, which was constructed of a high-quality asphaltic concrete containing crushed limestone. In item 3, the unbound crushed-stone base used in the MWHGL test section was replaced by a high quality asphaltic-concrete base, and the gravelly-sand subbase in the bottom 15 in. of the structure was stabilized with asphalt cement. Item 4 was identical with item 3, except that the gravelly sand was not stabilized. A 3-in.-thick surface layer of high quality asphaltic concrete was constructed over all test items.

The test items were subjected to traffic with a simulated C-5A main gear 12-wheel assembly with a 360,000-lb gross load and with a 75,000-lb single-wheel assembly.

The results of tests showed that:

a. The performance of the bituminous-bound base and subbase materials was superior to that of similar pavements constructed of unbound granular materials used in the original MWHGL test section at the WES.

b. The quality of aggregate used in the bituminous base courses had a significant effect on pavement performance.

c. The greatest benefit from bituminous stabilization was in upgrading the quality of poor-to-borderline materials.
STUDY OF BEHAVIOR OF BITUMINOUS-STABILIZED
PAVEMENT LAYERS

PART I: INTRODUCTION

Background

1. The concept by which a bituminous base is equal to more than its thickness of granular or unbound base has commonly been used rather extensively by highway departments and, to some extent, by civil airfield pavements designers. Some designers have assigned equivalency factors to various paving materials so that the strength and load-distributing characteristics of materials can be incorporated into flexible pavement design. This procedure has been justified on the basis of engineering judgment and experience, particularly on experience obtained from the American Association of State Highway Officials road test. However, the current Corps of Engineers (CE) design procedures make only limited allowance for the potentially desirable characteristics of stabilized materials such as bituminous base or subbase course.

2. In recent tests at the U. S. Army Engineer Waterways Experiment Station (WES), a field test section, which consisted of five different thicknesses of flexible pavement constructed over a 4.0-CBR clay subgrade, was trafficked to failure under a multiple-wheel, heavy gear load (MWHGL) assembly. The C-5A main gear was the specific assembly used. The test section was constructed with granular base and subbase materials meeting the requirements of the CE criteria. The results of these tests are reported in reference 1. A portion of this test section was utilized for the study reported herein.

Objectives

3. The objectives of this study were to:
   a. Compare the performance of bituminous-stabilized base and
subbase materials with that of unbound granular materials used in the original MWHGL test section.

b. Determine the difference in performance between high quality bituminous base constructed of crushed aggregate and bituminous base constructed of lower quality uncrushed material.

Scope

4. The objectives of this investigation were accomplished by the construction and traffic testing of a specially designed test section consisting of four items of flexible pavement as described herein. This report describes the test section, traffic testing, and results of the tests.
PART II: DESIGN

Layout

5. A plan and profile of the test section are shown in plate 1. The test section was 120 ft* long and 30 ft wide and was located at the west end and along the north side of the existing MWHGL test section, as indicated in plate 2. The test section was constructed and tested in conjunction with a membrane-enveloped soil layer (MESL) test section, which will be reported on in a separate document. The bituminous-base test section consisted of four test items, each 30 ft long and 30 ft wide. Items 1 and 2 both had a total pavement thickness of 15 in., and items 3 and 4 both had a total pavement thickness of 24 in. The existing heavy clay subgrade with a CBR of 4.0 was utilized for this test.** As can be seen in plate 2, the two 15-in.-thick pavement items (1 and 2) replaced item 1 of the original MWHGL test section, and items 3 and 4 (the 24-in.-thick pavements) replaced item 2 of the original test section. A description of the various test items is given in the following paragraphs.

Item 1

6. Item 1 consisted of a 12-in.-thick bituminous-stabilized base course, designed to meet the requirements specified in reference 2, and a 3-in.-thick asphaltic-concrete surface course, designed to meet the requirements specified in reference 3. The material used for the base course was the natural, uncrushed gravelly-sand material that had been used as the subbase in the original MWHGL test section; however, a 6.5 percent cement filler was added to improve the gradation. Grading curves of the gravelly sand with and without the cement filler are shown in plate 3. A laboratory mix design with the aggregate containing

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* A table of factors for converting British units of measurement to metric units is presented on page ix.

** Properties of the heavy clay were considered to be the same as had been determined for the MWHGL study; therefore, the laboratory tests were not repeated.
the cement filler and an 85-100 penetration-grade asphalt was made using the Marshall 50-blow compaction effort. The laboratory mix properties are shown in plate 4. From these data, an optimum asphalt content of about 5.5 percent was indicated.

7. The 3-in. surface course layer was made from 3/4-in. maximum-size crushed limestone, sand filler, and 85-100 penetration-grade asphalt. The aggregate and asphalt were from the same source as that used in the original MWHGL test section and met all the requirements specified in reference 3. An aggregate grading curve of the asphaltic-concrete mixture is shown in plate 5. Laboratory mix design properties are shown in plate 6. From these data, a design asphalt content of 5.0 percent was indicated for the asphaltic-concrete mixture.

Item 2

8. Item 2 was designed for a full-depth (15-in.-thick), high quality asphaltic concrete. The mixture used for the 3-in. surface layer of item 1 was used for the full 15-in. thickness over the subgrade; however, some variation occurred in asphalt content, as will be discussed later.

Item 3

9. Item 3 consisted of a 15-in.-thick, asphalt-stabilized subbase overlain by 9 in. of high quality asphaltic concrete. The subbase consisted of the natural gravelly-sand aggregate as used in the original MWHGL test section; however, the subbase was stabilized with 85-100 penetration-grade asphalt cement. A gradation curve for the aggregate is shown in plate 3. A laboratory mix design was obtained using a 50-blow Marshall compaction effort. Mix properties are shown in plate 7. From these data, an optimum asphalt content of about 6 percent was indicated. The top 9 in. of pavement consisted of high quality asphaltic concrete that was used in the surface layer of item 1 and in test item 2.

Item 4

10. Item 4 consisted of 15 in. of unstabilized gravelly-sand subbase material (the same as had been used in item 2 of the original MWHGL test section) overlain with 9 in. of high quality asphaltic concrete, the same mix as for item 3 above. Thus, the only difference between
items 3 and 4 was the asphalt stabilization of the gravelly-sand subbase in item 3.

Instrumentation

11. Instrumentation for the bituminous-stabilization project utilized a new type of strain sensor. The sensors were installed at various depths in each item of the test section in order to obtain measurements of partial vertical deflection, vertical strain, and lateral strain. Thermistor probes were installed for temperature measurements. A total of 29 instruments were installed, the locations of which are shown in plate 8.

12. The new type strain gage, called a strain sensor, was manufactured by Bison Instruments, Inc. The strain-gage system consisted of sensors and an external instrument package. The sensors were individual disk-shaped coils, and the principle of operation involved the mutual inductance coupling of two coil sensors placed in one of three alignments: lateral, parallel, or perpendicular. Separation of the coils was sensed by using the electromagnetic coupling between the two sensors. This change in electromagnetic coupling is a nonlinear function of strain; however, the change could be calibrated very accurately with resolution of spacing change at least as good as 0.001 in. The coils had no mechanical connection between them, and they operated at any spacing between one and four times the nominal coil diameter, as long as there was no disturbance of the induced electromagnetic field such as metal between or around the coils. Standard coils came in 1-, 2-, and 4-in.-diam sizes; the 4-in.-diam size was used in this study. Fig. 1 shows one 4-in.-diam coil. Columns of coils were used, as shown in plate 8, with the interior coils of the columns acting as common sensors to two locations; this dual usage will be discussed later.

13. The external instrument package, to which the sensors were connected, was a field-use instrument that contained all necessary driving, amplification, balancing, readout, and calibration controls and had a self-contained power supply. Changes in coil spacing were determined
by means of bridge balance, meter deflection from zero, or voltage output on a recorder connected to the rear panel of the instrument package. The instrument package used for this study detected both static and dynamic strain. Response of the instrument was about 0.1 msec.

14. Thermistor probes were installed in item 3 only, as shown in plate 8. Data from these 12 probes should be representative of all items and all depths within the bituminous items. The thermistor probes were Digitec Model S01N manufactured by United Systems Corporation. The probes were monitored with a direct-reading digital temperature meter, which was connected to a recorder for automatic monitoring.

15. The three objectives of the instrumentation program were to:
   a. Investigate the new type of strain gage.
b. Correlate the strain and deflection data of the bituminous-stabilized section with those of the MWHGL test section.

c. Collect data to aid in the analysis of performance of the different bituminous-stabilized items.
PART III: CONSTRUCTION OF TEST SECTION

General

16. The test section was constructed during the period 16 April - 8 June 1970. Details of the construction procedure are described in the following paragraphs.

Excavation

17. An area 30 ft wide and 120 ft long was laid out on the north side of items 1 and 2 of the original MWHGL flexible-pavement test section, along with an area 30 ft wide and 90 ft long on the south side of the two items. The area on the south side was used for a MESL test section. A gasoline-powered concrete saw was used to cut through the 3-in. asphaltic-concrete surface layer around the perimeter of the two test sections. A motor grader equipped with a single-tooth scarifier attachment was used to rip up the asphaltic concrete, which was then pushed up in piles, loaded into trucks, and hauled to a disposal area. The old crushed-stone base and subbase materials were pushed up into a pile, loaded with front-end loaders onto trucks, and hauled to the stockpile. Approximately 60 percent of the base and subbase materials was salvaged for reuse. All material was excavated to the surface of the existing heavy clay subgrade with the exception of the 15-in.-thick layer of gravelly-sand subbase material at the location of test item 4; this material was left in place.

Preparation of Subgrade

18. The clay subgrade in items 1, 2, and 3 was undercut approximately 0.2 to 0.3 ft to remove all rock and sand from the surface. New preprocessed heavy clay was then hauled in to replace the material removed, was spread, and was incorporated into the existing surface by use of a pulvimixer. The surface was then compacted by eight coverages of a
seven-wheel, self-propelled, rubber-tired roller loaded to 47,000 lb with a tire inflation pressure of 65 psi. After it had been tested to determine water content, density, and CBR, the subgrade was fine bladed to the desired elevation and sealed with an asphalt emulsion. The sealing protected the subgrade from drying or wetting prior to construction of the base and pavement. Sealing was accomplished by a spray application of approximately 0.5 gal/sq yd of cationic asphalt emulsion C-RS-2.* A view of the sealed subgrade is shown in fig. 2.

Fig. 2. Asphalt seal coat on subgrade

Bituminous-Treated Materials

Mixing operations

19. The bituminous-treated materials for the subbase and base course layers of the test section were mixed and placed during the period 5-7 May 1970, and the 3-in.-thick surface layer of asphaltic concrete was mixed and placed on 8 June 1970. All mixtures were made in a central hot-mix batch plant located at the WES. The mixing temperature ranged from about 275 to 300 F, and, since the distance from the plant to the test site was short, the temperatures at laydown were about the same.

Laydown and compaction

20. The bituminous-treated gravelly sand in item 3 was the first

* The American Society for Testing and Materials Designation C-RS-2 is the equivalent of The Asphalt Institute Designation RS-2K.
mix placed. The first lift was dumped directly on the subgrade (fig. 3) and spread with a D-4 dozer to form about a 9-in. lift. Initial efforts to compact the material with a self-propelled, rubber-tired roller and a steel-wheel roller, at normal rolling temperatures of 250°F or above, were unsuccessful due to the low stability of the mix. Therefore, the breakdown rolling was accomplished with a D-4 dozer (fig. 4). After the mix had cooled to 150-170°F, it was compacted by three coverages of a 10-ton tandem roller (fig. 5), followed by eight coverages of the seven-wheel, rubber-tired roller loaded to 47,000 lb and having 90 psi tire pressure (fig. 6).

21. The first lift of the bituminous-stabilized gravelly-sand mix with 6.5 percent cement filler (base course) was then placed in test item 1. The stabilized gravelly sand was placed directly on the subgrade in a 6-in.-thick lift with a Barber-Greene spreader. This material appeared to be much more stable than was the mix in item 3, and no difficulties were experienced in breakdown rolling with the tandem steel-wheel roller. Final compaction was accomplished by eight coverages of the rubber-tired roller described previously.

22. The third mix placed was the high quality crushed-stone mix used as the base course in item 2. A 6-in.-thick lift of this material (2.9 percent asphalt) was placed directly on the subgrade in item 2 by use of the Barber-Greene spreader. The material was placed at a temperature of 270-300°F, and breakdown rolling was accomplished immediately behind the spreader with no difficulties. Final compaction was accomplished with the rubber-tired roller. After completing the first 6-in. lift in item 2, the second lift of the bituminous-stabilized gravelly sand (approximately 6 in. thick) was placed in items 1 and 3. Compaction was accomplished with the steel-wheel and rubber-tired rollers previously described. Following the placing of the materials in items 1 and 3, a 6-in.-thick layer of high quality asphaltic-concrete mix (5.0 percent asphalt) was placed as the base course in items 2, 3, and 4. The mix was placed at a temperature of 260-300°F and was immediately compacted with the steel-wheel and rubber-tired rollers.

23. A tack coat of approximately 0.025 gal/sq yd of asphalt
Fig. 3. Dumping hot asphaltic gravelly-sand mix in item 3

Fig. 4. Initial compaction of bituminous-stabilized subbase in item 3
Fig. 5. Rolling bituminous-stabilized subbase in item 3

Fig. 6. Final compaction of stabilized subbase in item 3 with self-propelled, rubber-tired roller
emulsion was sprayed over the surface of the test section area prior to placing the final 3-in.-thick layer of asphaltic concrete. The mixing operations for this layer were the same as those previously described. The mix was placed in a single 3-in. layer with the Barber-Greene spreader (fig. 7). Breakdown rolling was accomplished with the 10-ton steel-wheel tandem roller, followed by the self-propelled, rubber-tired roller, and then a final rolling with the steel-wheel roller.

Fig. 7. Placing 3-in. surface course of asphaltic concrete

Properties of As-Constructed Pavements

Bituminous mixes

Hot-bin aggregate samples were taken periodically at the plant and checked to ensure that the desired gradation was being obtained. Plant-mixed samples of each mixture were also obtained for the laboratory extraction test and Marshall compaction tests. The results of the laboratory tests on the various mixes are summarized in table 1. From these data, it can be noted that the actual asphalt content used in stabilizing the gravelly-sand material in items 1 and 3 was only about 50 percent of the optimum indicated in the laboratory by the 50-blow Marshall mix design. However, from visual observations, it was apparent that the 2.9 percent was adequate to thoroughly coat the aggregate particles and the mix was more workable than it would have been at higher
asphalt contents. For the crushed-limestone surface course, the asphalt content was reduced from 5.0 percent, as used in the base course, to 4.5 percent for the top 3-in. layer. This reduction was made due to the low total-mix voids indicated in the base course layer.

Layer thickness and subgrade data

25. A summary of as-constructed thickness, CBR, water content, and density data of the heavy clay subgrade is shown in table 2. The subgrade data were obtained prior to placing the bituminous pavement layers. The thicknesses of the various types of bituminous layers were measured during construction. From these data, it can be noted that the as-built total thickness over the subgrade was very close to the design thickness in each test item.

Installation of Instruments

26. The installation of instruments was accomplished in conjunction with the construction of the test section. A total of 17 strain sensors were installed as follows:

a. Two coils in item 1 at an initial spacing of 12 in. in vertical alignment: one coil at a depth of 15 in. (at the top of the subgrade), and one coil at a depth of 3 in. (at the interface of the base and surface courses)
b. Three coils in item 2 at vertical alignment spacings of 9 and 12 in.: a coil at a depth of 24 in. (9 in. into the subgrade), a common coil at 15 in. (at the top of the subgrade), and a coil at a depth of 3 in. (base/surface course interface). The coil at 15 in. was the common coupling coil with the ones at 3 and 24 in.
c. Five coils in item 3 at vertical alignment spacings of 3, 6, and 9 in.: a coil at the 24-in. depth (at the top of the subgrade), a common coil at a depth of 15 in. (within the stabilized subbase), a common coil at a depth of 9 in. (at the base/subbase interface), a common coil at a 3-in. depth (between the base and surface course), and a surface coil.
d. Seven coils in item 4 at vertical alignment spacings of 9 and 6 in. and four coils at a lateral alignment spacing of 12 in.: a coil at a depth of 33 in. (9 in. into the subgrade), a common coil at the 24-in. depth (at the top of the subgrade), a common coil at a depth of 15 in.
(within the unstabilized subbase), two common coils at a
depth of 9 in. (at the base/subbase interface) with a
lateral spacing of 12 in., and two coils at a depth of
3 in. (between the base and surface course) with a
lateral spacing of 12 in.

27. These columns of coils were located 26-1/2 in. south of the
center line of the 12-wheel traffic lane and in the east-west center of
each item. The lateral-spaced coils in item 4 were located 12 in. north
of the column. The location of the coil columns was such that, when
static load tests were conducted with the 12-wheel configuration in the
center of the traffic lane of each item, the back inside tire of the
front six wheels would be directly over the columns.

28. The coils placed in the subgrade in items 2 and 4 were in-
stalled after the completion of the preparation of the subgrade. The
excavation was made to the proper depths, the coils were placed, and the
excavated hole was filled and machine tamped. Installation of all other
coils was attempted during construction. They were laid at the proper
elevations and in proper columnar alignment, covered by a few inches of
cold-mix asphaltic concrete to protect them from the hot asphaltic con-
crete and to hold them in place during construction; and the construc-
tion operations continued over them. The electrical cables of the coils
were also laid out, routed to the section side, and covered with cold
mix at the same time as the placing of the coils.

29. Several failures occurred during the above installation.
These failures were attributed to two causes: the cable-coil connec-
tions were weak, and the hot asphaltic concrete melted the cables. Not
all coils experienced these failures. A few coils and cables had to be
replaced. Replacing either the coils or the cables involved excavating
a hole through a lift of asphaltic concrete. Manufacture of the coils
has since been improved, and coil-cable connections are no longer a
weakness in the system; however, for placement in hot asphaltic concrete,
protection of the cable from the heat of the paving mix still remains a
problem.

30. Installation of the 12 thermistors was also accomplished dur-
ing the test section construction. The thermistors were installed in
duplicate at each depth in order to increase the probability of acquiring temperature measurements. The thermistors were installed at the surface and at 3-, 9-, 15-, 18-, and 24-in. depths (plate 8). The thermistors were also installed in columnar form with the locations at each depth being 2 and 3 ft south from the north edge of the section.

31. The installation technique was to place the thermistors at each depth and cover them with cold-mix asphaltic concrete prior to construction of the next lift. The cold mix was used as an attempt to protect the thermistors from the direct heat of the next lift of asphaltic concrete. This procedure was not completely successful; thermistors were lost and were not replaced. Also, damage occurred to some, and they did not operate long; however, for most of the test section life, at least one thermistor at each depth functioned properly.
PART IV: TRAFFIC TEST AND RESULTS

Test Conditions and Procedures

General

32. Test traffic was applied during June, July, and August 1970. Traffic was applied in two separate traffic lanes as indicated in plate 1. The initial traffic was applied in lane 1 with a 360,000-lb 12-wheel assembly. Lane 2 was tested with a 75,000-lb single-wheel load. Traffic was also applied in items 4 and 5 of the original MWHGL test section with the 75,000-lb single-wheel load. The test carts, traffic patterns, and failure criteria are discussed in the following paragraphs.

Test carts

33. The multiple-wheel gear assembly test cart used in lane 1 is shown in fig. 8. This assembly represented one main gear of the C-5A aircraft. The cart was powered by a prime mover with electric drive wheels that straddled the test lane. The 12-wheel assembly consisted of two load boxes, each of which was carried by 6 load wheels, resulting in the 12-wheel arrangement shown in plate 9. The boxes were loaded to a net weight of 360,000 lb distributed equally over the 12 wheels. Each tire was inflated to 100 psi and had a tire contact area of 285 sq in.

Fig. 8. Load cart with 12-wheel gear assembly.
per tire and a contact pressure of 100 psi.

34. The single-wheel assembly consisted of a load box supported by an A-frame and towed by a Caterpillar 619 tractor. The load box was equipped with a single test wheel with a 56x16, 38-PR tire inflated to 290 psi and having a tire contact area of 270 sq in. and an average contact pressure of 278 psi. The assembly was loaded to 75,000 lb.

Traffic patterns

35. Test lane 1 was 16 ft wide (plate 1). Traffic was applied with the 12-wheel assembly by following five guidelines, which were painted on the surface on 16-in. centers (approximately one tire-print width). The distribution of traffic coverages* over the 16-ft-wide traffic lane, after one complete pattern of traffic, is shown in plate 10a. To apply a traffic pattern, the test cart first traveled the full length of the test lane along guideline 1 (south side of traffic lane) and then traveled back along the same line; then the cart was shifted laterally to run the adjacent line. After line 5 at the north side of the lane had been tracked, the guidelines were traveled in reverse order. In order to produce even distribution of traffic coverages over the center 60 in. of the traffic lane, guideline 3 was tracked twice when the cart was traversing the lane from south to north but only once when the cart was going from north to south. This method of application resulted in a total of 22 passes of the load cart for each pattern of test traffic. Each pattern of traffic resulted in 32 coverages of a test wheel over the center 60 in. of the test lane.

36. The single-wheel traffic lane was 6 ft wide. Five guidelines, approximately one tire-print width apart, were painted on the surface. In the application of traffic, the load cart was driven forward and backward along the same path (guideline), then shifted laterally one tire-print width on each forward pass; and then the process was repeated. Therefore, when the test cart had traversed the full distance across the test lane, a total of two coverages had been applied on the test lane.

* The term coverages as used herein indicates a measure of wheel load repetitions for the full tire-print width on any given area of the pavement surface.
Traffic was applied to result in the normal distribution pattern shown in plate 10b. The center 14 in. of the traffic lane received 100 percent of the applied traffic patterns, while the exterior portions received 80 and 20 percent, as shown in plate 10b.

**Pavement temperature**

37. Traffic was applied to the two traffic lanes of the test section during the period 17 June-21 August 1970. The ambient temperature and the pavement temperatures, as determined from thermistors that were installed at various depths, were recorded hourly during portions of the traffic period. Plots of ambient and pavement temperature versus time for typical cool and hot days during traffic are shown in plates 11 and 12, respectively. Plots of the temperature data measured daily during the period 18-31 August 1970, at 10 a.m. and 4 p.m., are shown in plates 13 and 14, respectively. These data show that the pavement temperature near the surface (3-in. depth) responded rapidly to hourly variations in ambient temperature, whereas the pavement temperatures at depths of 18 and 24 in. responded very slowly and did not show much variation from day to day. The data in plates 13 and 14 show a gradual decrease in pavement temperatures at the 18- and 24-in. depths as the average ambient temperature decreased. Typical temperature-gradient curves for the full-depth bituminous pavement in test item 3 are shown in plate 15. These data were obtained on the date and hour indicated in the legend of plate 15.

**Failure criteria**

38. In judging structural failure of the test items, distinction was made between settlement due to traffic compaction and distortion due to shear deformation. Some settlement in the pavement structure due to traffic compaction was anticipated, because it was not possible to apply a heavy compaction effort on the lower layers of the pavement due to the weak subgrade. The term "shear deformation" as used herein refers to excessive plastic movement or, in the extreme, the rupture of any element in the pavement structure. Shear deformation could normally be detected by the occurrence of surface upheaval adjacent to the wheel path or traffic lane.
39. A pavement item was considered failed when either of the following conditions occurred:
   a. Surface upheaval of 1 in. or greater of the pavement adjacent to the traffic lane.
   b. Severe surface cracking to significant depths (for these tests, 3 in. or greater).

Behavior of Pavement Under Traffic

General

40. Visual observations of the behavior of the test items were recorded throughout the traffic test period. These observations were supplemented by photographs. Level readings were taken on the pavement prior to and at intervals during traffic to show the development of permanent pavement deformation and deflection of the pavement under the assembly load for the lane being observed. After each failure, a thorough investigation was made by excavating test trenches across the traffic lane and by establishing profiles of the surface of the various layers in the structure. The CBR of the subgrade and other pertinent test properties were measured to determine where failure had occurred. The data obtained during the traffic tests are presented in the following paragraphs.

Lane 1, 360,000-lb, 12-wheel assembly

41. Visual observations. A general view of the traffic lane prior to traffic is shown in photo 1. As traffic was applied, surface cracking and deformation developed quite rapidly in test item 1. Slight cracking and noticeable grooving were also evident in all test items within the first 40 coverages of traffic. A view of item 1 after 34 coverages, showing deformation and the badly cracked pavement in approximately a 5-ft square near the center of test item 1, is depicted in photo 2. This item was considered failed at the end of 98 coverages, when rather severe block cracking had developed throughout the test item (photo 3). Severe shear failure had developed in about the center third of the test item (photo 4).
42. By the end of 105 coverages, rutting and surface cracking were quite pronounced in test item 2 (photo 5). The maximum total deformation at this time was about 1-3/4 in. Individual ruts or tire-print grooves of about a 1/2-in. depth were left behind each load wheel following each pass of the load cart. Mostly, however, longitudinal cracking was quite pronounced along the ridges between the tire prints (photo 5). As the load cart was shifted laterally across the traffic lane, the cracks would tend to seal when under the tires and then open when between the tires. This behavior indicated that the pavement was experiencing plastic flow and was moving laterally from under the load wheels. A pavement core cut through one of the most pronounced surface cracks revealed that the crack only extended to a depth of about 3/4 to 1 in. into the pavement (photo 6).

43. As traffic continued, the pavement cracking and deformation in test item 2 continued to progress. A general view of the condition of item 2 at end of 425 coverages is shown in photo 7. A closeup view of the cracking pattern and measurement of the maximum deformation at this time is shown in photo 8. A view of pavement core cut through one of the worst cracks in the traffic lane is shown in photo 9. This core was approximately 15 in. long, the full depth of pavement. The surface crack extended about 3 in. into the pavement, the bottom 4 in. of the core was cracked, and some clay from the subgrade had worked up into the bottom of the pavement. It is not known whether this condition developed during construction or was caused by traffic. Due to the severe deformation and cracking of the asphaltic concrete, this test item was considered failed at the end of 425 coverages; however, traffic was continued to a total of 719 coverages. A view showing the total deformation and cracking pattern at the end of traffic is shown in photo 10. Plastic flow of the asphaltic concrete was evident from the widening of the traffic lane. The original width of the traffic lane was 16 ft and was defined by two parallel white lines painted on the pavement. By the end of the traffic period, the distance between the two white lines had increased by approximately 6-1/2 in. (photo 11).

44. In test item 3, slight grooving and surface cracking
developed early in the traffic period. A general view of the test item at the end of 112 coverages is shown in photo 12. There had been very little change in condition of the item by the end of 434 coverages, except that the surface cracking was slightly more pronounced (photo 13). A core cut through one of the widest cracks revealed that the cracking extended to only a very shallow depth (photo 14). The bottom portion of this core was the asphalt-stabilized gravelly-sand material. There was little or no apparent change in the condition of the item as traffic continued to 728 coverages. A general view showing permanent deformation at the end of 728 coverages is shown in photo 15. Although the asphaltic concrete in the top 9 in. of item 3 was identical with that in item 2, the pavement in item 3 did not appear to flow laterally as it had in item 2, and there was no change in the width of traffic lane at this time (photo 16). Traffic was continued to a total of 2198 coverages, when the item was considered failed due to extensive surface cracking, upheaval at the edge of traffic lane, and total deformation (photo 17). A number of cores taken inside the traffic lane at the end of traffic showed that the cracks extended only a shallow depth into the pavement (photo 18).

45. Pavement cracking and permanent deformation developed quite rapidly in item 4. The general behavior was quite similar to that described for test item 2, where grooving of the pavement developed in the wheel paths and cracks opened up in the ridges between tire prints. Photo 19 shows a general view of the pavement after 119 coverages of traffic. Note the contrast of the appearance of the pavement in item 3 (foreground, photo 19) with that of item 4. The only difference between the pavements of these two items was the asphalt used in the gravelly-sand subbase of item 3 and the unbound gravelly-sand subbase in item 4. By 442 coverages, the maximum deformation in item 4 was approximately 3 in. (photo 20). Cores taken through the most pronounced cracks in the surface of the pavement revealed that the cracks extended about 3/4 to 1 in. into the pavement (photo 21). By 734 coverages, maximum deformation had increased to about 4 in., and cracking was quite severe (photo 22). Measurements across the width of the traffic lane showed a
2-in. increase over the original lane width. This increase was due to plastic flow of the asphaltic-concrete mixture. Due to the large amount of permanent deformation, plastic flow, and cracking of the asphaltic concrete, the test item was considered failed at this point.

46. The pavement in item 4 was still structurally capable of carrying a load, and traffic was continued over the test item. Prior to continuing traffic, the east half of item 4 was sealed with polypropylene-asphalt membrane to determine the ability of the membrane to seal the surface cracks and to retard further cracking. Traffic was then continued to a total of 1466 coverages. The polypropylene-asphalt membrane appeared to be effective in sealing the cracks and in preventing new cracks from forming. A view of the east portion of the item taken after a total of 1358 coverages and following the application of the polypropylene-asphalt membrane at 734 coverages is shown in photo 23a. A comparative view of the west portion of the item, which had not been sealed, is shown in photo 23b.

47. Pavement deflection. Surface deflection measurements were made at about the center of each test item at zero coverages and at various intervals during traffic. The term "deflection" as used in this report indicates the total vertical movement that occurs with a single load application. These measurements were obtained with level instruments by reading rods (engineer scales) at prearranged positions on lines parallel and transverse to the direction of traffic. Readings were taken adjacent to and between the load wheels. Rod readings were first taken with the load off the pavement; then the test cart was moved forward until the centroid of the assembly was at a prearranged position, and a second series of readings were taken with the load on. The difference in rod readings indicated the total vertical movement of the pavement. In some instances, the rod readings were obtained for the unloaded condition after the test cart had been moved. The difference in rod readings with the load on and the load off indicated the rebound of the pavement.

48. Plots of the deflection measurements taken prior to test traffic in each of the four test items are shown in plate 16. The upper
plot shows the pavement deflection under the transverse center of the load cart in the direction parallel to traffic. The lower plot shows the deflection in the transverse direction along the axis of the four wheels of the six-wheel assembly of the front load box.

49. The upper plots of plate 17 show total deflection data for test item 3 as measured at zero coverages and at various coverage levels during the period of traffic. The lower plots of plate 17 show the total deflection and rebound measurements made in item 3 at the end of 242 coverages of traffic. From these later data, it can be seen that most of the vertical deflection was due to plastic flow of the bituminous material and that the rebound or recovery of the material after loading was only about 50 percent of the total deflection. Thus, as previously discussed in paragraph 44, slight rutting and ridges formed in the pavement during the early phases of traffic, and the ruts shifted with the lateral distribution of traffic.

50. **Permanent pavement deformation.** Level readings were taken prior to traffic and at various intervals of traffic across the test lane at predetermined stations. These observations were made to determine the magnitude of pavement deformation resulting from traffic. The cross sections, which were taken at two locations in each test item, are shown in plates 18-21.

**Instrumentation data**

51. As mentioned previously (paragraph 27), the strain sensors were located such that the back inside tire of the front 6 wheels of the 12-wheel assembly would be in position over a column of sensors at the time of the surface static-load deflection determinations. From previous work with the 12-wheel assembly, it was known that either of the back inside tires of the front six was a maximum load point for the range of depths considered.

52. As each item was statically loaded for surface deflection determinations, the strain sensors were monitored; readings were taken before, during, and after loading. The load assembly was left in place long enough for approximate equilibrium to occur before the final load reading was taken. After loading an item, the assembly was kept off of
the pavement until approximate equilibrium of rebound movement was reached. This first static loading was made before any other traffic was applied on the test section. A second set of pretraffic static loadings were made with the load point located at the centroid of the back four of the front six wheels. Also, another set of static loadings were conducted after the first 96 coverages on item 1.

53. A high loss of strain sensors occurred under the first static loading and the first coverages of traffic. This loss was attributed to weak connections between the coils and their cables. The coil in item 4 at the 33-in. depth stopped operating prior to completion of construction of the test section. This coil had a short circuit that was probably caused by moisture penetrating the coil housing.

54. During the first static-load tests, four couplings (electromagnetic coupling between two coils) out of seven were lost in item 4, and three out of five were lost in item 3. By the time the first 96 coverages of traffic had been applied, only two couplings in item 4, two couplings in item 3, and none in the other items were operating. The surface coil in item 3 had been replaced in order to keep the coupling between the surface and the 15-in. depth operating.

55. Temperatures of the asphaltic concrete were recorded automatically every half hour for 2½ hours a day, and this recording was continued for the duration of the tests. These temperatures have been discussed previously (paragraph 37).

56. Due to the high loss of couplings and the fact that duplicate responses were not obtained, great reliance was not placed in the instrumentation data for partial deflections. Also, due to the high loss of couplings, comparisons with respect to partial deflections or strains between the different test items were difficult to make. Reduction and analysis of the data were especially difficult, due in part to the small amount of data acquired but due mostly to the highly plastic action of the bituminous material.

57. The best results that could be obtained from the data are presented in table 3. In the table, positive values indicate deflection or compression, and negative values indicate the opposite. Table 4
presents the accumulated partial deflections for each item and also the optically determined surface deflections, for comparisons. As can be seen, the comparisons of total deflection were fairly good for all items, assuming that the subgrade deflection between the 15- and 24-in. depths in item 2 was true for item 1 and that subtracting the negative elastic deflection was appropriate in item 3. If the instrumentation data were correct, the majority of deflection occurred in the upper 24 in. in all items. There were not enough data acquired to warrant presentation of the static-load tests made after 96 coverages.

Lane 2, 75,000-lb, single-wheel assembly

58. Visual observations. A general view of the pavement in lane 2 prior to traffic is shown in photo 24. As traffic was applied to this lane with the 75,000-lb single-wheel load, the total vertical deflection in items 1 and 2 was in excess of 1 in.; the asphaltic concrete started cracking on the first pass of the load wheel. Item 1 was considered failed at the end of 6 coverages (photo 25), and item 2 was considered failed at the end of 8 coverages (photo 26). Test item 4 withstood only 12 coverages of the load wheel (photo 27), while item 3 withstood a total of 90 coverages. A view of item 3 at the end of 90 coverages is shown in photo 28. Failure in all cases was due to excessive deformation and cracking of the asphaltic concrete.

59. Pavement deflection. Plots of pavement deflections taken prior to traffic in the four test items are shown in plate 22. These data show the maximum deflections in items 1 and 2, where the total thickness of the pavement structure was 15 in., were about the same and ranged from about 1.1 to 1.2 in. The total deflections in items 3 and 4 were about equal and were about 0.6 to 0.7 in. These measurements indicated total vertical movement of the pavement, and the grooving behind the load wheel indicated that a major part of the deflection was permanent. Deflection and rebound measurements were made in item 3 at the end of 10 coverages. The pavement rebound was measured 30 min after removing the load and after 16 hr. Plots showing these data are shown in plate 23. From these data, it can be seen that the rebound
after 16 hr was only about 1/3 of the total deflection.

60. **Permanent deformation.** Cross sections taken across the traffic lane showing permanent deformation at the end of the traffic period are shown in plate 24.

Supplemental, 75,000-lb single-wheel-load traffic

61. **General.** In addition to the traffic applied on the bituminous-stabilized pavements, traffic was also applied to previously nontrafficked areas of items 4 and 5 of the original MWHGL test section. These pavement items consisted of a 3-in.-thick layer of asphaltic concrete, a 6-in.-thick crushed stone base, and a gravelly-sand subbase over a 4.0-CBR clay subgrade. The thickness of the gravelly-sand subbase was 24 in. in item 4 and 33 in. in item 5. Thus, there was a total thickness over the subgrade of 33 in. for item 4 and 42 in. for item 5. A complete description of these pavement items is given in reference 1. Traffic was applied to these items using the 75,000-lb single-wheel load used in lane 2 of the bituminous-stabilized pavement test section.

62. **Behavior under traffic.** A view of the traffic lane in MWHGL items 4 and 5 prior to traffic is shown in photo 29. Deflection measurements obtained prior to traffic indicated an initial deflection under the 75,000-lb single-wheel load of about 0.34 in. in item 4 and 0.23 in. in item 5. Hairline pavement cracks and noticeable deformation were observed in item 4 at the end of 2 coverages of traffic. The deflection of the pavement and cracking of the asphaltic concrete increased rapidly with load repetitions, and, by the end of 18 coverages, the item was considered failed. A general view showing deformation and cracking of the pavement in item 4 is shown in photo 30. Item 5 was still in good condition at the end of 18 coverages (photo 31). As traffic was continued in item 5, the deflection and permanent deformation appeared to increase and resulted in cracking of the asphaltic concrete. This item was considered failed at the end of 70 coverages (photo 32).
After-Traffic Testing

63. At the end of the traffic period, test trenches were excavated across the traffic lane in items 1, 2, and 4 of lane 1 and in all items of lane 2 of the bituminous-stabilization test section. No test trenches were excavated in items 4 and 5 of the original MWHGL test section. From the test trenches, elevation profiles were obtained at the surface of the pavement and at the surface of the subgrade in an effort to determine the extent of distortion of the various pavement elements. CBR, density, and water content determinations were made in the subgrade both inside and outside the traffic lanes. Undisturbed block samples and 4-in.-diam cores of the bituminous pavement material were obtained from all test items for laboratory determination of density, voids, stability, and tensile strength. A view of a test trench with 2-ft-square block samples cut and ready for removal is shown in photo 33. Views of cores removed from item 2 both inside and outside the traffic lane are shown in photos 34 and 35, respectively.

64. The results of observations and tests conducted in the test trenches and the laboratory test conducted on the pavement cores are presented in the following paragraphs.

Test pit profiles

65. Test pit profiles taken in items 1, 2, and 4 of lane 1 after failure of the various items are shown in plates 25-27, respectively. From plates 25 and 26 it can be noted that deformation extended through the pavement structure and into the subgrade of both items 1 and 2. There was also some upheaval adjacent to the traffic lane, which was caused by shear deformation in the subgrade. In item 2, the total pavement thickness above the subgrade decreased within the traffic lane and increased outside the traffic lane. These changes were caused by plastic flow of the asphaltic-concrete pavement. Plate 27 shows considerable settlement inside the traffic lane of item 4 at the end of 1408 coverages of traffic, as well as an upheaval adjacent to the traffic lane. This settlement and upheaval appeared to be due primarily to lateral shifting of the unbound gravelly-sand subbase material. From
the profile plots, it can be noted that the thickness of the gravelly-sand layer at the center of the traffic lane was about 15 in. as compared to a thickness of about 18 in. at 2 ft from the north edge of the traffic lane.

66. A test trench was not excavated in item 3 of lane 1, but a trench was excavated in item 3 of lane 2 after 100 coverages of traffic with the 75,000-lb single-wheel load. Pit profiles of the surface of the pavement and subgrade prior to and after traffic are shown in plate 28. These data indicate about the same degree of deformation in the subgrade as at the surface of pavement. However, the 75,000-lb single-wheel load was a much more severe loading than the loading of the C-5A assembly used in lane 1. Pit profiles in the other items of lane 2 showed slightly less deformation in the subgrade than indicated for item 3, but the other items failed due to severe cracking of the asphaltic concrete at a much earlier coverage level.

Thickness, CBR, water content, and density data

67. A summary of thickness measurements of the bituminous pavement layers and the total thickness over the subgrade as measured in the test trenches is shown in table 5. The design total thickness is also shown, for comparison. Also included in the table are CBR, water content, and density data obtained in the clay subgrade. The laboratory CE 55 density values shown in the table are for specimens compacted in the laboratory at the field in-place water content.

Laboratory tests on field cores of bituminous mixes

68. A summary of laboratory test data, including asphalt content, stability, flow, tensile strength, density, and voids of pavement cores, taken at various coverage levels in test lane 1 is shown in table 6. The cores identified as zero coverages were obtained outside the traffic lane at the end of the traffic period. These data were obtained from tests following standard CE test procedures with the exception of the test to determine tensile strength. The tensile strength was computed from results of an indirect tensile splitting test using equipment.
borrowed from the University of Texas. A description of the equipment and theory concerning the test can be found in reference 5.

69. The density data (table 6) indicated that approximately 100 percent of laboratory density had been obtained in the paving mixtures during construction rolling and that only a slight increase in density had developed during traffic testing. The exception to this procedure occurred with the bituminous-stabilized gravelly sand in item 3, where density values of 10.5 and 10.4 percent of the 50-blow Marshall density were measured at the 9- to 12-in. and the 12- to 15-in. depths, respectively, after 2198 coverages of traffic. The initial as-constructed density of these layers was not known as it had been impossible to recover cores of this material from nontraffic areas; therefore, it was evident that traffic had improved the stability of the bituminous-stabilized material.

70. The voids data shown in table 6 indicate that the asphalt content of the top 9 in. of the paving mixtures in items 2, 3, and 4 was slightly on the rich side, as the voids total mix were generally in the range of 2 to 3 percent. The richness of the mixtures was also indicated by the high flow values (table 6) and by the plastic flow of mixtures under traffic as previously discussed (paragraph 42).

Summary of Test Results

71. A summary of the traffic test results for the 12-wheel and single-wheel traffic is shown in table 7. Most of these data are self-explanatory; however, some columns need further explanation as given in the following paragraphs.

Rated subgrade CBR

72. The rated CBR values of the subgrade were based on the numerical average of the CBR values measured immediately after construction (table 2) and after traffic (table 5). In general, the CBR values used were from tests conducted at the surface and 6 and 12 in. into the subgrade. The values measured prior to traffic were assumed to be representative of the subgrade strength in both test lanes 1 and 2. For
items 4 and 5 of the original MWHGL test section, no test pits were excavated following the 75,000-lb single-wheel-load traffic, and it was assumed that the CBR of the subgrade was the same as that measured following the original MWHGL tests, a 4.0 CBR. Therefore, this 4.0-CBR value is shown in table 7.

**Deflection**

73. The deflection values tabulated were the maximum total vertical movements of the pavement measured prior to traffic or at the coverage levels indicated. From the rebound measurements discussed in paragraphs 47 and 57, the elastic deflection was determined as only about 35 to 50 percent of the total deflection.

**Maximum permanent deformation**

74. The maximum permanent deformation values listed in table 7 were obtained from cross-section elevation measurements taken prior to traffic and at the coverage levels indicated. The maximum deformation normally occurred at or near the center line of the traffic lane.

**Upheaval**

75. The tabulated upheaval values were obtained from cross-section elevation measurements taken prior to traffic and at the coverage levels indicated. Upheaval adjacent to the traffic lane was an indication of shear deformation in some element of the pavement structure. In this study, a test item was considered failed when upheaval of 1 in. or more had been measured.

**Pavement cracking**

76. Pavement cracks extending to depths of 1 in. or less into the pavement were classified as slight, those extending to depths greater than 1 in. but less than 3 in. were classified as moderate, and those extending 3 in. or more were classified as severe. For this study, a pavement item was considered failed when cracking had extended to depths of 3 in. or more.
PART V: ANALYSIS OF TEST RESULTS

Performance Comparisons

77. A plot of coverages versus thickness for the flexible pavement items subjected to the traffic of the 360,000-lb 12-wheel gear is shown in plate 29. All points plotted were for failure conditions, except for those of item 5 of the original MWHGL test section, which was still in satisfactory condition at the end of traffic (3850 coverages). This plot shows a straight-line relationship on a semilogarithmic scale of the coverages to failure versus the thickness for test items 1, 2, 3, and 4 of the original MWHGL test section. An extrapolation using this relationship indicated that it would have taken 15,000 to 20,000 coverages of this traffic to fail test item 5. The pavement items in the MWHGL test sections all consisted of a 3-in.-thick layer of asphalt concrete, 6 in. of crushed stone base, and a varied thickness of a gravelly-sand subbase (plate 2) over a 4.0-CBR clay subgrade. These materials all met the current CE design quality requirements for the various elements of the pavement structure.

78. The coverage level at failure versus the total thickness of the bituminous-stabilized test items is shown in plate 29 for direct comparison with the behavior of the unbound granular materials. It can be noted from these comparisons that, in all cases, the bituminous-stabilized test items sustained more traffic coverages prior to failure than those for the original MWHGL pavement items, where the unbound granular base and subbase materials were used. It can also be noted that a greater total thickness of the conventional pavement was required to equal the performance of the full-depth bituminous pavements.

79. A plot of coverages versus thickness for the test items subjected to traffic with the 75,000-lb single-wheel load is shown in plate 30. Only test items 4 and 5 of the original MWHGL test section were subjected to traffic with this loading.

Discussion of Test Results

80. The results of the tests reported herein indicated that the
pavement performance of the bituminous-stabilized material was better in all cases than conventional pavements of the same thickness where granular unbound base and subbase materials had been used. Also, the quality of the aggregate used in the pavement structure had a great influence on behavior. For example, the only significant difference in test items 1 and 2 of the stabilization test section was the quality of aggregate in the bottom 12 in. The aggregate in the bituminous base course of item 1 consisted of uncrushed gravel; failure of this item under the 12-wheel gear occurred at 98 coverages as compared to 425 coverages for item 2, where crushed limestone aggregate had been used in the bituminous base course.

81. In item 4, where only the 6-in. crushed-stone base had been replaced by a 6-in. asphaltic-concrete base, there was some improvement in performance under both the 360,000-lb 12-wheel-gear load and the 75,000-lb single-wheel-load traffic (plates 29 and 30, respectively). This slight increase in performance over a conventional pavement is believed to be due to the asphaltic-concrete base providing more confinement over the unstabilized gravelly sand than did the crushed-stone base.

82. In item 3, where the gravelly-sand subbase had been stabilized with asphalt, the overall performance was much better than that of item 4 of the stabilization section, or of a conventional pavement of unbound base and subbase materials as used in the MWHGL tests for both the 12-wheel-gear traffic and the 75,000-lb single-wheel-load traffic. This significant improvement in performance is believed to be primarily due to stabilization of the gravelly-sand subbase material. There had been some lateral movement of the unstabilized gravelly-sand subbase material under traffic in the MWHGL test section, whereas the asphalt stabilization as used in item 3 prevented this type movement.

83. It should be emphasized that the comparisons discussed herein are based on the total thickness of the base and subbase for the various test items of the MWHGL test section, of which only the 6-in.-thick base course layer consisted of a high-quality crushed-stone material. Later tests (report in preparation) conducted on a pavement item
consisting of a 21-in.-thick crushed-stone base and 3 in. of asphaltic-concrete surfacing for a total thickness of 24 in. over the same 4.0-CBR clay subgrade have shown that this item performed better than item 3 of the bituminous-stabilized section reported herein. However, the bottom 15 in. of item 3, which was stabilized with asphalt, consisted of a low-stability, uncrushed gravelly sand. The asphalt stabilization of the uncrushed gravelly sand was quite beneficial. It is not known what effect the addition of asphalt would have had on the performance of the 24-in. full-depth crushed-stone item.

84. One of the implications from this study and from related work being conducted at the WES is that the quality of material used in all elements of a pavement structure has a significant effect on the load-carrying capability of the pavement: where higher quality materials are used, thickness reductions can be made. In current CE design procedures, no credit is given for the use of subbase materials with strengths higher than the minimum required at a specified depth in the structure. Yet the test data reported herein show that equal performance can be obtained on thinner pavement structures, where the subbase material is upgraded by stabilization or replaced by a high quality crushed stone or bituminous base course.

85. This study was planned as a preliminary study to determine the validity of the concept by which a bituminous base is equal to more than its thickness of granular or unbound base material. The results indicate some validity of the concept; however, much more work is needed to develop adequate design criteria, and such work was beyond the scope of this study. Variables that need further investigation are asphalt content, material type, thickness and location of layers, subgrade strength, and loading condition. For example, the asphaltic-concrete mix used in the top 9 in. of the pavement for test items 2, 3, and 4 of this study appeared to be too rich, as some plastic flow of the pavement developed under traffic. It is not known, however, whether this richness was detrimental or beneficial. Pavement cracking developed in all test items with the rich mix. The cracks tended to reseal, and the life of the pavement may have been longer than it would have been if the mix
had been leaner. On the other hand, the bituminous mixes in the bottom layers of items 1, 2, and 3 were somewhat leaner than normal. It is not known what effect a higher asphalt content would have had on performance. The loading conditions used in this study were both heavy loads. Behavior patterns may vary with load; therefore, tests are needed with some less severe loading conditions. Material type is considered to have a great influence on the "equivalent thickness" concept, and tests should be conducted utilizing a wide range of materials.
PART VI: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

86. Based on the results of tests presented herein, the following conclusions are believed warranted:

a. The performance of the bituminous-bound base and subbase materials is superior to that of similar pavements constructed of unbound granular base and subbase materials.

b. The quality of aggregate used in bituminous base courses has a significant effect on pavement performance.

c. The greatest benefit from bituminous stabilization is in upgrading the quality of poor-to-borderline materials.

d. Due to the high loss of strain sensors and the limited amount of data obtained,* correlations based on instrumentation results between the stabilization test section and the MWHGL test section could not be made. The instrumentation data did indicate that the majority of deflection occurred in the upper 24 in. of all stabilized items. Comparisons between the optically measured surface deflections and the instrumentation results were good.

Recommendations

87. Based on the results of tests presented herein, the following recommendations are believed warranted:

a. A comprehensive study should be initiated to develop a pavement design procedure for thick asphaltic-concrete pavement layers, utilizing material strength properties.

b. Further field tests should be conducted to determine the effects of the following variables on thick asphaltic-concrete pavement layers:

(1) Asphalt content

(2) Material quality

(3) Loading conditions

* Manufacture of the Bison coil has been improved since these tests, and the coil-cable connections are no longer a weakness in the system; therefore, the use of these coils offers an economic and accurate means of measuring a wide spectrum of movements in pavement systems.
LITERATURE CITED


### Table 1
**Summary of Laboratory Test Properties**

<table>
<thead>
<tr>
<th>Sample Identification</th>
<th>Asphalt Content</th>
<th>Compaction Effort</th>
<th>Voids, %</th>
<th>Density</th>
<th>Stability</th>
<th>Flow Units of 1/100 in.</th>
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<tr>
<td>Item 1, bituminous-stabilized gravelly-sand cement base</td>
<td>2.9</td>
<td>50-blow Marshall</td>
<td>7.5</td>
<td>47</td>
<td>146.8</td>
<td>1345</td>
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<td>Item 3, bituminous-stabilized subbase</td>
<td>2.9</td>
<td>50-blow Marshall</td>
<td>12.0</td>
<td>35</td>
<td>139.4</td>
<td>200</td>
</tr>
<tr>
<td>Items 2, 3, &amp; 4, asphaltic-concrete base course</td>
<td>5.0</td>
<td>75-blow Marshall</td>
<td>2.2</td>
<td>84</td>
<td>153.9</td>
<td>3000</td>
</tr>
<tr>
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Note: All values tabulated are an average of three or more determinations.

### Table 2
**Summary of As-Constructed Thickness, CBR, Water Content, and Density Data**

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* All measurements made in center of item.

** See plate 1 for complete description of layers.

† From reference 1.

†† Based on CE 55 maximum density at field in-place water content from reference 1.
### Table 3

**Partial Deflections**

12-Wheel Configuration - Front 6-Wheel Bogie

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* Indications may not be very accurate due to the surface coil not being securely in place.

** Laterally aligned coils.
Table 4
Accumulation of Partial Deflections and Comparison with Optically Determined Surface Deflections

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* 12-wheel assembly, front 6-wheel bogie.
** Values determined by assuming item 2 movement between 15- and 24-in. depths is true for item 1.
† Value determined by subtracting the negative elastic portion.
### Table 5
Summary of After-Traffic Thickness, CBR, Water Content, and Density Data

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Lane 2, 75-Kip, Single-Wheel Assembly

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* See plate 1 for complete description of layers.
### Table 6
Summary of Laboratory Test Data for Field Cores

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<th>Test Item</th>
<th>Coverages</th>
<th>Material</th>
<th>Depth in.</th>
<th>Asphalt Content</th>
<th>Stability</th>
<th>Flow Units of Tensile Strength</th>
<th>Tensile Strength</th>
<th>Density</th>
<th>Voids, %</th>
<th>Total Mix</th>
<th>Condition of Core</th>
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* 50-blow Marshall compaction.

### Table 7
Bituminous Base Course Test Section
Summary of Traffic Test Data

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<th>Load per Tire, lb</th>
<th>Tire Inflation Pressure psi</th>
<th>Tire Contact Area, sq in.</th>
<th>Rated Subgrade</th>
<th>Coversages</th>
<th>Deflection, in.</th>
<th>Permanent Deformation, in.</th>
<th>Upheaval Pavement Cracking</th>
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* Rebound measurements indicate that only approximately one-third to one-half of the total deflection can be considered as elastic deflection.

** Items 4 and 5 of the original MWHGL flexible pavement test section.
Photo 1. General view of traffic lane 1 prior to traffic

Photo 2. Item 1, lane 1, after 34 coverages
Photo 3. General view of item 1, lane 1, after failure at 98 coverages.

Photo 4. Severe shear failure in center third of item 1, lane 1, after 98 coverages.
Photo 5. Rutting and cracking of pavement in item 2, lane 1, at 105 coverages.

Photo 6. Core taken from cracked area of item 2, lane 1, after 105 coverages.
Photo 7. General view of item 2, lane 1, after 425 coverages

Photo 8. Close-up showing deformation and cracking in item 2, lane 1, after 425 coverages
Photo 9. Pavement core taken through cracked area of item 2, lane 1, after 425 coverages.

Photo 10. Deformation and cracking in item 2, lane 1, at end of 12-wheel traffic (719 coverages).
Photo 11. Widening of traffic lane in item 2, lane 1, due to plastic flow of surface course after 719 coverages.

Photo 12. General view of item 3, lane 1, after 112 coverages.
Photo 13. Item 3, lane 1, after 434 coverages

Photo 14. Core cut through cracked area in item 3, lane 1, at 434 coverages
Photo 15. Close-up of deformation in item 3, lane 1, at end of 728 coverages

Photo 16. Measurement showing no change in width of traffic lane in item 3, lane 1, at end of 728 coverages
Photo 17. General view of item 3, lane 1, at failure (2198 coverages)

Photo 18. Cores taken from inside traffic lane after 2198 coverages of traffic in item 3, lane 1
Photo 19. Rutting and cracking of pavement in item 4, lane 1, after 119 coverages

Photo 20. Close-up of measurement of maximum deformation in item 4, lane 1, at end of 442 coverages
Photo 21. Core taken from cracked area of item 4, lane 1, after 442 coverages.

Photo 22. Four-in. permanent deformation in item 4 after 734 coverages.
a. East portion (polypropylene installed after 734 coverages)

b. West portion (no polypropylene on this area)

Photo 23. Item 4, lane 1, after 1358 coverages
Photo 24. General view of lane 2 prior to traffic

Photo 25. Failure of item 1, lane 2, after 6 coverages
Photo 26. Failure of item 2, lane 2, after 8 coverages

Photo 27. Failure of item 4, lane 2, after 12 coverages
Photo 28. Failure of item 3, lane 2, after 90 coverages

Photo 29. General view of items 4 and 5 of MWHGL test section prior to traffic
Photo 30. Failure of item 4, MWHGL test section, after 18 coverages of 75,000-lb single-wheel load

Photo 31. Item 5 of MWHGL test section after 18 coverages
Photo 32. Failure of item 5, MWHGL test section, after 70 coverages
Photo 33. Excavation for removal of undisturbed blocks of bituminous-stabilized material from item 1
Photo 34. Cores cut inside traffic lane of item 2, lane 2, after traffic

Photo 35. Cores cut outside traffic lane of item 2, lane 2, after traffic
PLAN

HEAVY CLAY (CH) SUBGRADE, CBR = 4

NOTE: LANE 1 TRAFFICKED WITH 360-KIP 12-WHEEL ASSEMBLY.
LANE 2 TRAFFICKED WITH 75-KIP SINGLE-WHEEL ASSEMBLY.

TEST SECTION LAYOUT
BITUMINOUS BASE COURSE STUDY
PLAN VIEW

SECTION A-A
FLEXIBLE PAVEMENT TEST SECTION

SECTION B-B
RIGID PAVEMENT TEST SECTION

LOCATION OF BITUMINOUS BASE COURSE STUDY
EXISTING MULTIPLE-WHEEL
HEAVY GEAR LOAD
TEST SECTION
SCREEN OPENING, IN.

PERCENT PASSING

SCREEN NUMBER

SPECIFICATION LIMITS

93.5% GRAVELY SAND WITH 6.5% CEMENT FILLER

GRAVELY SAND

NOTE: STABILIZED GRAVELY SAND WITH CEMENT FILLER WAS USED FOR THE BASE COURSE IN ITEM 1.
NOTE: THIS MIX USED FOR THE BASE COURSE IN ITEM I.

LABORATORY MIX DESIGN PROPERTIES
GRAVELLY SAND WITH 6.5% CEMENT FILLER
50-BLOW MARSHALL COMPACTION
SCREEN OPENING, IN.

PERCENT PASSING

SCREEN NUMBER

SPECIFICATION LIMITS

BLEND OF STOCKPILE AGGREGATES

ASPHALTIC CONCRETE AGGREGATE GRADING
LABORATORY MIX DESIGN PROPERTIES
ASPHALTIC CONCRETE
75-BLOW MARSHALL COMPACTION

PLATE 6
NOTE: THIS MIX USED FOR THE SUBBASE COURSE IN ITEM 3.

LABORATORY MIX DESIGN PROPERTIES
GRAVELLY SAND
50-BLOW MARSHALL COMPACTION

PLATE 7
INSTRUMENTATION FOR BITUMINOUS BASE COURSE STUDY

PLAN

ASPHALTIC CONCRETE SURFACE COURSE (4.5% AC)

BITUMINOUS STABILIZED BASE COURSE
GRAVELLY SAND WITH 6.5% CEMENT FILLER (2.9% AC)

SURFACE MIX (5.0% AC) BASE COURSE

SURFACE MIX (2.9% AC) BASE COURSE

BITUMINOUS STABILIZED GRAVELLY SAND SUBBASE (2.9%)

UNSTABILIZED GRAVELLY SAND SUBBASE

HEAVY CLAY (CH) SUBGRADE, CBR = 4

PROFILE

LEGEND
O 4-IN.-DIAM SOIL STRAIN SENSORS
△ THERMISTOR PROBES (DUPLICATE)
WHEEL ARRANGEMENT
360-KIP, 12-WHEEL ASSEMBLY
(ONE MAIN GEAR OF C-5A)
a. 360-KIP 12-WHEEL ASSEMBLY

b. 75-KIP SINGLE-WHEEL ASSEMBLY

TRAFFIC PATTERNS

RI00871SC

PLATE 10
TEMPERATURE VS TIME
TYPICAL COOL DAY
22 JULY 1970
TEMPERATURE VS TIME
TYPICAL HOT DAY
20 AUGUST 1970

LEGEND
- - - - AMBIENT
  3-IN. DEPTH
  6-IN. DEPTH
  9-IN. DEPTH
TEMPERATURE VS TIME
10:00 AM READINGS

LEGEND

- - - - AMBIENT
- - - - 3-IN. DEPTH
- - - - 18-IN. DEPTH
- - - - 24-IN. DEPTH

DATE, AUGUST 1970
LEGEND

- AMBIENT
- 3-IN. DEPTH
- 18-IN. DEPTH
- 24-IN. DEPTH

TEMPERATURE VS TIME
4:00 PM READINGS

PLATE 14
LEgend

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<td>75</td>
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<td>06-08</td>
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* 24 HR AFTER PLACING 6-IN.-THICK LAYER OF AC BASE.
† 4 HR AFTER PLACING TOP 3 IN. AC SURFACE LAYER.

TEMPERATURE VS DEPTH
24-IN.-THICK BITUMINOUS PAVEMENT
**a. PARALLEL TO TRAFFIC**

**b. TRANSVERSE TO TRAFFIC**

**TOTAL DEFLECTION**

**PRIOR TO TRAFFIC**

**360-KIP 12-WHEEL ASSEMBLY**
a. DEFLECTION MEASURED TRANSVERSE TO TRAFFIC

b. DEFLECTION AND REBOUND MEASURED TRANSVERSE TO TRAFFIC (242 COVERAGES)

TOTAL DEFLECTION AND REBOUND DURING TRAFFIC ON ITEM 3
360-KIP 12-WHEEL ASSEMBLY
CROSS SECTIONS
ITEM 3, LANE 1
TOTAL DEFLECTION
PRIOR TO TRAFFIC
TRANSVERSE TO TRAFFIC
75-KIP SINGLE-WHEEL ASSEMBLY

PLATE 22
ELASTIC DEFLECTIONS
TRANSVERSE TO TRAFFIC
75-KIP SINGLE-WHEEL ASSEMBLY
ITEM 3 AFTER 10 COVERAGES
TEST PIT PROFILE
ITEM I, STA 3+50
AFTER 98 COVERAGES
360-KIP 12-WHEEL ASSEMBLY
TEST PIT PROFILE
ITEM 2, STA 3+20
AFTER 425 COVERAGES
360-KIP 12-WHEEL ASSEMBLY
TEST PIT PROFILE
ITEM 4, STA 2+58
AFTER 1408 COVERAGES
360-KIP 12-WHEEL ASSEMBLY
TRAFFIC COVERAGES TO FAILURE

TOTAL THICKNESS, IN.

LEGEND

\[ \text{MWHGL TEST SECTION, ITEM NUMBER} \]
\[ \text{BITUMINOUS STABILIZATION STUDY, ITEM NUMBER} \]

NOTE: ITEM 5 OF MWHGL TEST SECTION DID NOT FAIL.

COVERAGES VS THICKNESS
12-WHEEL GEAR
360,000-LB GEAR LOAD
100-PSI TIRE PRESSURE
TRAFFIC COVERAGE TO FAILURE

LEGEND

○ MWHGL TEST SECTION, ITEM NUMBER

• BITUMINOUS STABILIZATION STUDY, ITEM NUMBER

COVERAGES VS THICKNESS
75,000-LB SINGLE-WHEEL LOAD
290-PSI TIRE PRESSURE
STUDY OF BEHAVIOR OF BITUMINOUS-STABILIZED PAVEMENT LAYERS

The investigation reported herein was conducted to (a) compare the performance of bituminous-stabilized base and subbase materials with that of unbound granular materials as used in the original multiple-wheel, heavy gear load (MWHGL) test section and (b) determine the difference in performance between a high quality bituminous base constructed of crushed aggregate and a bituminous base constructed of a lower quality uncrushed material. A test section was constructed within the existing MWHGL test section at the U. S. Army Engineer Waterways Experiment Station (WES), utilizing the existing 4.0-CBR clay subgrade. The test section consisted of four test items. Items 1 and 2 were constructed to a thickness of 15 in., and items 3 and 4 to a total thickness of 24 in. In item 1, the granular base and subbase used in the original construction were replaced by a bituminous-stabilized base constructed of the uncrushed gravelly-sand subbase material used in the original MWHGL test section. Cement filler of 6.5 percent was used with the aggregate to improve the gradation. Item 2 was identical with item 1, except for the 12-in. base, which was constructed of a high quality asphaltic concrete containing crushed limestone. In item 3, the unbound crushed-stone base used in the MWHGL test section was replaced by a high quality asphaltic-concrete base, and the gravelly-sand subbase in the bottom 15 in. of the structure was stabilized with asphalt cement. Item 4 was identical with item 3, except that the gravelly sand was not stabilized. A 3-in.-thick surface layer of high quality asphaltic concrete was constructed over all test items. The test items were subjected to traffic with a simulated C-5A main gear 12-wheel assembly with a 350,000-lb gross load and with a 75,000-lb single-wheel assembly. The results of tests showed that: the performance of the bituminous-bound base and subbase materials was superior to that of similar pavements constructed of unbound granular materials used in the original MWHGL test section at the WES, the quality of aggregate used in the bituminous base courses had a significant effect on pavement performance, and the greatest benefit from bituminous stabilization was in upgrading the quality of poor-to-borderline materials.
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