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EVALUATION OF STRUCTURAL LAYERS IN FLEXIBLE PAVEMENT

Army Engineer Waterways Experiment Station

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Office of the Chief of Engineers

May 1973

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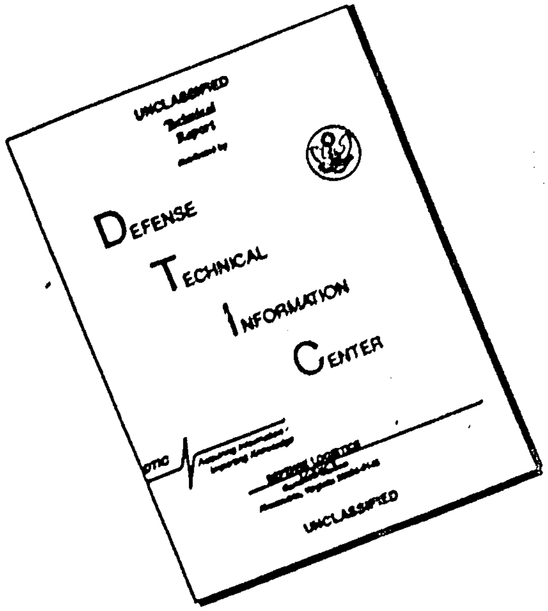

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MISCELLANEOUS PAPER S-73-26

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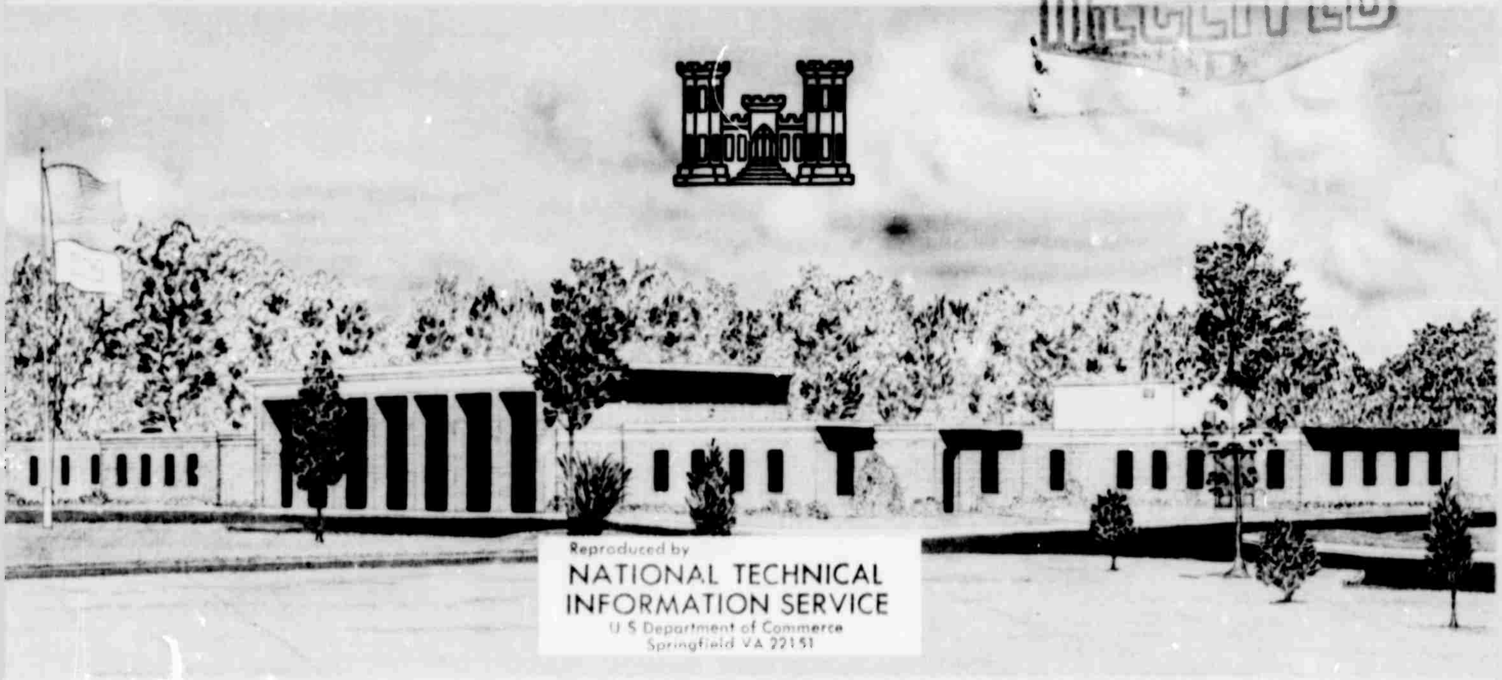
by

R. W. Grau



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13. ABSTRACT

The investigation reported herein was conducted to evaluate the effectiveness of stabilized structural layers (lime- and cement-stabilized layers) in pavement performance and to determine the comparative performance between a full-depth high-quality crushed stone and the stabilized layers during simulated aircraft traffic. The comparative performance between the stabilized layers and similar pavements consisting of unbound granular base and subbase materials previously tested in the Multiple Wheel Heavy Gear Load (MWHGL) test section was also determined. A test section was constructed within two items of the existing MWHGL test section at the U. S. Army Engineer Waterways Experiment Station in order to utilize the existing 4-CBR clay subgrade. The test section consisted of four 24-in.-thick items. The structural layers above the subgrade for the respective items were: item 1, a 15-in.-thick lime-stabilized lean clay layer overlaid with 6 in. of crushed stone and 3 in. of asphaltic concrete (AC); item 2, a 15-in.-thick cement-stabilized lean clay layer overlaid with 6 in. of crushed stone and 3 in. of AC; item 3, a 21-in.-thick crushed stone base and 3 in. of AC; and item 4, a 21-in.-thick cement-stabilized clayey gravelly sand layer overlaid with 3 in. of AC. Items 1 and 2 were trafficked with a 360-kip 12-wheel assembly, a 160-kip twin-tandem assembly, and a 50-kip single-wheel assembly; items 3 and 4 were trafficked with a 200-kip twin-tandem assembly and a 75-kip single-wheel assembly. Mixed traffic was applied to item 4 with the 360-kip 12-wheel and 75-kip single-wheel assemblies. The test items utilizing stabilized structural layers as elements in the flexible pavement performed as well as or better under traffic than the same thicknesses of conventional pavement previously tested in the MWHGL test section.

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FOREWORD

The investigation reported herein was sponsored by the Office, Chief of Engineers, U. S. Army, as a part of the Military Engineering Design and Expedient Construction Criteria Program, Task 02, "Design Criteria for Expedient Airfields and Heliports," Work Unit 024 (formerly 023), "Exploitation of Thin Structural Layers."

Responsibility for conducting the investigation was assigned to the Soils and Pavements Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES). The investigation reported herein was conducted from October 1970 to April 1971.

The investigation was conducted under the general supervision of Messrs. J. P. Sale and R. G. Ahlvin, Chief and Assistant Chief, respectively, of the Soils and Pavements Laboratory. Engineers of the Soils and Pavements Laboratory actively engaged with the planning, testing, analyzing, and reporting phases of this study were Messrs. R. L. Hutchinson, C. D. Burns, W. N. Brabston, R. W. Grau, and R. H. Ledbetter. Engineering technicians responsible for the conduct of the tests were Messrs. J. E. Watkins and B. R. King. This report was written by Mr. Grau; portions, regarding instrumentation, were written by Mr. Ledbetter.

COL Ernest D. Peixotto, CE, was the Director of the WES during the conduct of this study and the preparation of this report. 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:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
square inches	6.4516	square centimeters
pounds (mass)	0.45359237	kilograms
kips	453.59237	kilograms
tons	907.18474	kilograms
gallons (U. S. liquid) per square yard	0.004527	cubic meters per square meter
pounds (force) per square inch	0.6894757	newtons per square centimeter
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
miles per hour	1.609344	kilometers per hour
Fahrenheit degrees	5/9	Celsius or Kelvin degrees*

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

SUMMARY

The investigation reported herein was conducted to evaluate the effectiveness of stabilized structural layers (lime- and cement-stabilized layers) in pavement performance and to determine the comparative performance between a full-depth high-quality crushed stone and the stabilized layers during simulated aircraft traffic. The comparative performance between the stabilized layers and similar pavements consisting of unbound granular base and subbase materials previously tested in the Multiple Wheel Heavy Gear Load (MWHGL) test section was also determined.

A test section was constructed within two items of the existing MWHGL test section at the U. S. Army Engineer Waterways Experiment Station in order to utilize the existing 4-CBR clay subgrade. The test section consisted of four 24-in.-thick items. The structural layers above the subgrade for the respective items were: item 1, a 15-in.-thick lime-stabilized lean clay layer overlaid with 6 in. of crushed stone and 3 in. of asphaltic concrete (AC); item 2, a 15-in.-thick cement-stabilized lean clay layer overlaid with 6 in. of crushed stone and 3 in. of AC; item 3, a 21-in.-thick crushed stone base and 3 in. of AC; and item 4, a 21-in.-thick cement-stabilized clayey gravelly sand layer overlaid with 3 in. of AC.

Items 1 and 2 were trafficked with a 360-kip 12-wheel assembly, a 160-kip twin-tandem assembly, and a 50-kip single-wheel assembly; items 3 and 4 were trafficked with a 200-kip twin-tandem assembly and a 75-kip single-wheel assembly. Mixed traffic was applied to item 4 with the 360-kip 12-wheel and 75-kip single-wheel assemblies.

The test items utilizing stabilized structural layers as elements in the flexible pavement performed as well as or better under traffic than the same thicknesses of conventional pavement previously tested in the MWHGL test section.

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EVALUATION OF STRUCTURAL LAYERS IN FLEXIBLE PAVEMENT

PART I: INTRODUCTION

Background

1. Recent tests conducted at the U. S. Army Engineer Waterways Experiment Station (WES) have shown that flexible pavements constructed using asphalt-stabilized subbases and base courses and membrane-enveloped soil layers (MESL) over a low-strength clay subgrade sustained considerably more traffic of a given loading than did comparable sections of the same total thickness of conventional pavement in which untreated granular base and subbase materials were used.[1,2] This performance indicates that the quality of construction materials used in the layered system has a significant effect on pavement behavior. Therefore, it was decided to evaluate the effectiveness of lime- and cement-stabilized layers in pavement performance and to determine comparative performance between a full-depth high-quality crushed stone and the stabilized layers. The same type wheel configurations, which simulated multiple-wheel gears of large new aircraft (such as the C-5A and Boeing 747), were used on the structural layer test sections as were used during the recent Multiple Wheel Heavy Gear Load (MWHGL) tests conducted at the WES.[3]

Objective

2. The objective of this investigation was to investigate the relationship between the strength of stabilized soil layers incorporated in pavement systems and the load-carrying ability of such systems.

Scope

3. The objective of this investigation was accomplished by the

construction and traffic testing of a specially designed test section consisting of four items as described herein. This report describes the test section, traffic testing, and results of the tests.

PART II: TEST SECTION

Design

General

4. In the recently completed[3] MWHGL test section, to which these tests relate, the total thicknesses of the flexible pavement test items were based on medium-load pavement requirements[4] with reductions in thickness so that failures would occur at traffic-volume levels that normally could be expected during a period of from a few weeks to several years on a prototype airfield. From the results of these tests and all prior pavement behavior data, it was determined for the purpose of this test section that a total thickness of about 24 in.* was adequate to provide a test pavement that would generally fail at a practical coverage level and yet yield sufficient data for analysis. The primary variable being evaluated in this test section was the structural layer and type stabilizer and/or material type used for subbase and base course layers.

Description

5. The structural layer test section was located in the west portion of the MWHGL test section where items 1 and 2 of the original test section had been constructed (plate 1). The bituminous base course and MESL test sections, which were constructed and tested after the MWHGL test section, also had been located in about this same area. The maneuver area on the west end was reconstructed; test item 3 of the original MWHGL test section was used as the maneuver area on the east end.

6. A plan and profile of the test section are shown in plate 2. The test section was 120 ft long and 60 ft wide and consisted of four items, each 30 ft long and 60 ft wide. All four items had total thicknesses of 24 in. and were constructed over a heavy clay subgrade having an initial strength of about 4 CBR. Grades were established for a

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

transverse slope of 0.75 percent from west to east. These grades allowed the test section to tie in with the remnants of the MWHGL test section. A detailed description of each item is as follows:

Location	Construction
Item 1	3-in. asphaltic concrete 6-in. crushed stone base 15-in. lean clay subbase stabilized with 3.5 percent lime
Item 2	3-in. asphaltic concrete 6-in. crushed stone base 15-in. lean clay subbase stabilized with 10 percent cement
Item 3	3-in. asphaltic concrete 21-in. crushed stone base
Item 4	3-in. asphaltic concrete 21-in. clayey gravelly sand base stabilized with 6 percent cement

Heavy clay subgrade

7. The subgrade of the MWHGL test section was used for this test section. It consisted of a heavy clay (CH) material having a liquid limit (LL) of 73, plastic limit (PL) of 25, and plasticity index (PI) of 48. The heavy clay material was classified as a CH soil according to the Unified Soil Classification System (USCS).[5] Classification data for this soil are shown in plate 3. Laboratory compaction and CBR data for the as-molded and soaked conditions are shown in plates 4 and 5. These data indicate a CBR of about 4 to 5 at molding water contents of from 30 to 32 percent for both the as-molded and soaked conditions.

Subbase courses

8. Hydrated lime- and portland cement-stabilized lean clay subbase courses, 15 in. thick, were constructed in items 1 and 2, respectively. Prior to stabilization, the soil that was stabilized was classified as a lean clay (CL) according to the USCS with a LL of 34 and PI of 12. Classification data for the lean clay are shown in plate 3.

9. Laboratory compaction and CBR tests were performed on the untreated, lime-treated, and cement-treated lean clay. The compaction requirements for the lean clay when used as a subgrade or subbase in

airfield construction would normally be about equal to a CE 12 compaction effort;[6] therefore, the CE 12 laboratory compaction effort was used in preparing the specimens for tests. In previous soil stabilization studies conducted at WES with lime and cement stabilization, unconfined compression tests have been used for evaluating the strength of the stabilized materials. Typical data, taken from a contract study conducted for WES,[7,8] showing the relation between water content, dry density, and unconfined compressive strength for the lean clay treated with 3, 6, and 10 percent cement are given in plate 6. The linear relationship between log CBR values and log unconfined compressive strength values determined in the same contract study is shown in plate 7. For this study the CBR test was used as an indicator for measuring the strength of the untreated and lime- and cement-treated lean clay soil.

10. Laboratory tests were performed on soil and lime-soil specimens to determine the amount of lime needed to raise the pH of the lime-soil to 12.4 and the lime content to a level above which further strength increases would not be significant.[9] Density (CE 12 compaction) and CBR tests were performed on untreated and treated (3 and 5 percent lime) specimens in the as-molded unsoaked and after-soaking conditions (plate 8). These data showed that an untreated specimen compacted at 16 percent water content resulted in an as-molded strength of about 48 CBR as compared to 92 and 99 CBR for specimens with lime contents of 3 and 5 percent, respectively. After a four-day soaking period, the strength of the untreated specimen was reduced to about 10 CBR, showing the sensitivity of the soil to water saturation. However, the CBR's of the specimens treated with lime and molded at about 16 percent water content were between 90 and 95 after a four-day soaking period. Based on the results of these tests and the possibility of loss of lime during the mixing process, a lime content of 3.5 percent was selected to stabilize the lean clay.

11. Plate 8 also shows compaction and CBR data for soil specimens treated with 5, 7, and 10 percent portland cement. These cement contents were selected by following the procedure for estimating cement requirements for expedient subgrade construction described in reference 9.

The as-molded strengths of the specimens treated with 5 and 10 percent cement and compacted at a water content of 16 percent ranged from 210 to 315 CBR. After a four-day soaking period, the strengths increased to a range of 242 to 355 CBR. Based on the results of these laboratory tests, a design cement content of 10 percent was used to stabilize the lean clay subbase of item 2.

Base course

12. The material used for the conventional base course in the west maneuver area and items 1-3 of the test section was a crushed limestone that met the requirements of Guide Specification CE 807.07.[10] Classification data are shown in plate 3.

13. A 21-in.-thick portland cement-stabilized clayey gravelly sand base course was constructed in item 4. This material, classified as SP-SC according to the USCS,[5] had a LL of 23 and PI of 11 (plate 3). Laboratory compaction and CBR data for the untreated soil are shown in plate 9. These data indicated that maximum density was obtained at water contents between 7.5 and 9 percent, depending on the compaction effort. After the optimum water content was determined for the untreated soil, laboratory specimens were then prepared at cement contents of 5, 7, and 10 percent. These cement contents were selected using the procedure described in reference 9. Duplicate cement-treated specimens were compacted, using the CE 55 effort only, at optimum water content (7.5 percent) and at two percent wet of optimum (9.5 percent). Only the CE 55 compaction effort was used, because compaction requirements for an SP-SC soil when used as a base material in airfield construction would normally be equal to the CE 55 compaction effort. One test specimen at each molding water content and percent cement treatment was tested for CBR after a seven-day humid cure, and then the duplicate specimen was tested after a seven-day curing plus a four-day soaking period. The results of these tests are shown in plate 10. Both the compaction and CBR data in plate 10 showed that higher densities and strengths were obtained when the water content of the soil prior to the treatment with portland cement was 7.5 percent and after a four-day

soaking period. A design cement content of 6 percent was selected to stabilize the clayey gravelly sand.

Asphaltic concrete

14. A mix design for the asphaltic-concrete surfacing layer was prepared utilizing 3/4-in. maximum-size crushed limestone, sand filler, and 85-100 penetration grade asphalt. The limestone was obtained in two sizes: 3/4-in. to No. 4 aggregate and minus No. 4 screenings. Average gradation curves for the two limestone materials and the sand filler are shown in plate 11. The gradation curve of the blended stockpile aggregates used for the asphaltic-concrete mixture and the gradation specification limits are shown in plate 12. The gradation limits were taken from table II, gradation 11 of Guide Specification CE 807.22.[11] Laboratory mix design properties are shown in plate 13. From these data, a design asphalt content of 4 percent was selected for the asphaltic-concrete mixture.

Instrumentation

15. All four test items were instrumented with stress gages (pressure cells) and strain sensors, as shown in plate 14. Stresses were measured only at the top of the subgrade, but movements within the soil-pavement system were measured at depths of from 3 to 33 in. Vertical movement as well as horizontal movements in two directions were measured in the subbase and subgrade of items 2 and 4. Details of the instrumentation, installation of sensors, and collection and reduction of instrumentation data are given in Appendix A.

Construction

General

16. Excavation, construction of the structural layers, and final paving phases of the test section occurred during October and November 1970. First, excavation to a depth of approximately 24 in. below the existing grade was accomplished in the existing MESL and bituminous base test sections, which were located in items 1 and 2 of the original MWHGL flexible pavement test section. This excavated area was 60 ft wide and

120 ft long. The material to be removed in this area was pushed up with a D-6 dozer, end loaded into a dump truck, and hauled away. The excavation operation is shown in photo 1.

Subgrade

17. The existing subgrade at the test site was the heavy clay described in paragraph 7. This subgrade, which was constructed for the MWHGL test section and was later used for the MESL and bituminous base test sections, had a strength of about 5.2 CBR at the surface. Therefore, the upper 6 in. of material was reprocessed to reduce the strength to about a CBR of 4 and then fine bladed to the design elevation of 24 in. below finished grade. Photo 2 shows the subgrade after fine blading and preparation of the trenches for instrumentation cables.

Subbase and base courses

18. Stabilized soil. The structural layers of items 1-4 were constructed concurrently. Prior to applying a stabilizing agent, the subbase and base materials to be stabilized were processed to approximately the desired water content and then placed in the various test items in approximately 6-in.-thick loose layers, which resulted in 5-in. compacted lifts. After placement of the loose materials for each lift, bags of lime or cement* were placed in the respective items at predetermined intervals to give the desired amount of stabilization (photo 3). The lean clay in items 1 and 2 was stabilized with 3.5 percent lime and 10 percent cement, respectively. The clayey gravelly sand in item 4 was stabilized with 6 percent cement. The stabilizing agent was spread in each item and then thoroughly mixed to a depth of about 6 in. with a pulvimixer (photo 4).

19. After the lime or cement was mixed in the soil and prior to compaction of each lift, water content determinations were made. The average results of these data were 17.6, 16.7, and 7.0 percent for items 1, 2, and 4, respectively.

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* The lime used as a stabilizing agent in item 1 was high-calcium, normal hydrated, Type N, meeting ASTM Specification C-207. Type 1 normal portland cement conforming to Federal Specification SS-C-192b was used in items 2 and 4.

20. To prevent rutting of the subgrade, a relatively light roller was used for compaction of the bottom two layers of material (approximately 10 in.). The roller was a self-propelled, 30-ton, seven-wheeled, rubber-tired roller (photo 5) with a tire pressure of 90 psi. The third layer, 15 in. above the subgrade, was compacted with eight coverages of a 50-ton, four-wheeled, rubber-tired roller with a tire inflation pressure of 150 psi.

21. The average as-constructed water content of the subbase in items 1 and 2 was 17.6 and 15.7 percent, respectively, which resulted in an initial dry density of 106.9 pcf and a CBR of 80 for item 2. A water content of 4.5 percent, dry density of 133.7 pcf, and strength of 285 CBR were measured for the cement-stabilized base of item 4 at a depth of 9 in. below the finished grade. The upper 6-in. portion of the cement-stabilized base in item 4 was placed following completion of the crushed stone base in items 1-3. The water content, dry density, and strength of this portion were 2.7 percent, 139.8 pcf, and 202 CBR, respectively. A summary of the as-constructed soil data is presented in table 1.

22. <u>Crushed stone.</u> Three different sizes of crushed stone (1-1/2-in. maximum, 3/4-in. maximum, and screenings) were proportioned through the cold-bin feeder at the asphalt plant to produce the gradation shown in plate 3. The material was saturated in a surgebin hopper prior to loading on dump trucks for transport to the section and placement in items 1-3. The lifts of crushed stone in the lower 15 in. of item 3 were placed concurrently with the lower lifts in items 1, 2, and 4. The base course in items 1 and 2 and the upper portion of item 3 were placed in one lift of approximately 6 in. by use of a spreader (photo 6). The as-constructed water content, dry density, and strength of the crushed stone base in item 3 at a depth of approximately 9 in. below finished grade were 1.7 percent, 140.6 pcf, and 35 CBR, respectively. Compaction was accomplished, as shown in photo 7, with 8 coverages of the 30-ton and 30 coverages of the 50-ton, rubber-tired rollers described in paragraph 20. As shown in table 1, the strength of the surface of the crushed stone base in items 1-3 ranged

from 132 to 200 CBR. After compaction was completed, the section was primed (see photo 8) with approximately 0.4 gal per sq yd of MC-1 cutback asphalt.

Asphaltic concrete

23. The asphaltic concrete for the wearing course was mixed in a central hot-mix batch plant at WES and was placed with a Barber-Greene asphalt finisher in 10-ft-wide longitudinal lanes (photo 9). Placement temperature of the mixture was about 300 F. The wearing course, which was about 3 in. thick after compaction, was placed in one lift. Shortly after placement, the mixture was compacted by breakdown rolling with a 10-ton tandem steel-wheel roller, followed by 10-12 coverages of a 30-ton self-propelled, rubber-tired roller with tire inflation pressure of 90 psi (see photo 10). Finish rolling was accomplished with the tandem steel-wheel roller.

24. A summary of stability, flow, voids, and density data for laboratory- and field-compacted asphaltic-concrete specimens is shown in table 2. Data from the field-compacted mixture are for cores cut immediately after compaction and after various coverages of traffic with the 12-wheel assembly. These data will be discussed in more detail later in this report.

Instrumentation installation

25. Installation of the instruments was accomplished in conjunction with the construction operations of the test sections. The installation is described in Appendix A.

PART III: TRAFFIC TESTS AND RESULTS

Test Conditions and Procedures

General

26. Traffic tests were performed on the lanes indicated in plate 2 from December 1970 to March 1971. Three specially designed test carts were used to traffic the lanes. A description of the test carts, traffic patterns, failure criteria, and performance of the test section during traffic are discussed in the following paragraphs.

Test carts

27. The 12-wheel-assembly test cart shown in photo 11 was used to traffic lane 1. This assembly represented one main gear of the C-5A aircraft. The cart was powered by a prime mover with electric drive wheels and was operated in such a manner that these drive wheels did not traffic the test lane. The 12-wheel assembly consisted of two load boxes, each of which was carried by six load wheels, resulting in the 12-wheel arrangement shown in plate 15. The boxes were loaded to a net weight of 360,000 lb, which was distributed equally over the 12 wheels. Each test wheel was equipped with 49x17, 26-ply rating tires inflated to 100 psi, resulting in a tire contact area of 285 sq in. per tire and a contact pressure of 106 psi.

28. Twin-tandem-assembly traffic was applied using the test cart shown in photo 12. The wheel spacing, shown in plate 15, typified one twin-tandem component of the 747 aircraft assembly. The test cart consisted of a load box supported by an A-frame and was towed by a Caterpillar Model 619 tractor. The load box, which was carried by the four test wheels equipped with 49x17, 26-ply rating tires, was loaded to a net weight of 160,000 lb (40,000 lb per wheel) or 200,000 lb (50,000 lb per wheel), depending on the items trafficked. The 200,000-lb load was selected for items 3 and 4 so failure would occur at a reasonable number of coverages. At the 40,000-lb-per-wheel load, the tires were inflated to 140 psi, giving a contact area of about 290 sq in. and an average contact pressure of 138 psi. When the test wheels were loaded to

50,000 lb per wheel, the tire inflation pressure was raised to 180 psi, which resulted in an average contact area of about 285 sq in. and a contact pressure of approximately 176 psi.

29. The 50,000- and 75,000-lb single-wheel assemblies 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-ply rating tire. The tire on the test wheel was inflated to 170 psi for the 50,000-lb load and to 290 psi for the 75,000-lb load. The resulting tire contact area and average contact pressure for the 50,000-lb loaded test wheel were 280 sq in. and 179 psi, respectively. A contact area of 270 sq in. was measured when the test wheel was loaded to 75,000 lb and inflated to 290 psi, which resulted in an average contact pressure of 278 psi.

Test lanes and traffic patterns

30. Plate 2 shows the location, width, and length of each lane trafficked and the assembly used to traffic the lane. The lanes are identified according to the number of wheels and net weight of the assembly. Except for the additional 75-kip single-wheel-assembly traffic applied in the center of the 360-kip 12-wheel lane of item 4, each lane consisted of a portion of the test section on which no traffic had previously been applied.

31. Lane 1 was 200 in. wide and 120 ft long (plate 2). All four items were trafficked at an assembly net weight of 360,000 lb. 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 width). The distribution of traffic coverages* over the 200-in.-wide traffic lane, after one complete pattern of traffic, is shown in plate 16. To apply a traffic pattern, the test cart first traveled forward for the full length of the test lane along guideline 1 (south side of traffic lane) and backward along the same line; then the cart was shifted laterally to run the adjacent line. After tracking line 5 at

* 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.

the north side of the lane, 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 going from north to south. This procedure 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.

32. Twin-tandem-assembly traffic was distributed over lane 2, as shown in plate 16, by following five guidelines, which were painted on the pavement. Items 1 and 2 were trafficked with an assembly weight of 160,000 lb, and items 3 and 4 were trafficked with an assembly weighing 200,000 lb. To apply the traffic over the 10-ft-wide lane (plate 16), the load cart first traveled forward for the full length of the test lane along guideline 1 (south edge of the traffic lane) and then backward along the same line. The cart was then shifted laterally to run the adjacent line in the same manner. This procedure was followed for all five guidelines, which positioned the load cart on the north edge of the traffic lane. To obtain the desired traffic distribution shown in plate 16, the procedure used for trafficking lines 1-5 was repeated three times for lines 2-4, and then two additional passes were applied with the test cart following guideline 3. This procedure completed one pattern of traffic for the twin-tandem assembly. A total of 30 passes of the test cart was required to apply one pattern of test traffic. Each pattern of traffic resulted in 20 coverages of a test wheel over the center 60 in. of the test lane, 16 coverages over the adjacent 15 in., and 4 coverages on the exterior 15-in. portions.

33. Lane 3 was 98 in. wide. Items 1 and 2 were trafficked with a 50,000-lb single-wheel load, and items 3 and 4 were trafficked with a 75,000-lb single-wheel load. In the application of traffic, the vehicle was driven forward and backward along the same path (one of seven guidelines), then shifted laterally a distance equal to one tire print width, 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 over the test lane. Traffic was applied in an approximately normal distribution pattern, as shown in plate 16. The interior 42 in. of the traffic lane received 100 percent of the applied traffic, and the exterior portions of the lane received 80 and 20 percent, as shown. Single-wheel-assembly traffic was applied in the same manner to the 98-in. interior portion of item 4, lane 1, after completion of 12-wheel-assembly traffic.

Pavement temperature

34. Trafficking of the test section commenced during December 1970 and continued through March 1971. The average pavement temperature, as determined from measurements at the surface and the bottom of the asphaltic-concrete layer, ranged between 40 and 92 F.

35. A traffic and pavement temperature distribution curve for each of the traffic lanes is shown in plate 17. These curves were derived from records of the pavement temperature and number of coverages made hourly during traffic. As can be seen in plate 17, the 12-wheel-assembly traffic was applied when the pavement temperature was coolest (between 40 and 77 F), and then the additional 75-kip single-wheel traffic in the 12-wheel lane of item 4 was applied when the pavement temperature was 15 to 25 F warmer. The test section was trafficked by the two twin-tandem assemblies when the pavement temperature was between 47 and 85 F and by the single-wheel assemblies when the pavement temperature was between 54 and 92 F.

Failure criteria

36. In judging failure of the test items, distinction was made between settlement due to traffic compaction and distortion due to shear deformation. Settlement, as the result of densification of the base and subbase under accelerated traffic, was anticipated because it was not possible to apply a maximum compaction effort on the bottom layers of the subbase directly above the weak clay subgrade. The term "shear deformation" as used herein refers to excessive plastic movement or, in the extreme, to rupture of any element in the pavement structure.

37. A pavement item was considered failed when either of the following conditions occurred:

a. Upheaval in excess of 1 in. of the pavement surface adjacent to the traffic lane.

b. Surface cracking to the extent that the pavement was no longer waterproof.

Collection of Instrumentation Data

38. Static loading tests were made with the 30-kip-per-wheel single- and 12-wheel assemblies before any traffic was allowed on the test section. Twin-tandem and single-wheel 50-, 60-, and 75-kip load tests were made after 10,000 coverages of the 12-wheel assembly on item 4. Data were recorded under static test loads at various coverage levels, under special moving load tests (runs about 3-5 mph down selected rows), and under all of the 12-wheel traffic (3-5 mph) on each item. Recorded data for 12-wheel traffic to and past failure ranged from 201 coverages on item 1 to 10,000 coverages on item 4 and an additional 200 coverages of 75-kip single-wheel traffic on the 12-wheel lane of item 4. Details of the collection of instrumentation data are given in Appendix A.

Behavior of Pavement Under Traffic

39. Observations of the behavior of the test items were recorded throughout the traffic test period. These observations were supplemented by photos. Level readings were taken on the pavement prior to and at intervals during traffic to show the development of permanent deformation of the pavement under the assembly load for the lane being observed. After failure, a thorough investigation was made by excavating test trenches across the traffic lanes and by establishing profiles of the surface of the various layers in the structure, along with CBR measurements and other pertinent tests to determine where failure had occurred. The behavior of each item under traffic is summarized below. The data obtained during the traffic tests are presented in Appendix B.

Item 1

40. The pavement structure in item 1 was composed of 3 in. of

asphaltic concrete over a 6-in. crushed stone base with a 15-in. subbase of lean clay stabilized with 3.5 percent lime. The behavior of this pavement was as follows:

Assembly	Load, kips Per Tire	Load, kips Total	Coverages to Failure
12 wheel	30	360	198
Twin tandem	40	160	140
Single wheel	50	50	40

Item 2

41. The pavement structure in item 2 was the same as that of item 1, except that the lean clay subbase was stabilized with 10 percent cement. The performance of this item is summarized below:

Assembly	Load, kips Per Tire	Load, kips Total	Coverages to Failure
12 wheel	30	360	1200
Twin tandem	40	160	1000
Single wheel	50	50	120

Item 3

42. Item 3 consisted of 3 in. of asphaltic concrete over a 21-in.-thick (full depth) crushed stone base. This item performed as follows:

Assembly	Load, kips Per Tire	Load, kips Total	Coverages to Failure
12 wheel	30	360	5000
Twin tandem	50	200	890
Single wheel	75	75	50

Item 4

43. The pavement structure of item 4 was composed of 3 in. of asphaltic concrete over a 21-in. base course of clayey gravelly sand stabilized with 6 percent cement. The behavior of this pavement was as follows:

Assembly	Load, kips Per Tire	Total	Coverages to Failure
12 wheel	30	360	10,406*
Twin tandem	50	200	1,810
Single wheel	75	75	120
Mixed-12 wheel	30	360	
-Single wheel	75	75	200**

* Traffic discontinued, pavement in satisfactory condition.

** Same area trafficked by 10,406 coverages of 12-wheel assembly. Failed by 200 coverages of 75-kip single-wheel load.

Summary

44. A summary of the traffic test results for the various loading conditions on the test section is shown in table 3. Most of these data are self-explanatory; however, some columns need further explanation as given in the following paragraphs.

45. Rated subgrade CBR. The rated CBR values of the subgrade were based on the numerical average of the CBR values measured immediately after construction and after traffic (table 1). The CBR values used were obtained from tests conducted at the surface of the subgrade and at depths of 6 and 12 in. in the subgrade. All values obtained in a given test item from the various traffic lanes were used in the averages for rating the strength of the test item. In general, the CBR of the subgrade was quite uniform in each test item; the rated subgrade CBR values were 5.0, 4.3, 4.3, and 4.2 for test items 1-4, respectively.

46. Deflection. The deflection values shown in table 3 represent the maximum total deflection extrapolated from the measured values taken prior to traffic testing and at the coverage level indicated.

47. Maximum permanent deformation. The values listed in table 3 were obtained from cross-section elevation measurements taken on the pavement surface prior to traffic and at the coverage level indicated.

48. Upheaval. The upheaval values tabulated were obtained from cross-section elevation measurements taken prior to traffic and at the coverage level 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

measurements of 1 in. or more were made.

49. Pavement cracking. Pavement cracking extending through the 3-in.-thick asphaltic concrete was a condition considered in the failure criteria. Where pavement cracking is described in table 3 as "severe," this condition (i.e., pavement cracking through the 3-in.-thick asphaltic concrete) existed, and the items were evaluated as failed. "Slight" cracking denotes narrow cracks that did not extend through the asphaltic concrete layer.

50. Rating of test items. It can be noted that pavement failure developed in all test items with the exception of item 4 when subjected to the 360-kip 12-wheel-assembly traffic. Traffic was discontinued in this item prior to failure, because the performance of the item during traffic, along with no increase in deflection, indicated that a large number of coverages would be required to produce failure. From failure-investigation test trenches, it was determined that failure of the items was primarily from fatigue cracking of the wearing surface due to high deflections of the pavement structure. There was no distinct evidence of subgrade shear deformation in any of these items after failure. Some consolidation of the crushed stone base material in items 1-3 occurred during traffic. Severe alligator cracking, which was indicative of excessive movement of one or more of the underlying layers and/or fatigue of the surface, was present after all failures. At failure, this type of cracking had reached the point that the pavement was no longer waterproof.

PART IV: ANALYSIS OF TEST RESULTS

Service Life Versus CBR

51. A relationship between traffic coverages to failure (service life) and the as-constructed soil strength (CBR) of the stabilized subbase courses in items 1 and 2, the base course in item 3, and the stabilized base course in item 4 is shown in plate 18. The subbase of item 1 was stabilized with lime, while cement was used to stabilize the subbase of item 2 and base of item 4. The base course of item 3 consisted of crushed limestone and was not chemically stabilized. Plate 18 shows that the number of coverages to failure increased as the as-constructed CBR of the structural layers increased and that item 4, for which the CBR of the stabilized base was initially 285, was in satisfactory condition when traffic was discontinued at 10,406 coverages.

Performance Comparisons

52. Number of coverages to failure versus thickness for flexible pavement items subjected to traffic of the 360-kip 12-wheel assembly are shown in plate 19. This comparison includes data from the MWHGL and the structural layer test sections.

53. The pavement items in the MWHGL test section all consisted of a 3-in.-thick layer of asphaltic concrete and 6 in. of crushed stone base over various thicknesses of gravelly sand subbase with a 4-CBR clay subgrade (plate 1). The pavement, base, and subbase materials all met current CE design quality requirements for the various elements of the pavement structure; the MWHGL test section was considered to represent conventional pavement construction.

54. All of the points plotted in plate 19 for the MWHGL test section are for failure conditions, except for item 5, which was still in satisfactory condition at the end of traffic after 3850 coverages. This plot shows a straight-line relationship, on a semilogarithmic scale, of coverages to failure versus thickness for test items 1-4 of the MWHGL

test section. Extrapolation of this relationship indicated that approximately 20,000 coverages of the 360-kip 12-wheel-assembly traffic would have been required to fail test item 5.

55. Coverage levels at failure versus total thickness for the structural layer test items are also shown in plate 19 for direct comparison with the behavior of the conventional construction. Each of the test items in the structural layer test section had a total thickness of 24 in. over a 4-CBR clay subgrade. (For a complete description of each item, see paragraph 6.) Item 1 of the structural layer test section failed at 198 coverages, as compared to 104 coverages for item 2 of the MWHGL test section; both of these items had a total thickness of 24 in. By reading the thickness required for a conventional pavement, according to plate 19, it is indicated that a total thickness of 26.2 in. would be required to sustain 198 coverages of 360-kip 12-wheel-assembly traffic. By this same type of comparison, a total thickness of 32.6 and 37.2 in. of conventional pavement would be required to withstand 1200 and 5000 coverages, respectively, which were the respective failure levels for items 2 and 3 of the structural layer test section. The data shown in plate 19 indicate that about 40 in. of conventional pavement would be required to sustain approximately 10,000 coverages of traffic, which was the coverage level when traffic was discontinued on unfailed item 4 of the structural layer test section.

56. Similar plots of coverages versus thickness for test items subjected to 50- and 75-kip single-wheel-assembly traffic are shown in plates 20 and 21, respectively. Test items 1 and 2 of both test sections were subjected to 50-kip single-wheel-assembly traffic, while traffic was applied with the 75-kip single-wheel assembly to items 3 and 4 of the structural layer test section and to items 4 and 5 of the MWHGL test section. The coverage-thickness relationship shows that, for the 50-kip single-wheel-assembly traffic, only about 20 in. of conventional pavement would be needed to sustain 40 coverages (failure of structural layer test section item 1) and approximately 22.5 in. of conventional pavement would support 120 coverages (failure of structural layer test section item 2). During the 75-kip single-wheel-assembly

traffic, failure occurred in structural layer test section item 3 after 50 coverages and in item 4 after 120 coverages. The coverages versus thickness relationship shown in plate 21 for the 75-kip single-wheel assembly indicates that 40 and 45.5 in. of conventional pavement would be required to withstand 50 and 120 coverages, respectively.

Discussion of Traffic Results

57. Test results indicate that the full-depth high-quality crushed stone base (item 3) performed better under the 360-kip 12-wheel assembly than did the pavement structures with lime- or cement-stabilized lean clay subbase layers (items 1 and 2). Item 3 was rated as failed after 5037 coverages, as compared to 198 and 1432 coverages for items 1 and 2, respectively. However, the full-thickness cement-stabilized clayey gravelly sand layer (item 4) performed much better than item 3. After 10,406 coverages, traffic was discontinued in item 4 when it was still evaluated as in satisfactory condition.

58. Items 1 and 2 were trafficked with a lighter gross load than items 3 and 4, with both the twin-tandem and single-wheel assemblies. Under these loading conditions, item 1 failed before item 2, and item 4 failed after item 3. Approximately two times the amount of traffic was required to reach failure in item 4 as was required for failure in item 3.

59. The results of the comparisons reported herein indicate that the performance of pavement structures that incorporated stabilized materials was better in most cases than that of conventional pavements of the same thickness where granular unbound base and subbase materials were used. Under the 360-kip 12-wheel-assembly traffic, a pavement consisting of 3 in. of asphaltic concrete, with either a 6-in.-thick crushed stone base and 15-in.-thick cement-stabilized lean clay subbase, a 21-in.-thick crushed stone base, or a 21-in.-thick cement-stabilized clayey gravelly sand base, will withstand 10 or more times the volume of traffic as will the same thickness of conventional pavement. This relationship is also true for the full-depth crushed stone base item and

the cement-stabilized clayey gravelly sand thickness trafficked with a 75-kip single-wheel assembly.

60. Data measured in the failure-investigation pits indicate that the CBR's of the lime- and cement-stabilized lean clay layers were greater at failure than when they were constructed and that the CBR of the cement-stabilized clayey gravelly sand was somewhat less after failure (202 as compared to 159 CBR). These failure data were taken after additional traffic had been applied to the respective items and, in most instances, after a time lapse of 20 to 30 days from the date of failure. During this time lapse, cementation in the stabilized layers could have developed. The data in plate 22 indicate that the stabilized materials increased in CBR with time. The data in this plate were obtained in tests of material outside the traffic lanes or in an undisturbed area. Therefore, it is believed that the strength data recorded in the stabilized layers are not necessarily indicative of the strengths of each respective layer at failure. Although the data for the strength of the stabilized soil are questionable, the observations and the deflection and deformation measurements made during the trafficking and at failure of these items indicate that:

a. The full-depth high-quality crushed stone performed better than the structures that included lime- or cement-stabilized lean clay subbases.

b. The full-depth cement-stabilized clayey gravelly sand performed better than the full-depth high-quality crushed stone layer.

Instrumentation Data Reduction and Analysis

61. An initial reduction of data has been performed; however, since test section and instrumentation work under the Military Engineer Design and Expedient Construction Criteria Program is still being conducted, the presentation of the data and a formal analysis will be given in a future report. Listed in Appendix A are some preliminary maximum values measured in the test section.

PART V: CONCLUSIONS

62. Based on the results of tests reported herein, the following conclusions are believed warranted.

a. The concept of utilizing stabilized structural layers in flexible pavement is highly recommended.

b. The performance of the lime-stabilized subbase material was as good as that of similar pavements constructed of unbound granular base and subbase materials, as used in the MWHGL test section at WES, when trafficked with a 360-kip 12-wheel assembly.

c. The performance of the cement-stabilized base and subbase materials and the full-depth high-quality crushed stone base course material was better than that of similar pavements constructed of unbound granular base and subbase materials, as used in the MWHGL test section, when trafficked with a 360-kip 12-wheel assembly.

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Table

Summary of Thickness, CBR, Wa

Test Item	Assembly	Station	Location	Material	Elevation ft	Layer Thickness ft	Layer Thickness in.	Total Thickness in. Design	Total Thickness in. Act
								As-Constr	
1	-	3+45	℄ item	Asphaltic concrete	193.88	0.19	2.28	24.00	22
				Crushed stone base	193.69	0.40	4.80		
				Lime-stabilized subbase (CL)	193.29	1.28	15.36		
				Subgrade (CH)	192.01				
				(CH)	191.51				
				(CH)	191.01				
				(CH)	190.01				
2	-	3+15	℄ item	Asphaltic concrete	193.81	0.21	2.52	24.00	22
				Crushed stone base	193.60	0.41	4.72		
				Cement-stabilized subbase (CL)	193.19	1.31	15.72		
				Subgrade (CH)	191.88				
				(CH)	191.38				
				(CH)	190.88				
				(CH)	189.88				
3	-	2+85	℄ item	Asphaltic concrete	193.68	0.23	2.76	24.00	23
				Crushed stone base	193.39	1.74	20.88		
				Crushed stone base	192.89				
				Subgrade (CH)	191.65				
				(CH)	191.15				
				(CH)	190.65				
				(CH)	189.65				
4	-	2+55	℄ item	Asphaltic concrete	193.52	0.23	2.76	24.00	24
				Cement-stabilized base	193.29	1.82	21.84		
				Cement-stabilized base	192.79				
				Subgrade (CH)	191.47				
				(CH)	190.97				
				(CH)	190.47				
				(CH)	189.47				
								After-T	
1	360 kip 12 wheel	3+50	℄ traffic lane	Asphaltic concrete	193.74	0.23	2.76	24.00	23
				Crushed stone base	193.51	0.41	4.92		
				Lime-stabilized subbase (CL)	193.10	1.36	16.34		
				Subgrade (CH)	191.74				
				(CH)	191.24				
				(CH)	190.74				
1	360 kip 12 wheel	3+50	outside traffic lane	Asphaltic concrete	193.66	0.25	3.00	24.00	2
				Crushed stone base	193.41	0.39	4.68		
				Lime-stabilized subbase (CL)	193.02	1.24	14.89		
				Subgrade (CH)	191.78				
				(CH)	191.28				
				(CH)	190.78				

* CE 55 compaction effort.
** CE 12 compaction effort.
† All base and subbase as-constructed data were taken one day after compaction of the respective layers.

Table 1

, Water-Content, and Density Data

l ess Actual	Traffic Coverages	Water Content Prior to Compaction %	CBR	Water Content %	Dry Density pcf In-Place A	Dry Density pcf Laboratory B	Percent Laboratory Density A/B	Date Tested†
nstructed Data								
22.44	0		-	-	-	-	-	-
			200	4.9	140.1	140*	100	November 6, 1970
		18.7	38	17.6	104.5	103.4**	101	November 4, 1970
			4.2	31.3	87.5	91.5*	96	-
			3.3	32.2	86.4	88.0*	98	-
			6.0	32.7	86.3	86.5*	100	-
			4.6	34.0	83.8	86.0*	97	-
22.96	0		-	-	-	-	-	-
			180	6.0	132.5	140*	95	November 6, 1970
		18.3	80	15.7	106.9	105.5**	101	November 4, 1970
			2.7	32.7	85.3	86.5*	99	-
			2.7	33.1	84.5	86.5*	98	-
			6.1	31.8	87.1	88.0*	99	-
			5.7	31.3	87.3	90.0*	97	-
23.64	0		-	-	-	-	-	-
			132	4.4	141.4	140*	101	November 6, 1970
			35	1.7	140.6	140*	100	November 4, 1970
			3.9	31.3	87.5	90.5*	97	-
			3.0	32.7	85.8	86.5*	99	-
			6.3	31.9	86.8	88.0*	99	-
			4.8	33.1	84.3	86.0*	98	-
24.[illegible]	0		-	-	-	-	-	-
		6.2	202	2.7	139.8	-	-	November 6, 1970
		7.0	285	4.5	133.7	-	-	November 4, 1970
			3.7	32.5	85.3	88.5*	96	-
			3.3	32.2	85.8	88.0*	98	-
			5.5	32.3	87.3	86.5*	101	-
			6.3	31.8	86.2	88.5*	97	-
er-Traffic Data								
23.02	198		-	-	-	-	-	-
			44	5.7	146.8	140*	105	December 23, 1970
			84	19.4	103.1	103.3**	100	December 23, 1970
			4.2	29.2	88.7	92.6*	96	-
			6	26.9	91.1	96.8*	94	-
			6	29.2	88.6	92.6*	96	-
22.57	0		-	-	-	-	-	-
			85	2.3	149.0	140*	107	December 23, 1970
			72	19.1	103.0	103.3**	100	December 23, 1970
			3.9	2[illegible].5	88.6	92.0*	96	-
			5.0	28.7	89.2	91.5*	98	-
			5.5	-	-	-	-	-

Test Item	Assembly	Station	Location	Material	Elevation ft	Layer Thickness ft	Layer Thickness in.	Tot Thick i Design
2	360 kip 12 wheel	3+20	℄ traffic lane	Asphaltic concrete	193.63	0.32	3.84	24.00
				Crushed stone base	193.31	0.33	3.96	
				Cement-stabilized subbase (CL)	192.98	1.33	16.00	
				Subgrade (CH)	191.65			
				(CH)	191.15			
				(CH)	190.65			
			outside traffic lane	Asphaltic concrete	193.68	0.37	4.44	24.00
				Crushed stone base	193.31	0.33	3.96	
				Cement-stabilized subbase (CL)	192.98	1.38	16.55	
				Subgrade (CH)	191.60			
				(CH)	191.10			
				(CH)	190.60			
3	360 kip 12 wheel	2+80	℄ traffic lane	Asphaltic concrete	193.34	0.22	2.64	24.00
				Crushed stone base	193.12	1.68	20.16	
				Subgrade (CH)	191.14			
				(CH)	190.94			
				(CH)	190.44			
			outside traffic lane	Asphaltic concrete	193.39	0.29	3.48	24.00
				Crushed stone base	193.10	1.64	19.68	
				Subgrade (CH)	191.46			
				(CH)	190.96			
				(CH)	190.46			
4	160 kip Twin tandem	3+50	℄ traffic lane	Asphaltic concrete	193.91	0.29	3.48	24.00
				Crushed stone base	193.70	0.44	5.28	
				Lime-stabilized subbase (CL)	193.26	1.24	14.88	
				Subgrade (CH)	192.02			
				(CH)	191.52			
				(CH)	191.02			
			outside traffic lane	Asphaltic concrete	194.00	0.24	2.88	24.00
				Crushed stone base	193.76	0.42	5.04	
				Lime-stabilized subbase (CL)	193.34	1.27	15.24	
				Subgrade (CH)	192.07			
				(CH)	191.57			
				(CH)	191.07			
2	160 kip Twin tandem	3+20	℄ traffic lane	Asphaltic concrete	193.92	0.26	3.12	24.00
				Crushed stone base	193.46	0.38	4.56	
				Cement-stabilized subbase (CL)	193.08	1.38	16.56	
				Subgrade (CH)	191.70			
				(CH)	191.20			
				(CH)	190.70			
			outside traffic lane	Asphaltic concrete	193.84	0.22	2.64	24.00
				Crushed stone base	193.62	0.47	5.64	
				Cement-stabilized subbase (CL)	193.16	1.25	15.00	
				Subgrade (CH)	191.90			
				(CH)	191.40			
				(CH)	190.90			

Table 1 (Continued)

al ess . Actual	Traffic Coverages	Water Content Prior to Compaction %	CBR	Water Content %	Dry Density pcf In-Place A	Dry Density pcf Laboratory B	Percent Laboratory Density A/B	Date Tested
23.80	1871		-	-	-	-	-	-
			72	1.9	149.5	140*	107	January 6, 1971
			122	20.5	107.4	-	-	January 6, 1971
			3.9	34.5	85.1	84.0*	101	-
			5.5	32.2	87.2	87.5*	100	-
			7	32.6	87.4	86.9*	101	-
24.95	0		-	-	-	-	-	-
			59	3.2	157.4	140*	112	January 6, 1971
			121	20.9	106.6	-	-	January 6, 1971
			2.3	34.2	84.9	84.1*	101	-
			5.9	30.1	91.0	90.8*	100	-
			7	33.6	86.7	85.0*	102	-
22.80	5037		-	-	-	-	-	-
			86	1.9	144.4	140*	103	January 12, 1971
			1.2	32.5	86.7	86.9*	100	-
			8	29.9	87.7	91.4*	96	-
			5.3	32.1	85.6	87.5*	98	-
23.16	0		-	-	-	-	-	-
			87	1.6	147.4	140*	104	January 12, 1971
			1.0	33.4	84.9	85.7*	99	-
			8	29.5	91.2	92.1*	99	-
			7	31.0	88.8	89.9*	99	-
22.68	140		-	-	-	-	-	-
			30	2.8	147.4	140*	105	March 30, 1971
			68	20.4	98.5	103.3**	95	March 30, 1971
			4.1	33.0	86.3	86.1*	100	-
			6	31.9	87.0	88.0*	99	-
			6	32.9	85.7	86.4*	100	-
23.16	0		-	-	-	-	-	-
			58	2.5	149.7	140*	107	March 30, 1971
			118	19.4	102.4	103.3**	99	March 30, 1971
			3.5	32.7	86.6	86.8*	100	-
			5	30.9	89.3	90*	99	-
			7	31.8	87.1	88.1*	99	-
24.24	1000		-	-	-	-	-	-
			76	1.8	145.2	140*	104	April 6, 1971
			200	19.9	107.0	-	-	April 6, 1971
			3.2	35.0	84.6	83.5*	101	-
			4.9	32.8	86.8	86.8*	100	-
			5	33.2	86.5	85.9	101	-
21.60	0		-	-	-	-	-	-
			58	3.1	143.2	140*	102	April 6, 1971
			253	19.0	107.4	-	-	April 6, 1971
			2.7	33.9	84.4	84.9*	99	-
			4.7	32.5	86.1	86.9*	99	-
			6	33.3	85.1	85.8*	99	-

(sheet 2 of 3)

Test Item	Assembly	Station	Location	Material	Elevation ft	Layer Thickness ft	Layer Thickness in.	Total Thickness in. Design	Total Thickness in. Ac
3	200 kip Twin tandem	2+90	℄ traffic lane	Asphaltic concrete	193.40	0.23	2.76	24.00	2
				Crushed stone base	193.17	1.57	18.84		
				Subgrade (CH)	191.60				
				(CH)	191.10				
				(CH)	190.60				
			outside traffic lane	Asphaltic concrete	193.77	0.19	2.28	24.00	2
				Crushed stone base	193.58	1.88	22.56		
				Subgrade (CH)	191.70				
				(CH)	191.20				
				(CH)	190.70				
1	50 kip Single wheel	3+50	℄ traffic lane	Asphaltic concrete	193.94	0.19	2.28	24.00	2
				Crushed stone base	193.75	0.47	5.64		
				Lime-stabilized subbase (CL)	193.28	1.12	13.44		
				Subgrade (CH)	192.16				
				(CH)	191.66				
				(CH)	190.16				
2	50 kip Single wheel	3+20	℄ traffic lane	Asphaltic concrete	193.78	0.21	2.52	24.00	2
				Crushed stone base	193.57	0.52	6.25		
				Cement-stabilized subbase (CL)	193.05	1.21	14.52		
				Subgrade (CH)	191.84				
				(CH)	191.34				
				(CH)	190.84				
3	75 kip Single wheel	2+90	℄ traffic lane	Asphaltic concrete	193.61	0.23	2.76	24.00	2
				Crushed stone base	193.38	1.66	19.92		
				Subgrade (CH)	191.72				
				(CH)	191.22				
				(CH)	190.72				
4	75 kip Single wheel	2+30	℄ traffic lane	Asphaltic concrete	-	-	-	24.00	
				Cement-stabilized base	-	-	-		
			outside traffic lane	Asphaltic concrete	-	-	-	24.00	
				Cement-stabilized base	-	-	-		

**This is in addition to 10,400 coverages of the 360-kip, 12-wheel assembly.

Table 1 (Concluded)

Total Thickness in. Design	Total Thickness in. Actual	Traffic Coverages	Water Content Prior to Compaction %	CBR	Water Content %	Dry Density pcf In-Place A	Dry Density pcf Laboratory B	Percent Laboratory Density A/B	Date Tested
24.00	21.60	890		-	-	-	-	-	-
				63	1.3	143.2	140*	106	April 8, 1971
				2.1	30.8	88.3	90.0*	98	-
				4.0	31.2	88.2	89.4*	99	-
				4.0	3[illegible].7	87.8	88.1*	100	-
24.00	24.84	0		-	-	-	-	-	-
				67	0.6	138.4	140*	99	April 8, 1971
				2.3	32.2	86.0	87.5*	98	-
				3.6	30.3	88.7	90.6*	98	-
				6	32.2	87.3	87.5*	100	-
24.00	21.36	40		-	-	-	-	-	-
				53	3.9	151.8	140*	108	March 30, 1971
				53	18.8	103.3	103.3**	100	March 30, 1971
				2.9	33.0	86.1	86.1*	100	-
				4.7	31.7	87.5	88.1*	99	-
				6	32.7	87.3	86.8*	101	-
24.00	23.29	120		-	-	-	-	-	-
				42	25	146.9	140*	105	-
				109	19.9	106.2	-	-	April 6, 1971
				1.7	35.5	83.2	82.0*	101	April 6, 1971
				2.9	32.3	87.1	87.5*	100	-
				3.9	34.8	83.9	83.2*	101	-
24.00	22.63	50		-	-	-	-	-	-
				94	1.3	146.1	140*	104	April 8, 1971
				3.0	30.9	88.7	87.5*	101	-
				4.3	29.9	89.9	91.4*	98	April 16, 1971
				4.6	31.9	87.6	88.0*	100	-
24.00	-	200++		-	-	-	-	-	-
				159	6.2	141.1	-	-	April 16, 1971
24.00	-	0		-	-	-	-	-	-
				261	6.7	137.7	-	-	-

Table 2

Summary of Stability, Flow, Voids, and Density Data: Asphaltic Concrete

Test Item	Coverages	Asphalt Content % Total Weight	Stability lb	Flow 1/100 in.	Percent Voids: Total Mix	Percent Voids: Filled with Asphalt Cement	Unit Weight Total Mix pcf	Percent Plant Laboratory Density
Plant-Mixed, Laboratory-Compacted Samples								
		4.0	3673	9	4.3	69.1	154.4	100.0
Field Cores - Prior to Traffic								
1	0	4.0	1274	18	6.3	59.9	151.3	98.0
2	0	4.0	1108	15	7.5	55.6	149.3	96.7
3	0	4.0	1253	18	6.0	61.4	151.8	98.3
4	0	4.0	1099	18	7.0	57.5	150.1	97.2
Field Cores - After Traffic								
1	198	4.0	780	28	8.6	51.6	147.4	95.5
	0	4.0	966	19	6.9	57.6	150.2	97.3
2	1,342	4.0	912	24	6.1	60.9	151.6	98.2
	0	4.0	854	22	6.4	59.6	151.1	97.9
3	5,037	4.0	1108	26	5.1	65.2	153.2	99.2
	0	4.0	1182	22	5.8	62.1	151.9	98.4
4	10,406	4.0	1184	22	6.1	60.9	151.5	98.1
	0	4.0	1267	21	6.0	61.2	151.7	98.3

[illegible]


Photo 1. Excavation for test section construction

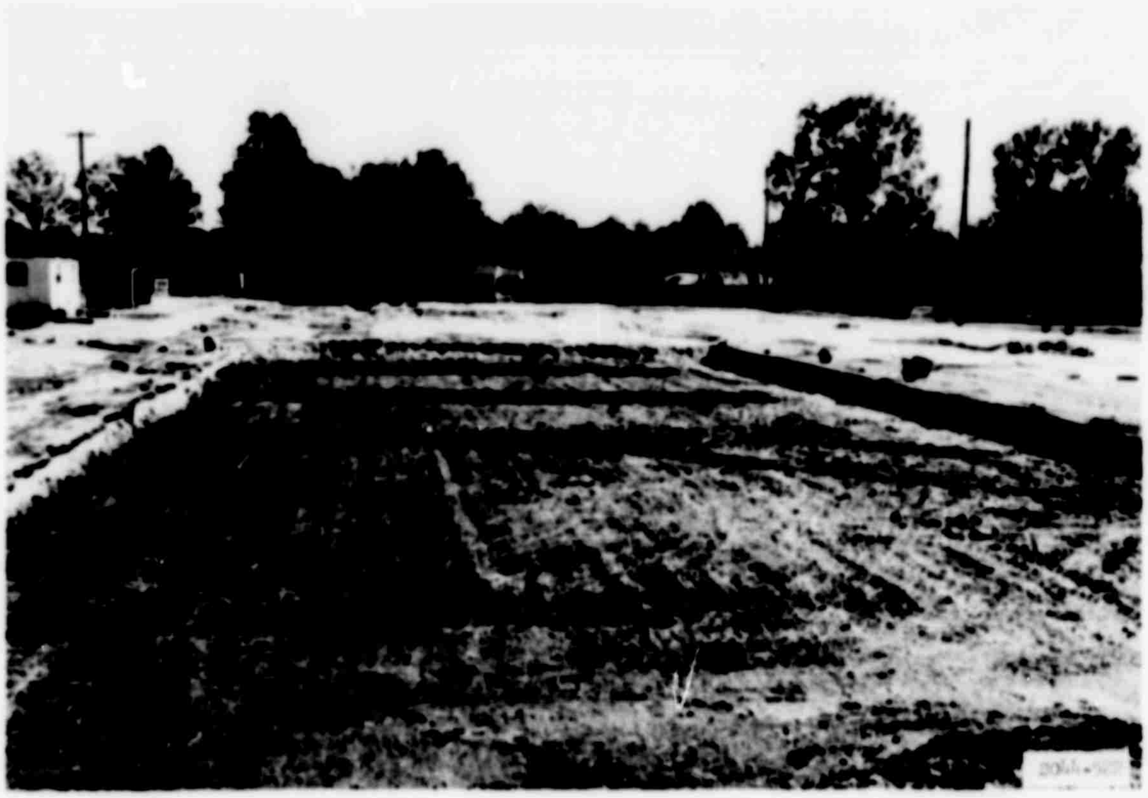


Photo 2. View of finished subgrade

Photo 3. Bags of stabilizing agent placed at predetermined intervals for desired percent of treatment

Photo 4. Mixing soil and stabilizing agent in items 1 and 2

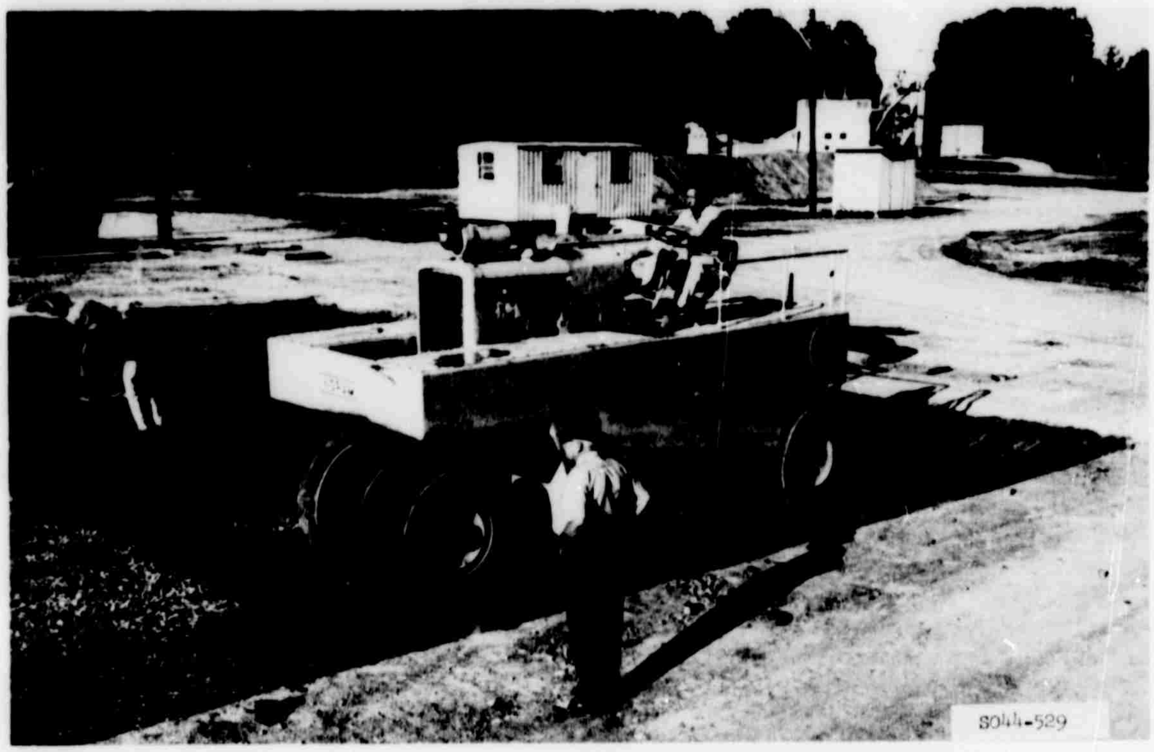


Photo 5. Compaction of cement-stabilized base in item 4 with 30-ton roller

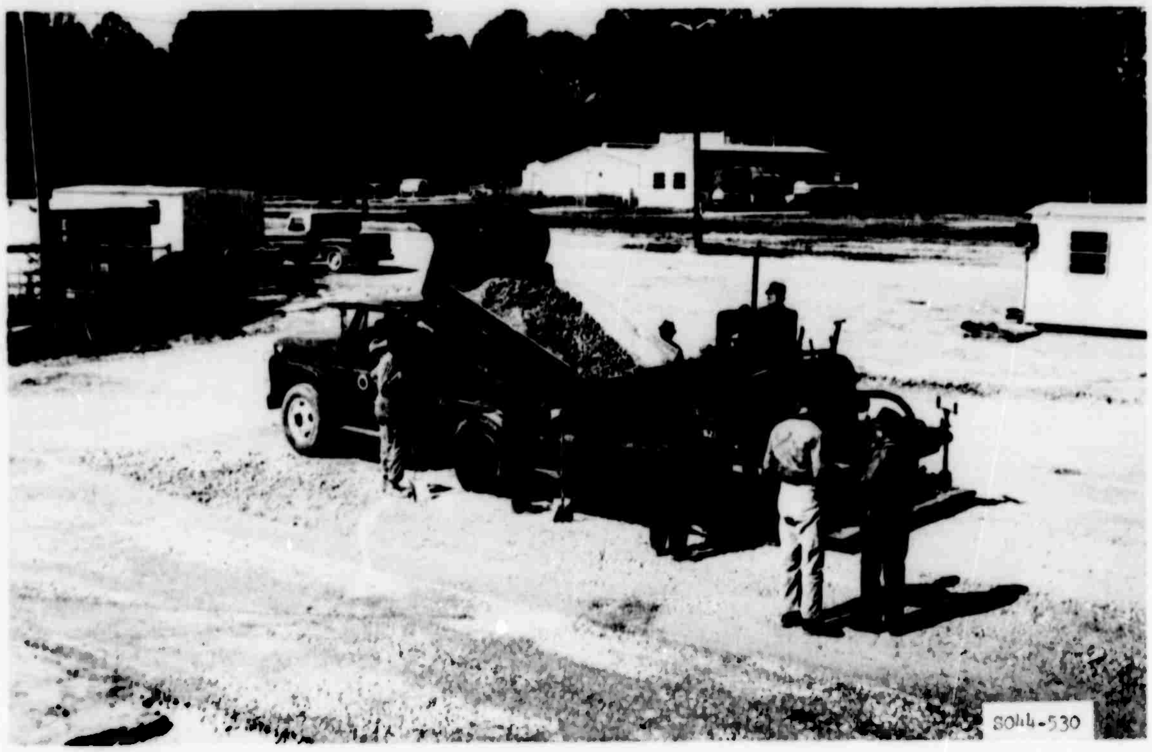


Photo 6. Placing final lift of crushed stone base course

Photo 7. Compacting final lift of base course

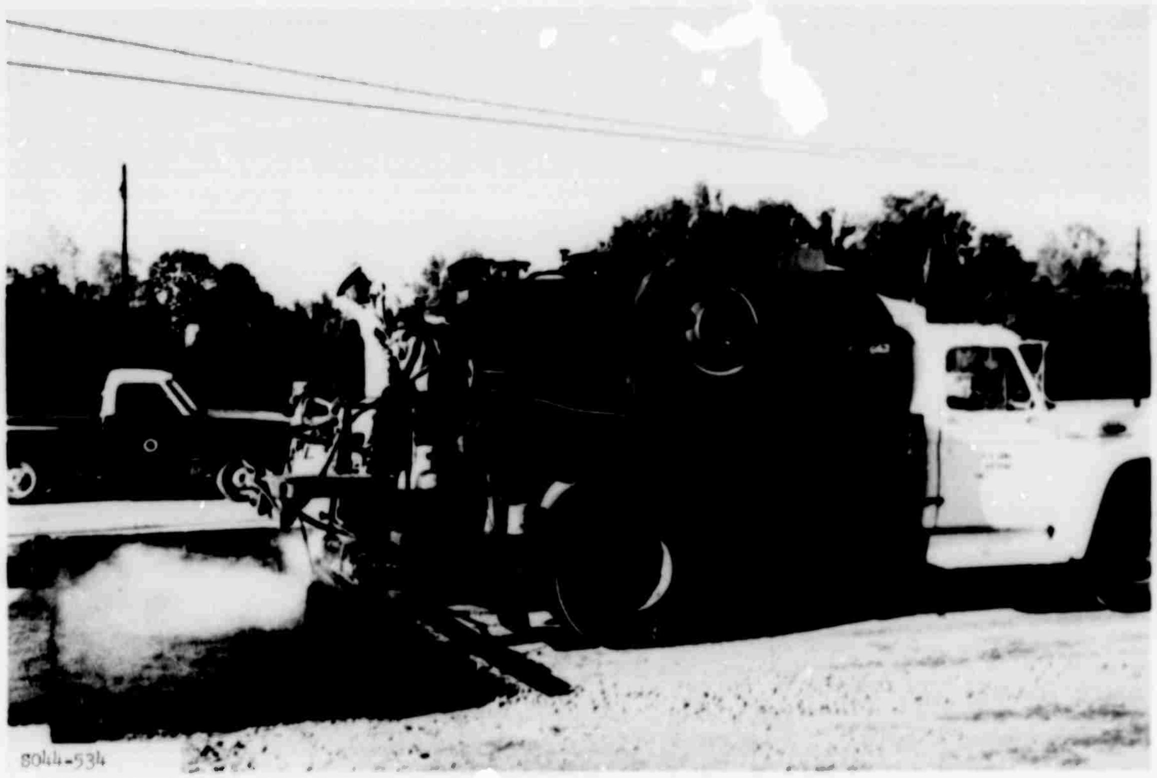


Photo 8. Priming crushed stone base course with MC-1 cutback asphalt

Photo 9. Placement of asphaltic concrete

Photo 10. Breakdown rolling of asphaltic concrete with a steel-wheel roller and final compaction with a 30-ton rubber-tired roller

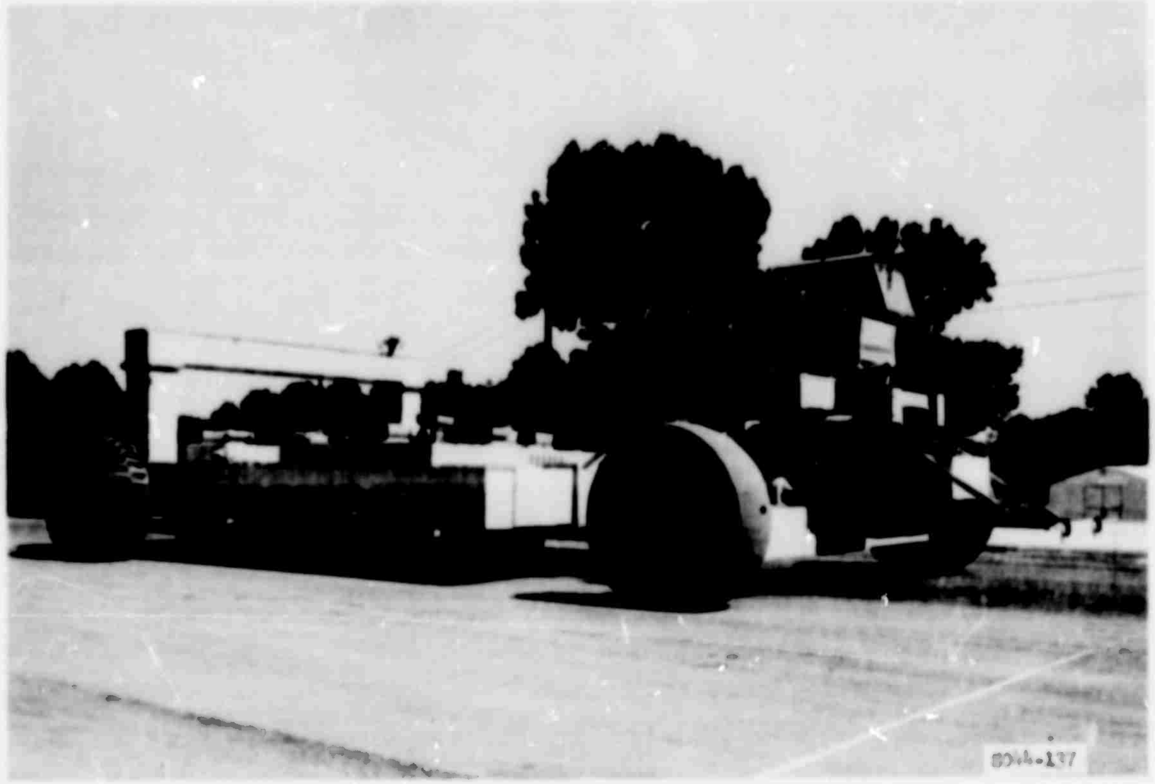


Photo 11. 360-kip, 12-wheel-assembly test cart

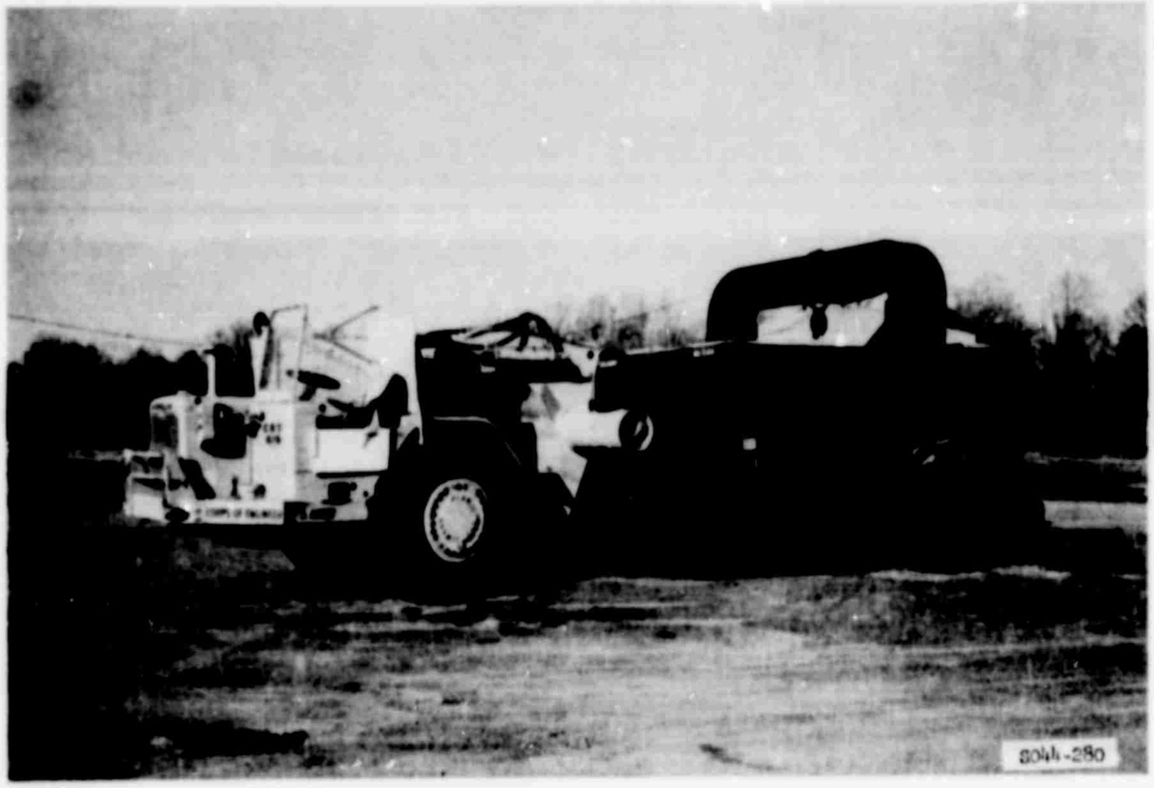


Photo 12. Twin-tandem-assembly test cart

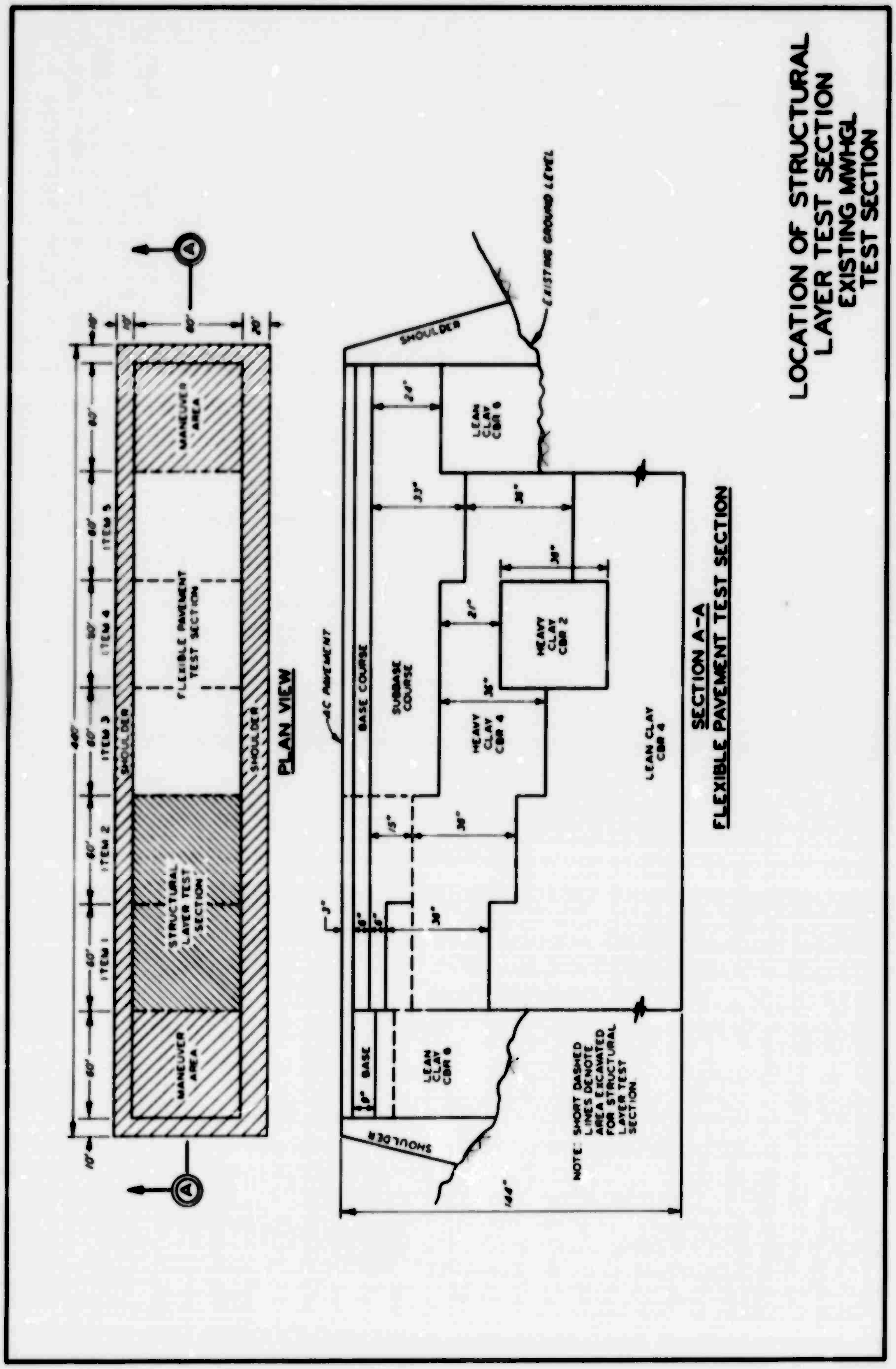
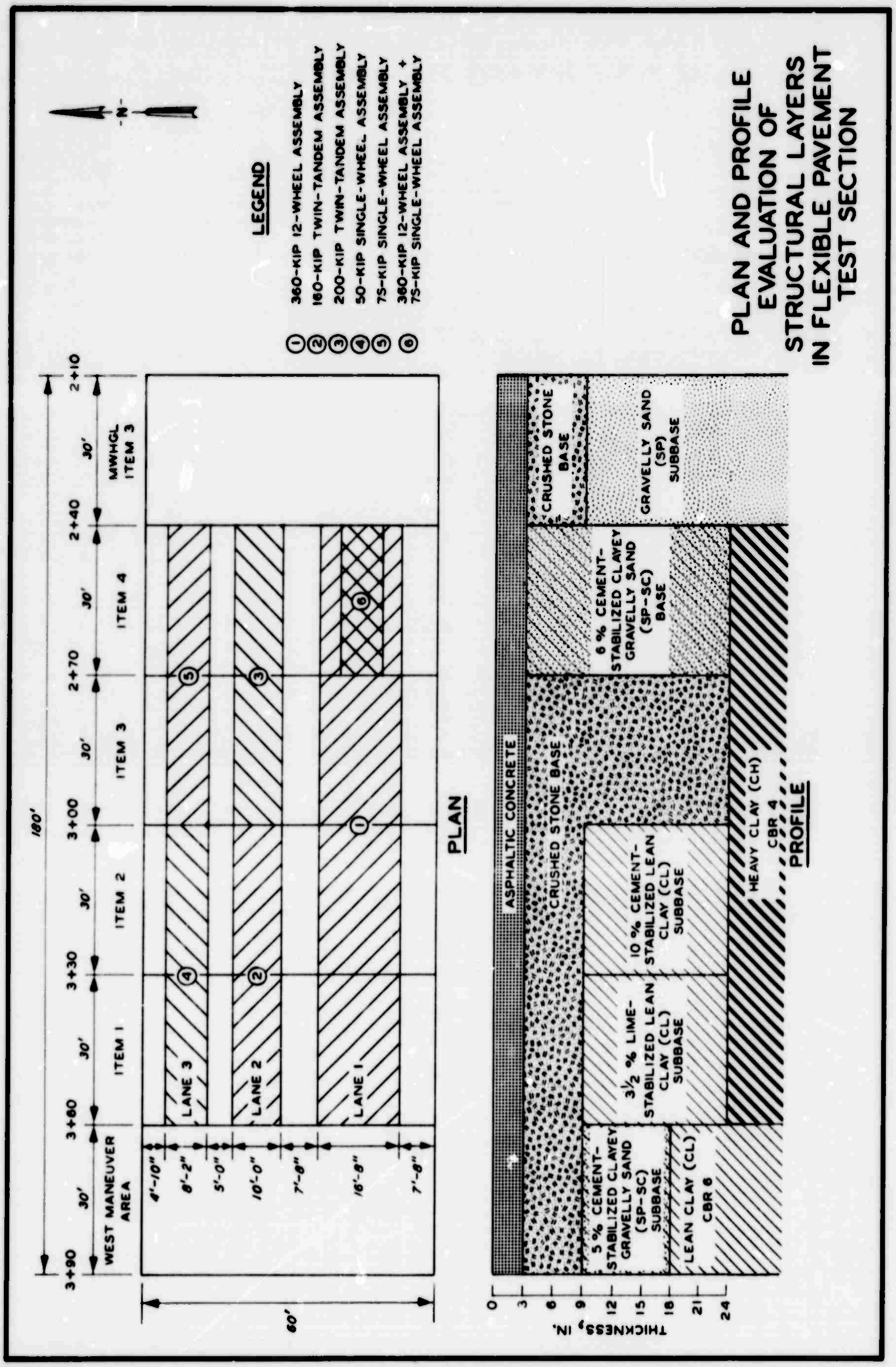
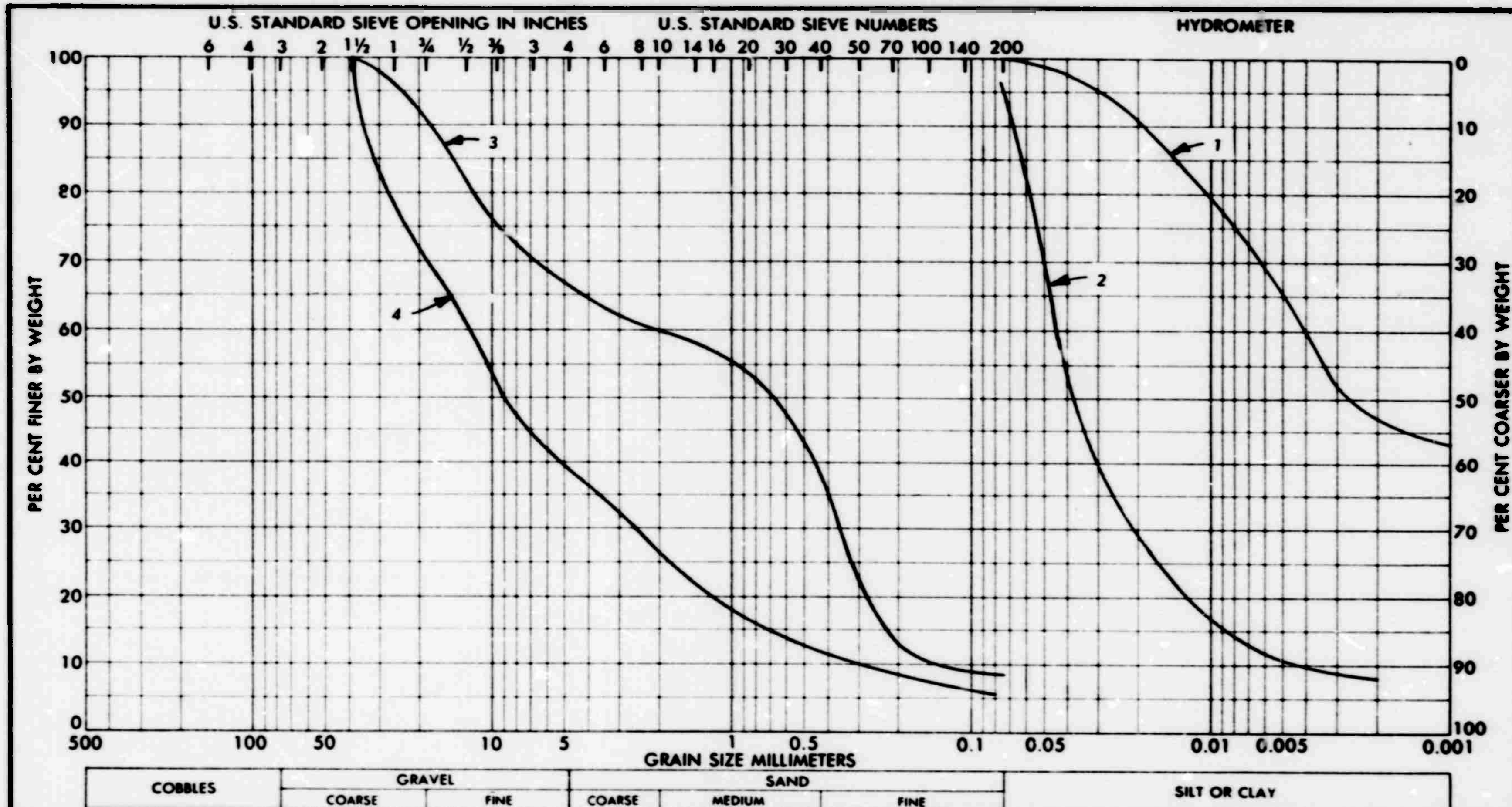

LOCATION OF STRUCTURAL LAYER TEST SECTION EXISTING MWHGL TEST SECTION
PLAN VIEW
MANEUVER AREA
STRUCTURAL LAYER TEST SECTION
FLEXIBLE PAVEMENT TEST SECTION
MANEUVER AREA
SHOULDER
SHOULDER
ITEM 1
ITEM 2
ITEM 3
ITEM 4
ITEM 5
SECTION A-A
FLEXIBLE PAVEMENT TEST SECTION
AC PAVEMENT
BASE COURSE
SUBBASE COURSE
BASE
LEAN CLAY CBR 6
LEAN CLAY CBR 6
HEAVY CLAY CBR 4
HEAVY CLAY CBR 2
LEAN CLAY CBR 4
SHOULDER
SHOULDER
EXISTING GROUND LEVEL
NOTE: SHORT DASHED LINES DENOTE AREA EXCAVATED FOR STRUCTURAL LAYER TEST SECTION.

PLATE 1



PLAN AND PROFILE
EVALUATION OF
STRUCTURAL LAYERS
IN FLEXIBLE PAVEMENT
TEST SECTION
LEGEND
1 360-KIP 12-WHEEL ASSEMBLY
2 160-KIP TWIN-TANDEM ASSEMBLY
3 200-KIP TWIN-TANDEM ASSEMBLY
4 50-KIP SINGLE-WHEEL ASSEMBLY
5 75-KIP SINGLE-WHEEL ASSEMBLY
6 360-KIP 12-WHEEL ASSEMBLY + 75-KIP SINGLE-WHEEL ASSEMBLY
-N-
180'
3+90
3+60
3+30
3+00
2+70
2+40
2+10
30'
WEST MANEUVER AREA
ITEM 1
ITEM 2
ITEM 3
ITEM 4
MWHGL ITEM 3
LANE 3
LANE 2
LANE 1
4'-10"
8'-2"
5'-0"
10'-0"
7'-8"
16'-8"
7'-8"
60'
PLAN
ASPHALTIC CONCRETE
CRUSHED STONE BASE
CRUSHED STONE BASE
GRAVELLY SAND (SP) SUBBASE
6 % CEMENT-STABILIZED CLAYEY GRAVELLY SAND (SP-SC) BASE
10 % CEMENT-STABILIZED LEAN CLAY (CL) SUBBASE
3 1/2 % LIME-STABILIZED LEAN CLAY (CL) SUBBASE
5 % CEMENT-STABILIZED CLAYEY GRAVELLY SAND (SP-SC) SUBBASE
LEAN CLAY (CL) CBR 6
HEAVY CLAY (CH) CBR 4
PROFILE
THICKNESS, IN.
0
3
6
9
12
15
18
21
24

PLATE 2

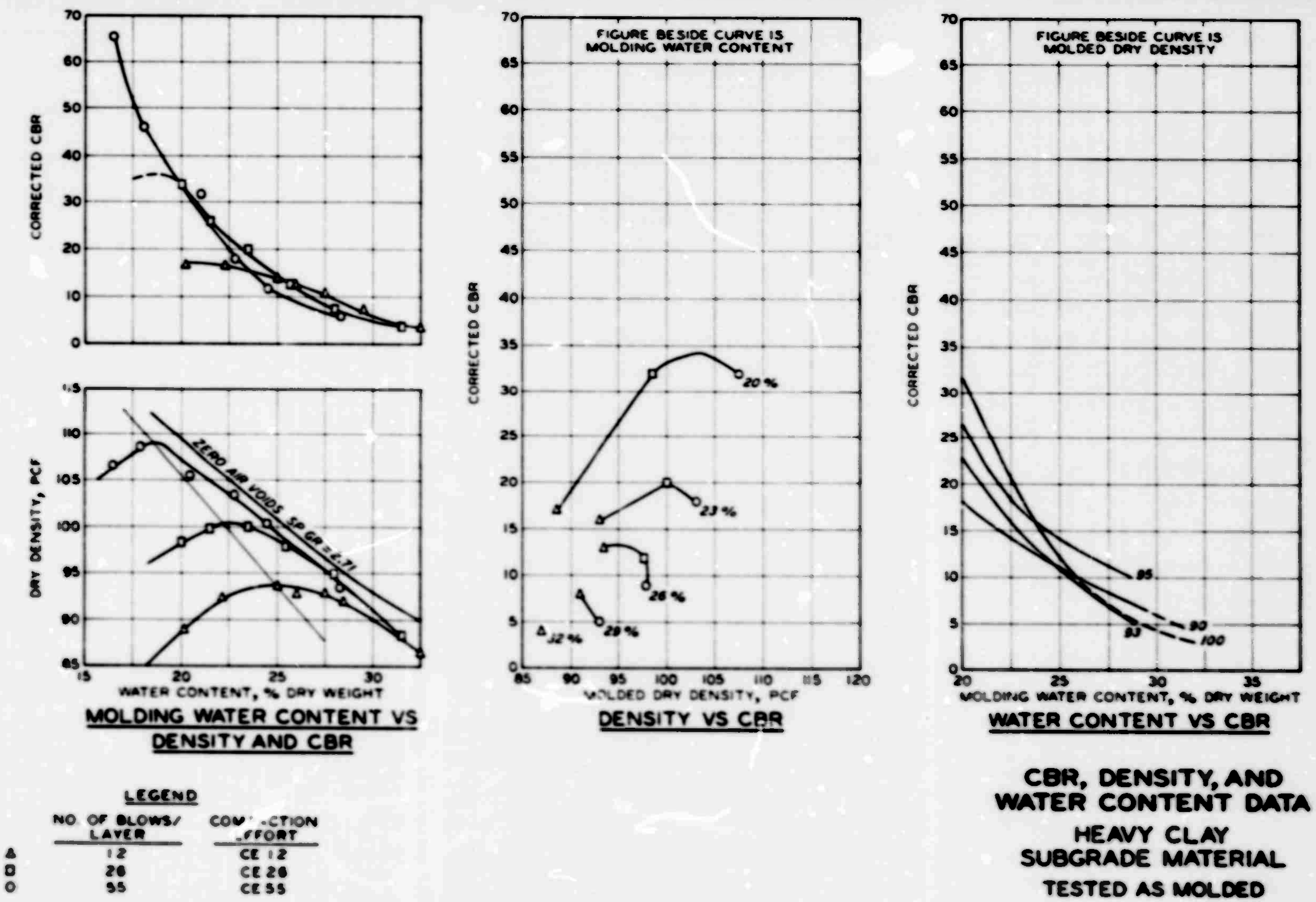


SAMPLE NO	MATERIAL	CLASSIFICATION	NAT W%	LL	PL	PI
1	SUBGRADE	HEAVY CLAY (CH)		73	25	48
2	SUBBASE	LEAN CLAY (CL)		34	22	12
3	BASE	CLAYEY GRAVELLY SAND (SP-SC)		23	12	11
4	BASE	CRUSHED STONE (GW)		NONPLASTIC		

CLASSIFICATION DATA

PLATE 3

PLATE 4

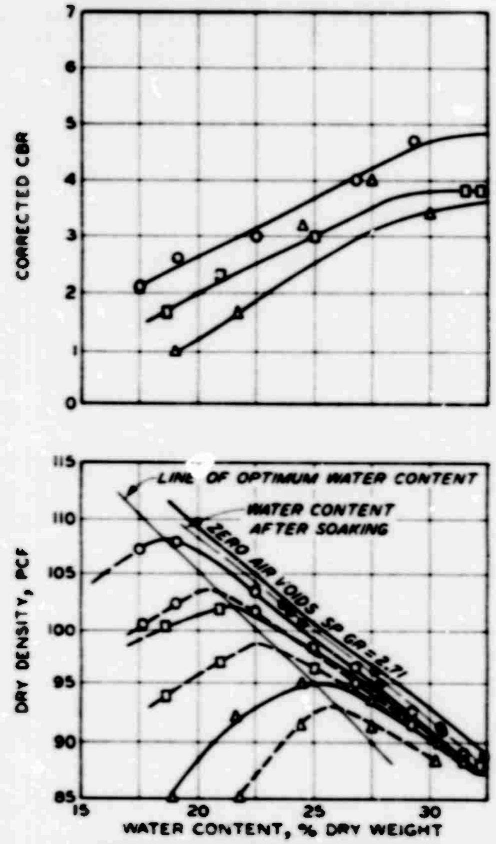


CBR, DENSITY, AND WATER CONTENT DATA

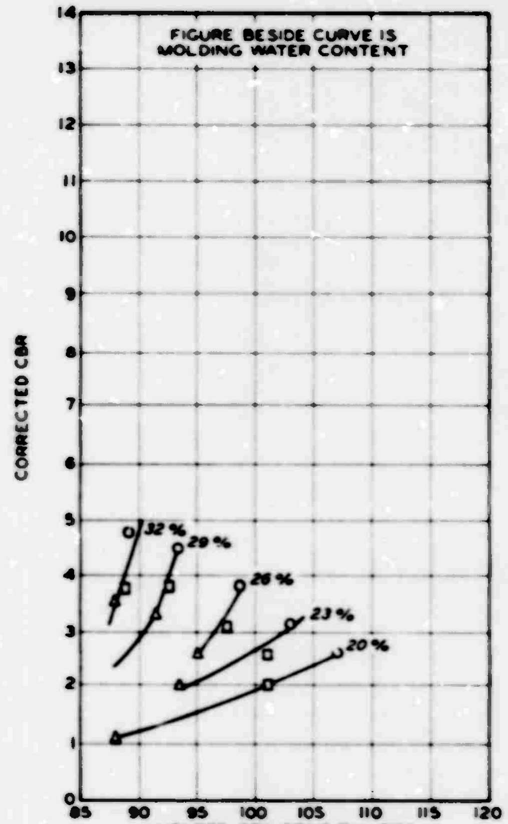
HEAVY CLAY

SUBGRADE MATERIAL

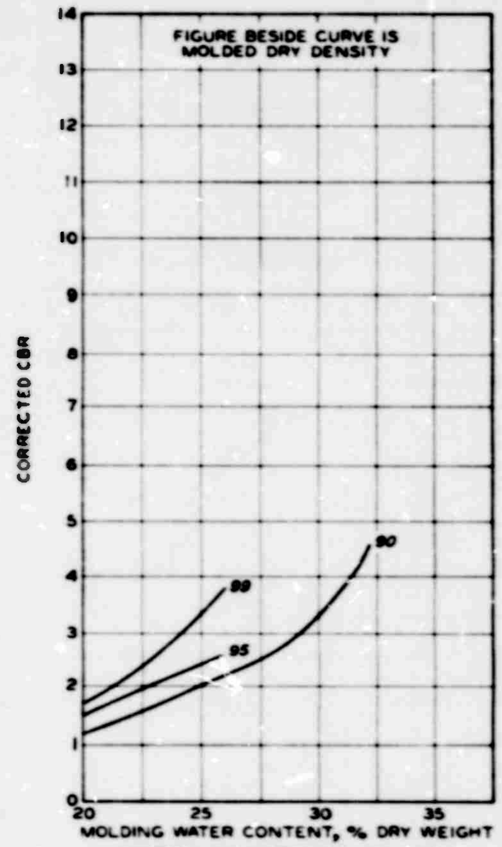
TESTED AS MOLDED



MOLDING WATER CONTENT VS DENSITY AND CBR



DENSITY VS CBR



WATER CONTENT VS CBR

LEGEND

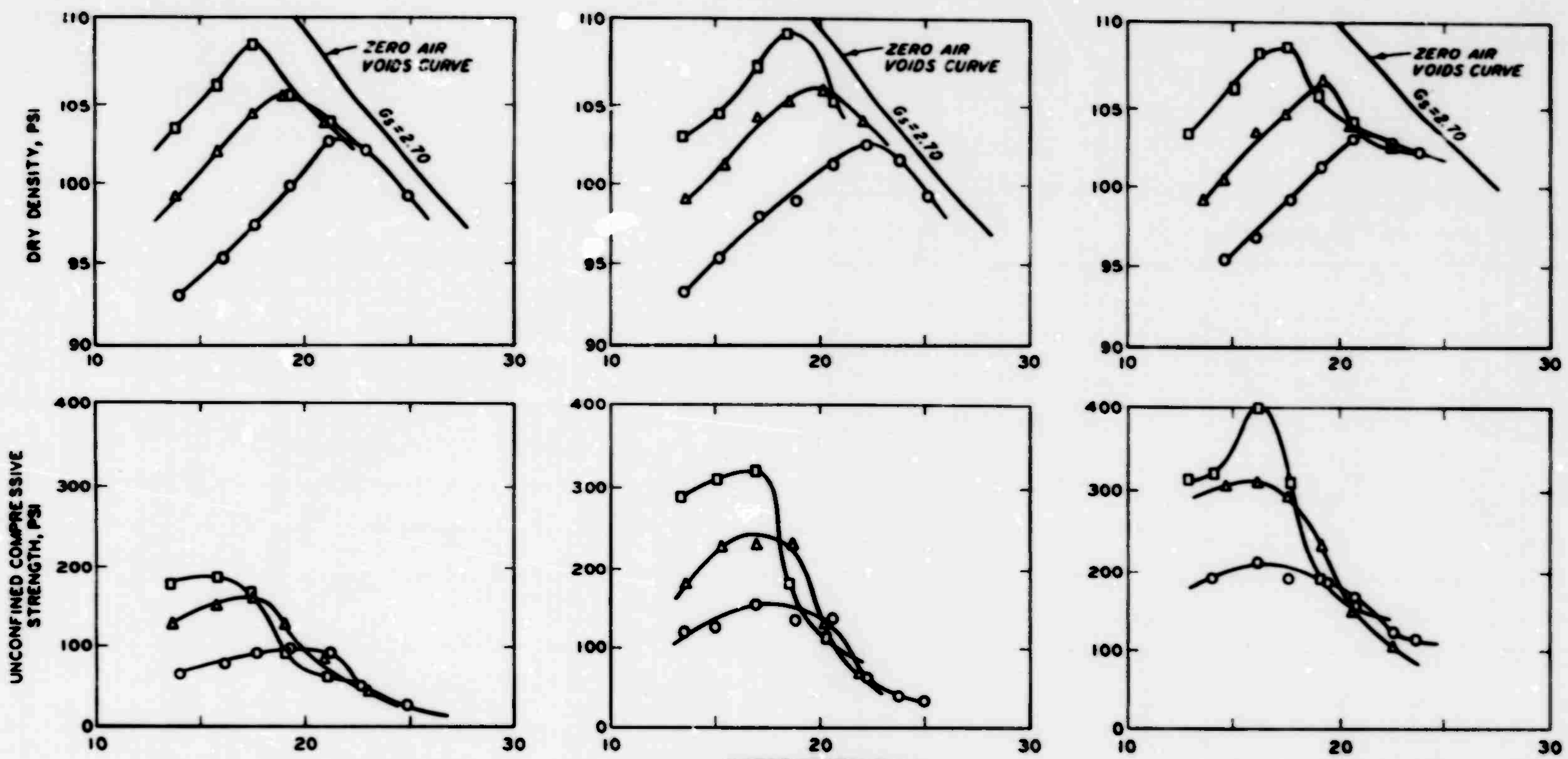
	NO. OF BLOWS/ LAYER	COMPACTION EFFORT
△	12	CE 12
□	26	CE 26
○	55	CE 55

— AS-MOLDED DENSITY
--- DENSITY AFTER SOAKING
● WATER CONTENT AFTER SOAKING

CBR, DENSITY, AND WATER CONTENT DATA

HEAVY CLAY SUBGRADE MATERIAL

TESTED AFTER SOAKING



a. 3% CEMENT

b. 6% CEMENT

c. 10% CEMENT

LEGEND

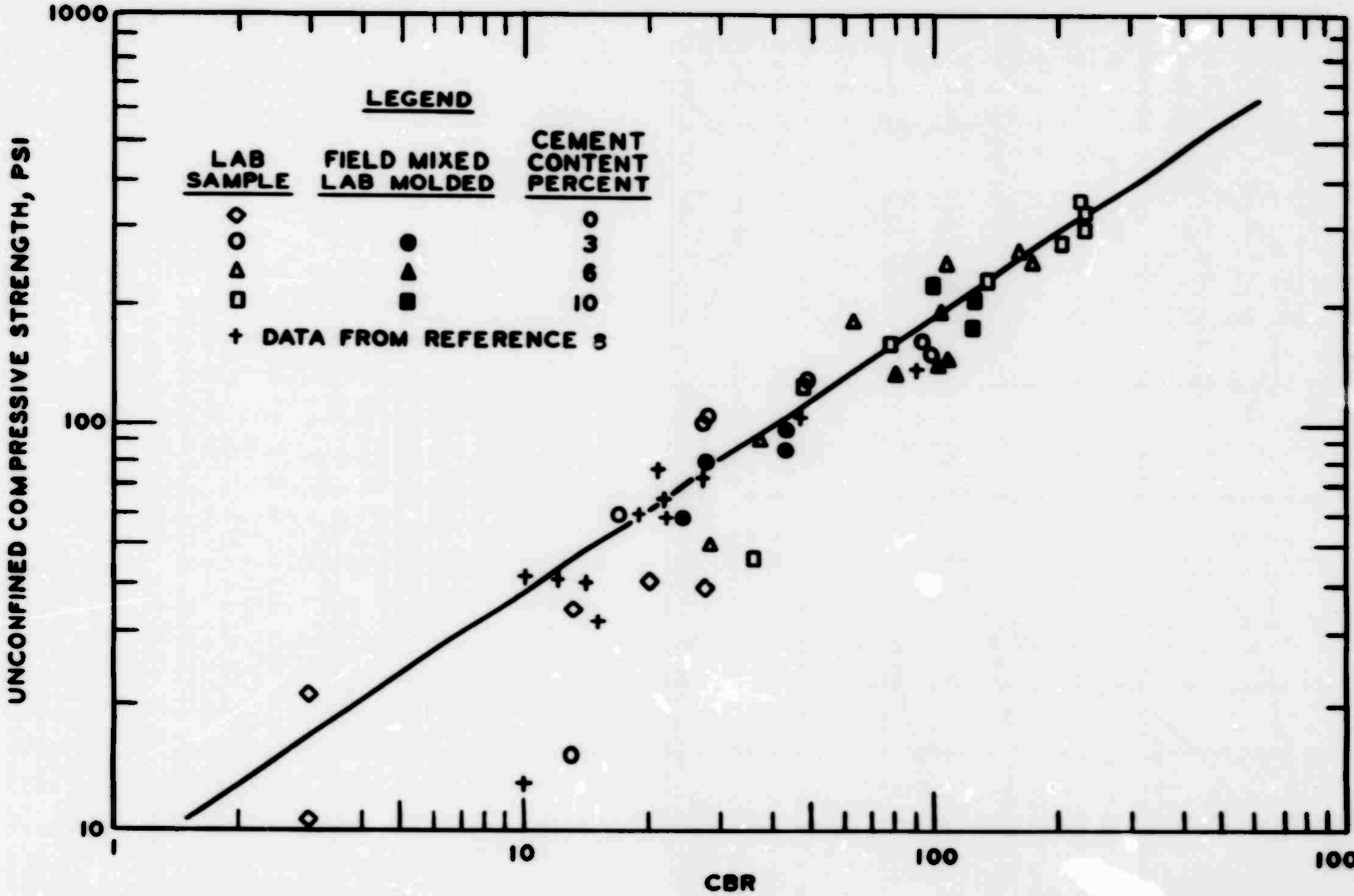
	TAMPS PER LAYER*	SPRING TAMPER, LB
○	10	20
△	10	40
□	50	40

* EACH SAMPLE COMPACTED IN FIVE LAYERS, CURED 7 DAYS PRIOR TO TESTING.

NOTE: DATA FROM REFERENCE 7.

WATER CONTENT, DRY DENSITY, AND STRENGTH

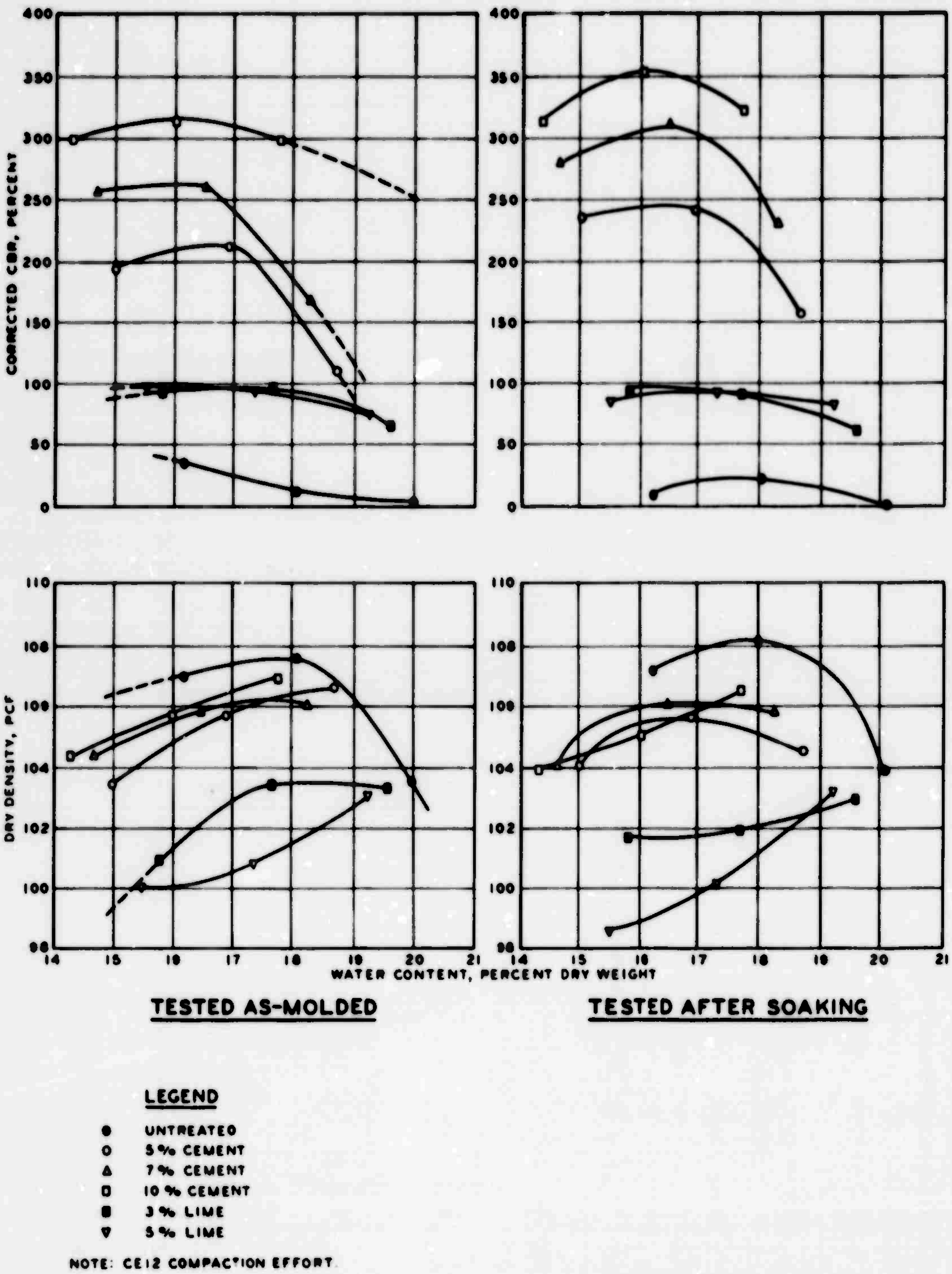
STABILIZED LEAN CLAY



NOTE: FROM REFERENCE 7.

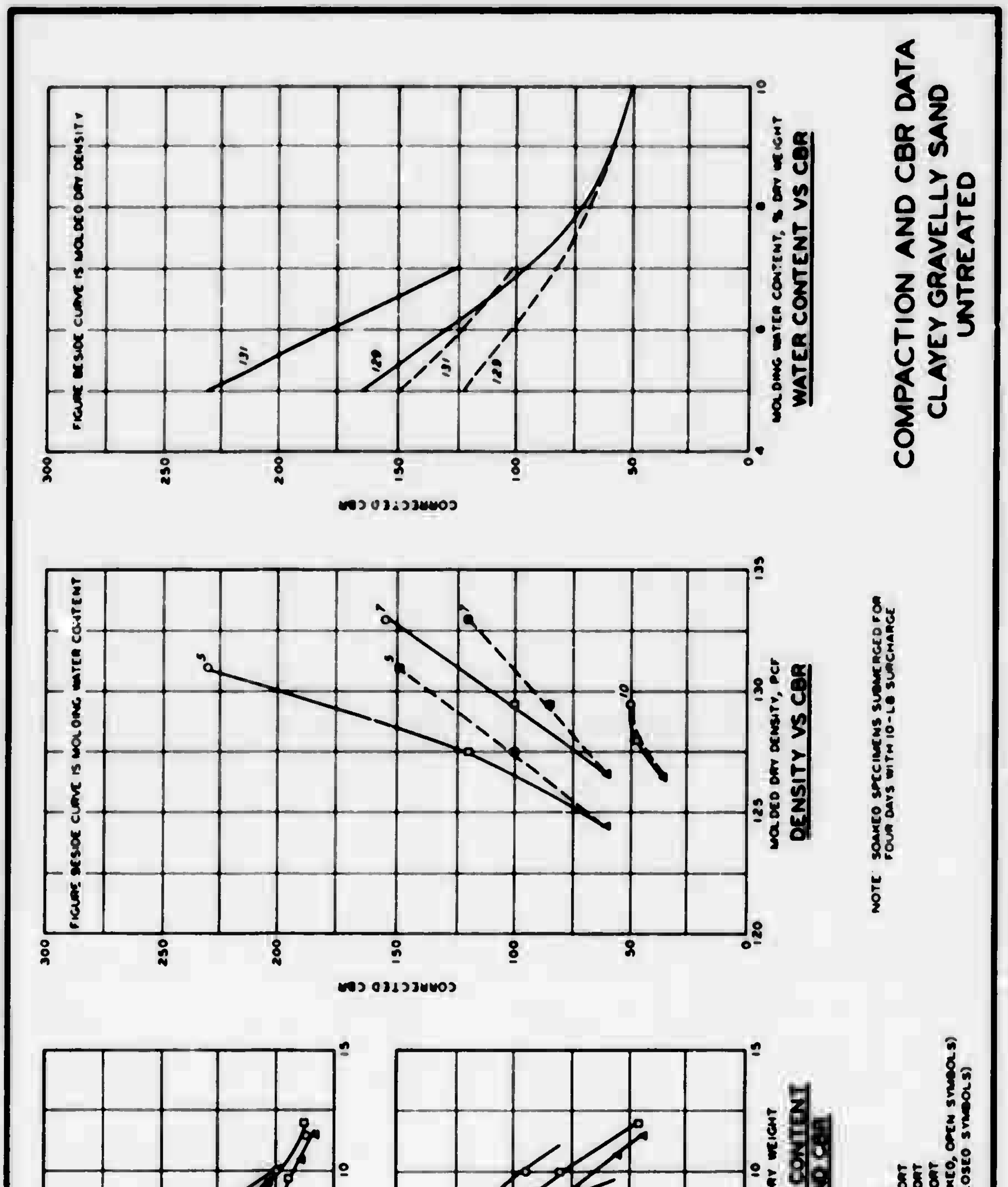
RELATION BETWEEN CBR AND UNCONFINED COMPRESSIVE STRENGTH
CEMENT-STABILIZED LEAN CLAY

PLATE 7



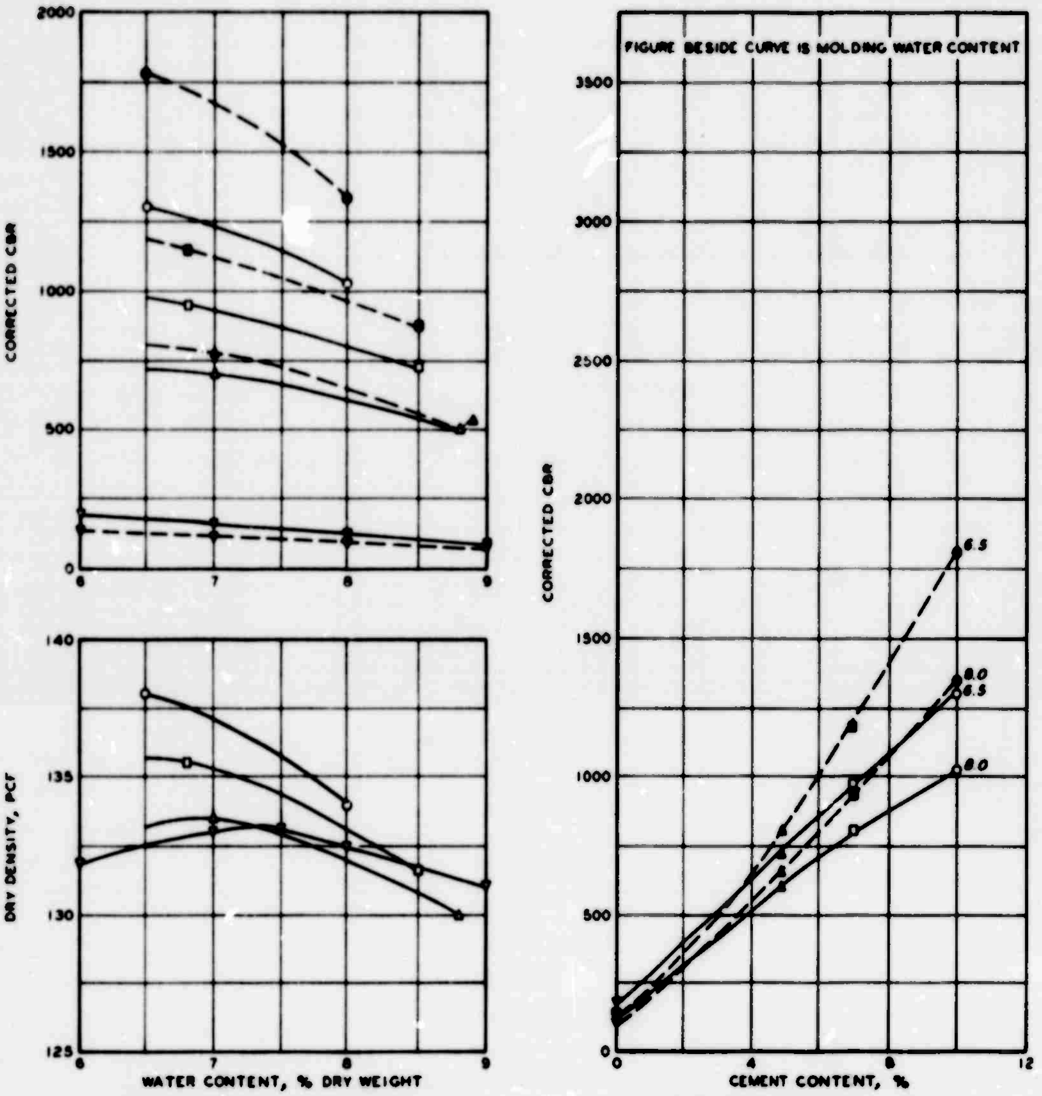
CBR, DENSITY, AND WATER CONTENT DATA
LEAN CLAY
TESTED AS-MOLDED AND AFTER SOAKING

PLATE 8



COMPACTION AND CBR DATA
CLAYEY GRAVELLY SAND
UNTREATED
WATER CONTENT VS CBR
FIGURE BESIDE CURVE IS MOLDED DRY DENSITY
MOLDING WATER CONTENT, % DRY WEIGHT
CORRECTED CBR
DENSITY VS CBR
FIGURE BESIDE CURVE IS MOLDING WATER CONTENT
MOLDED DRY DENSITY, PCF
NOTE: SOAKED SPECIMENS SUBMERGED FOR FOUR DAYS WITH 10-LB SURCHARGE
MOLDING WATER CONTENT VS DENSITY AND CBR
WATER CONTENT, % DRY WEIGHT
DRY DENSITY, PCF
LEGEND
CE 55 COMPACTION EFFORT
CE 26 COMPACTION EFFORT
CE 12 COMPACTION EFFORT
CBR AS MOLDED (UNSOAKED, OPEN SYMBOLS)
CBR AFTER SOAKING (CLOSED SYMBOLS)

PLATE 9



MOLDING WATER CONTENT VS DENSITY AND CBR

CEMENT CONTENT VS CBR

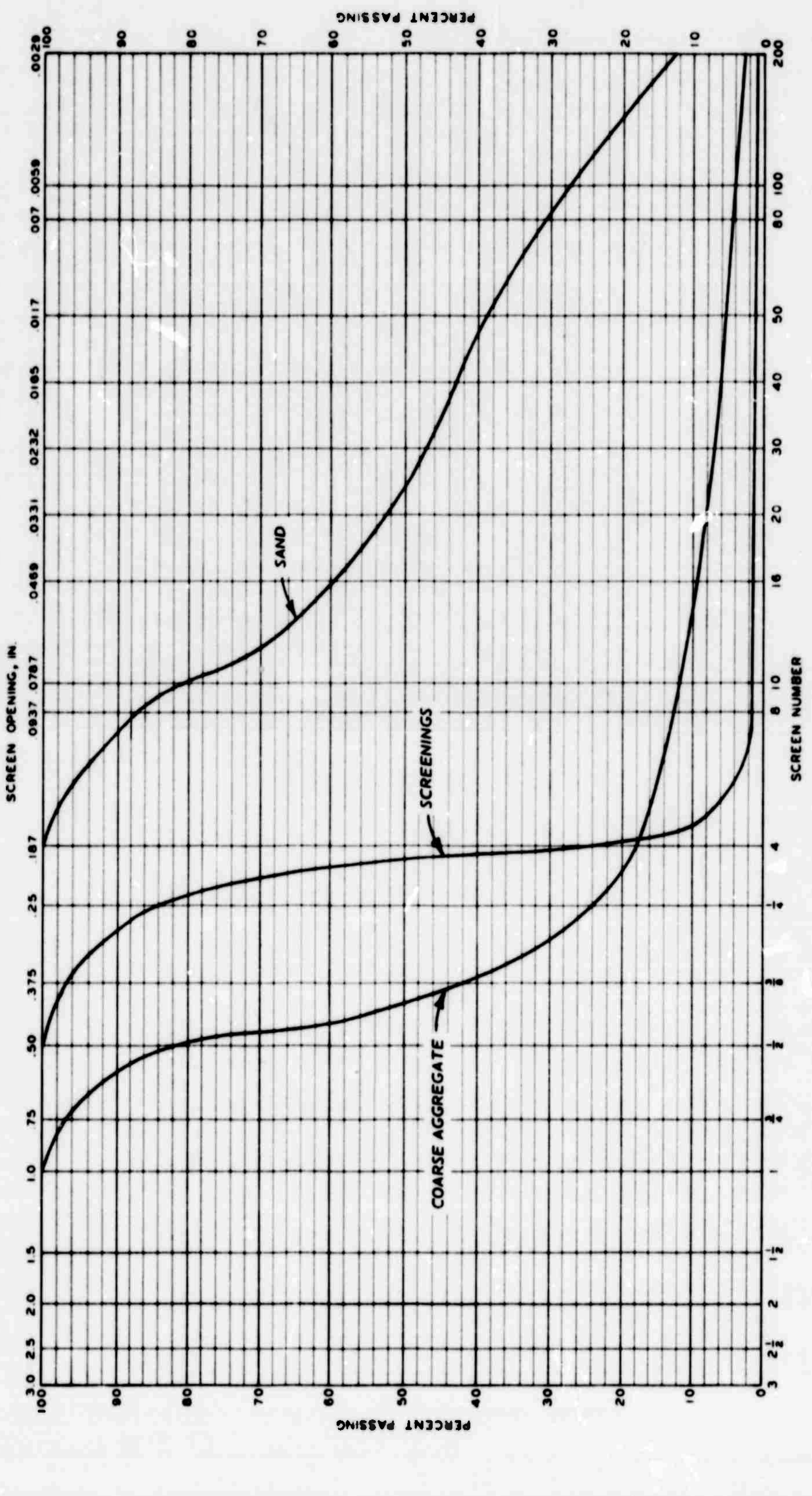
LEGEND

- ▼ UNTREATED
- ○ TREATED WITH 10% CEMENT
- □ TREATED WITH 7% CEMENT
- △ TREATED WITH 5% CEMENT
- ——— CBR AFTER 7 DAYS MOIST CURE (OPEN SYMBOLS)
- – – – CBR AFTER 7 DAYS MOIST CURE PLUS 4 DAYS SOAKING (CLOSED SYMBOLS)

NOTE: CE 55 COMPACTION EFFORTS

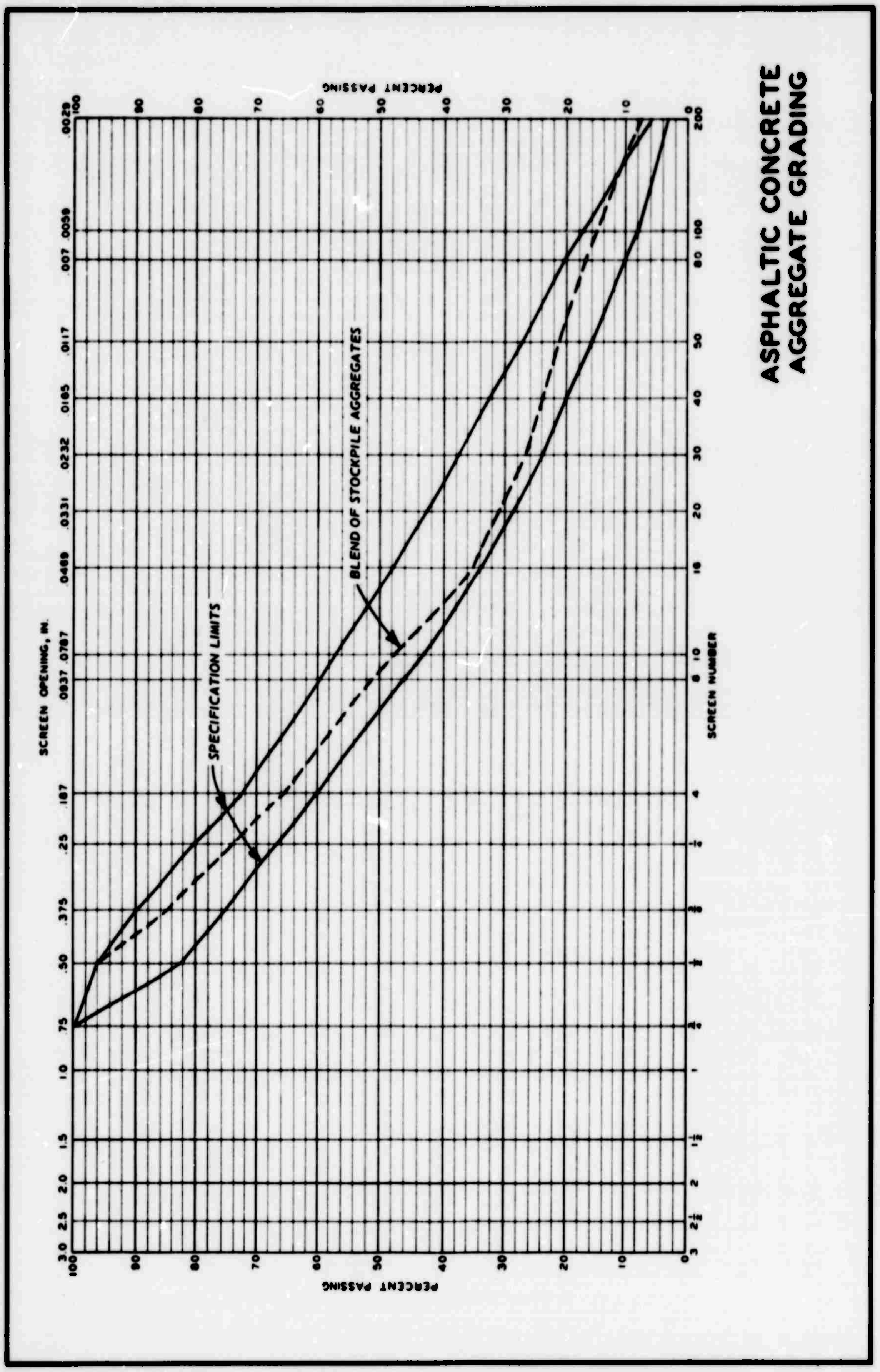
EFFECT OF MOLDING WATER CONTENT AND CEMENT CONTENT ON STRENGTH OF CLAYEY GRAVELLY SAND

PLATE 10



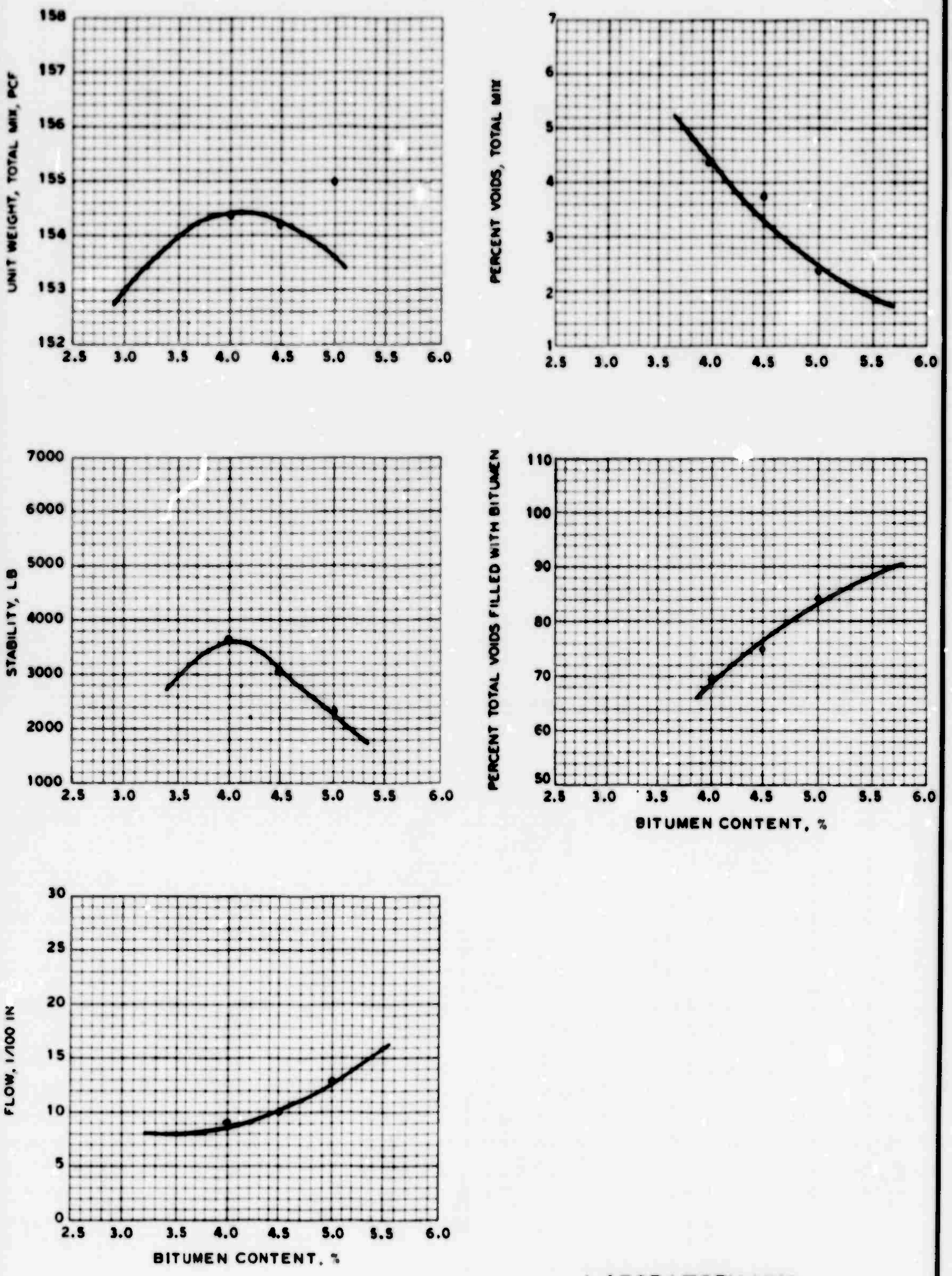
GRADING CURVES
STOCKPILE AGGREGATES

PLATE 11



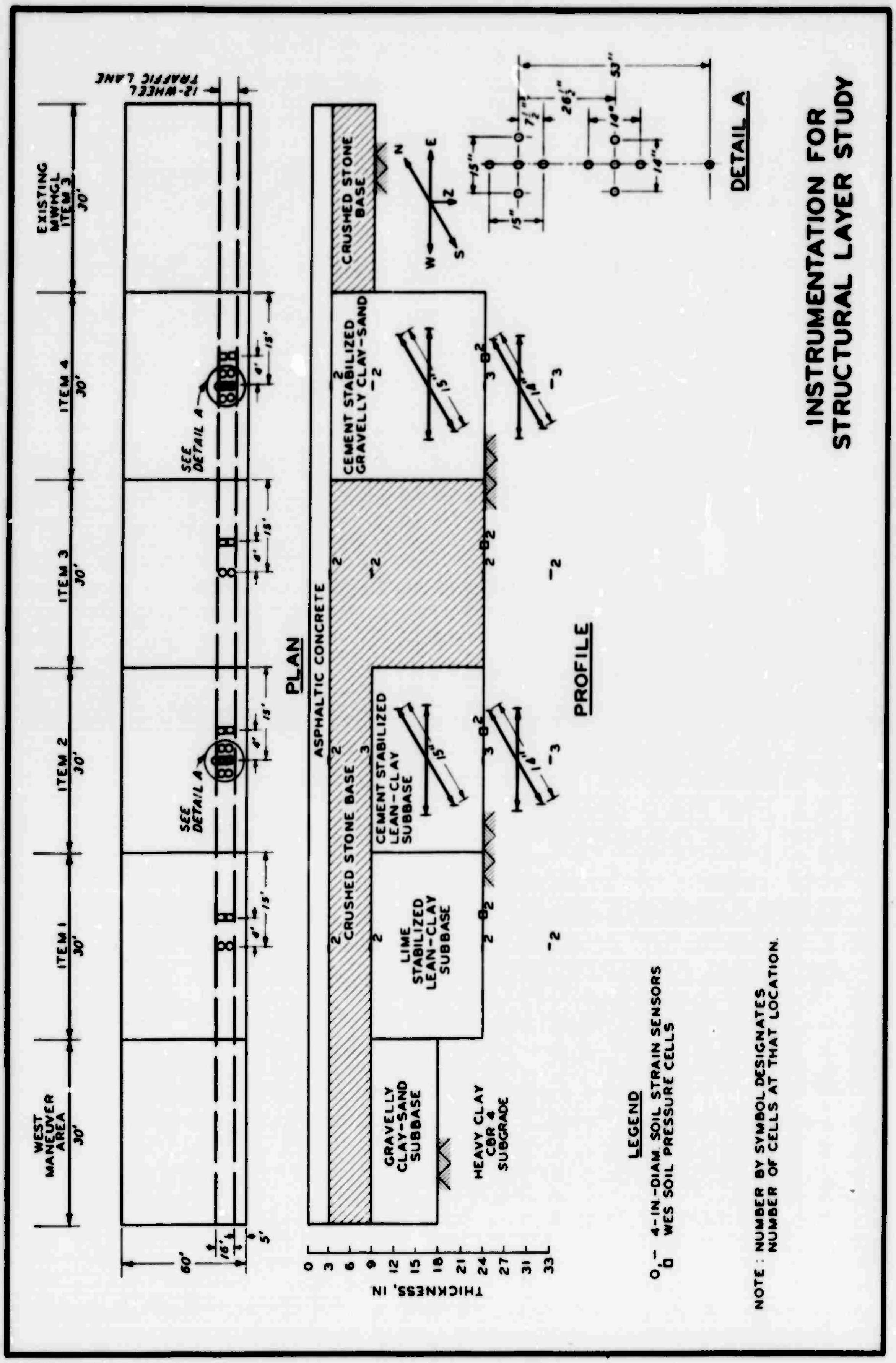
ASPHALTIC CONCRETE
AGGREGATE GRADING

PLATE 12



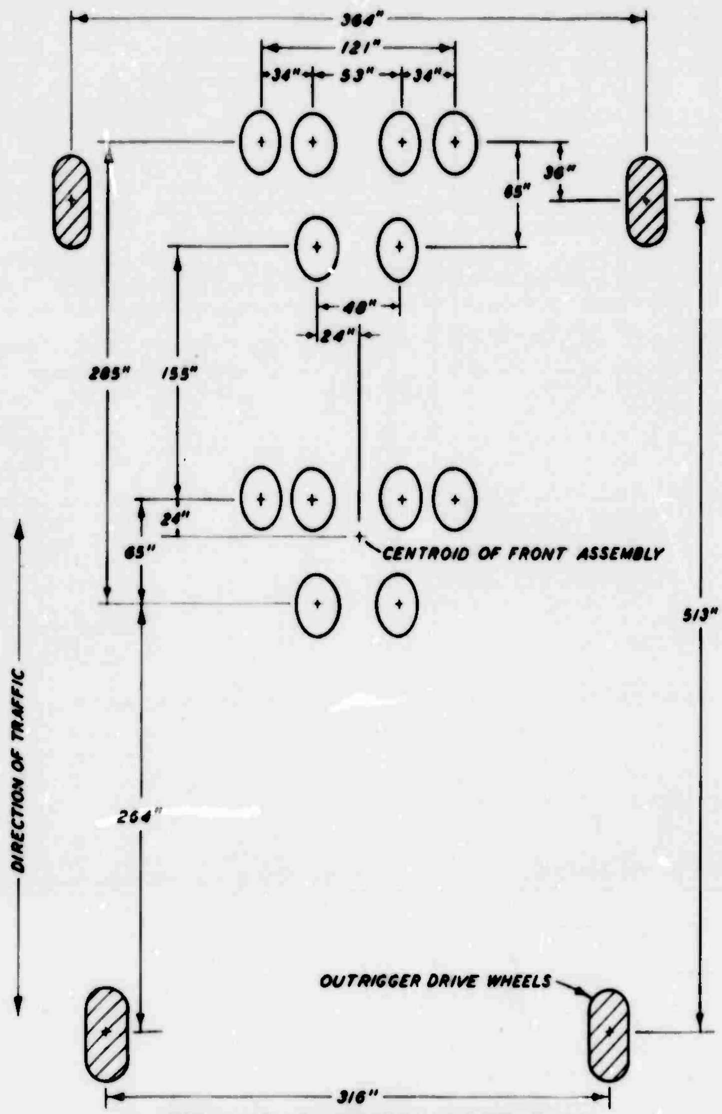
LABORATORY MIX
DESIGN PROPERTIES

ASPHALTIC CONCRETE

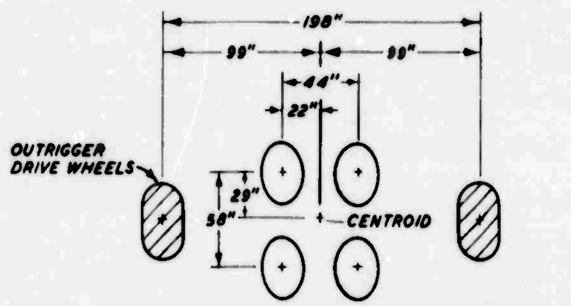


INSTRUMENTATION FOR STRUCTURAL LAYER STUDY

PLATE 14

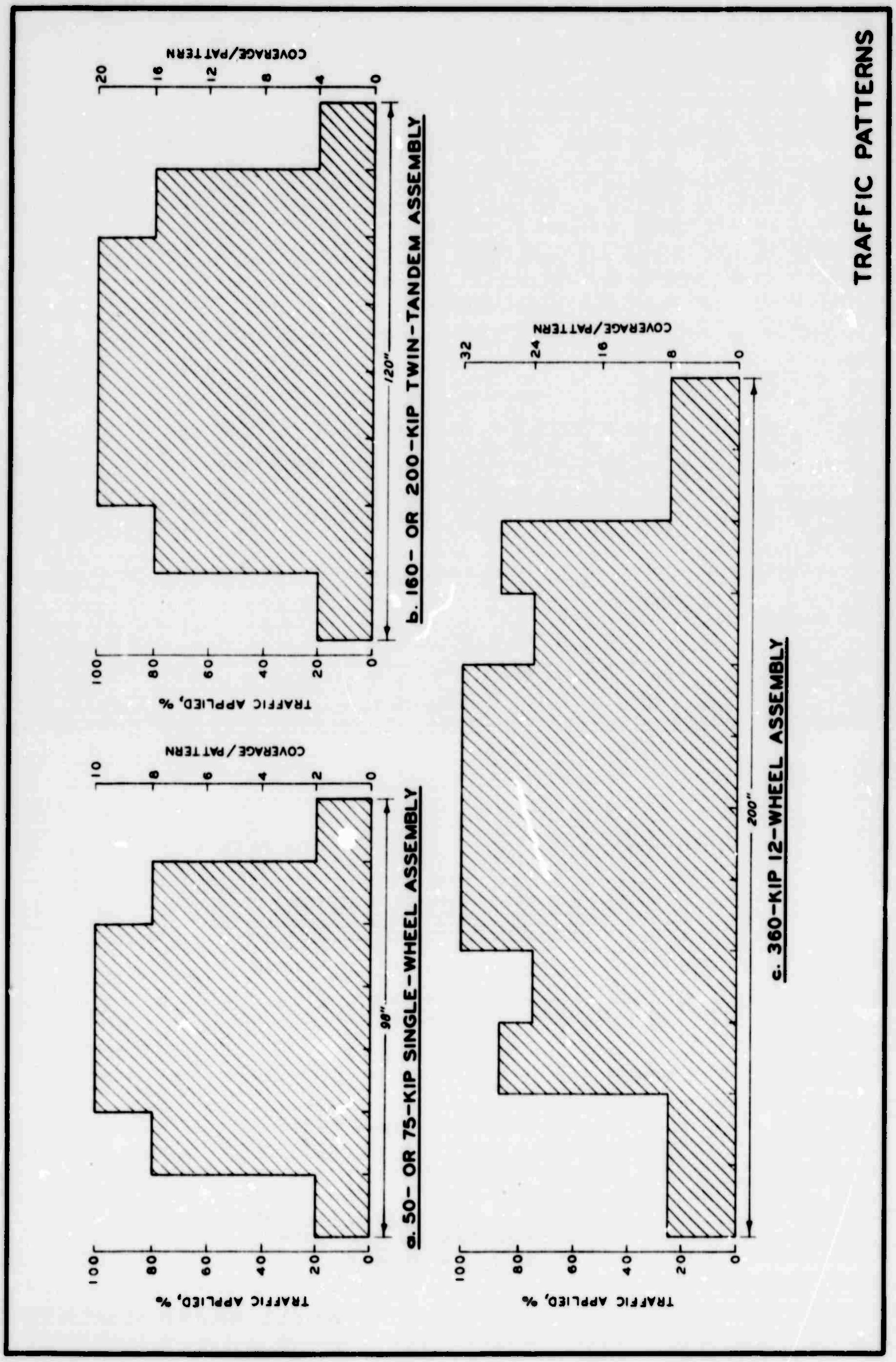


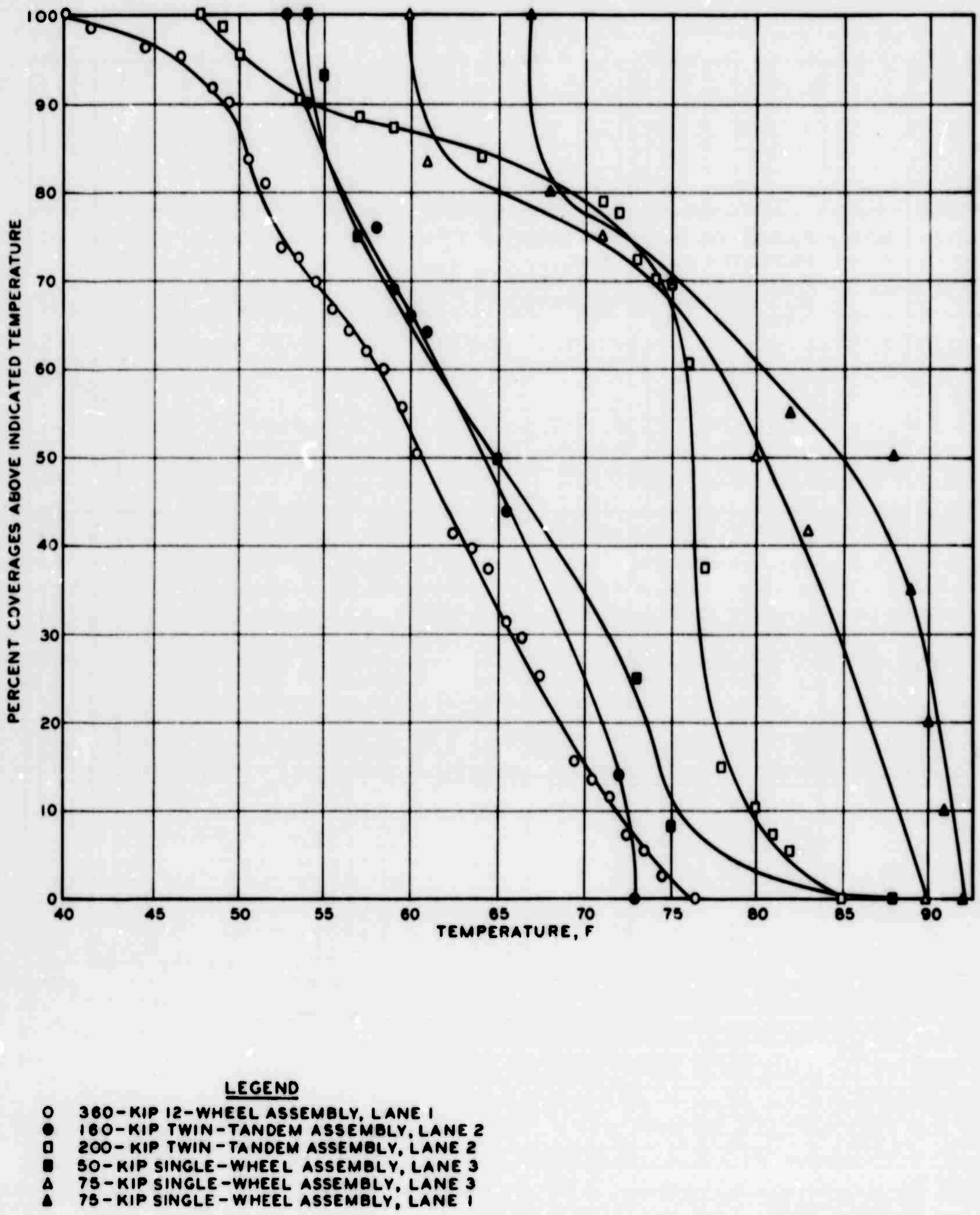
a. 360-KIP 12-WHEEL ASSEMBLY
(ONE MAIN GEAR OF C-5A)



b. 160- OR 200-KIP TWIN TANDEM
(ONE TWIN-TANDEM COMPONENT
OF 747 ASSEMBLY)

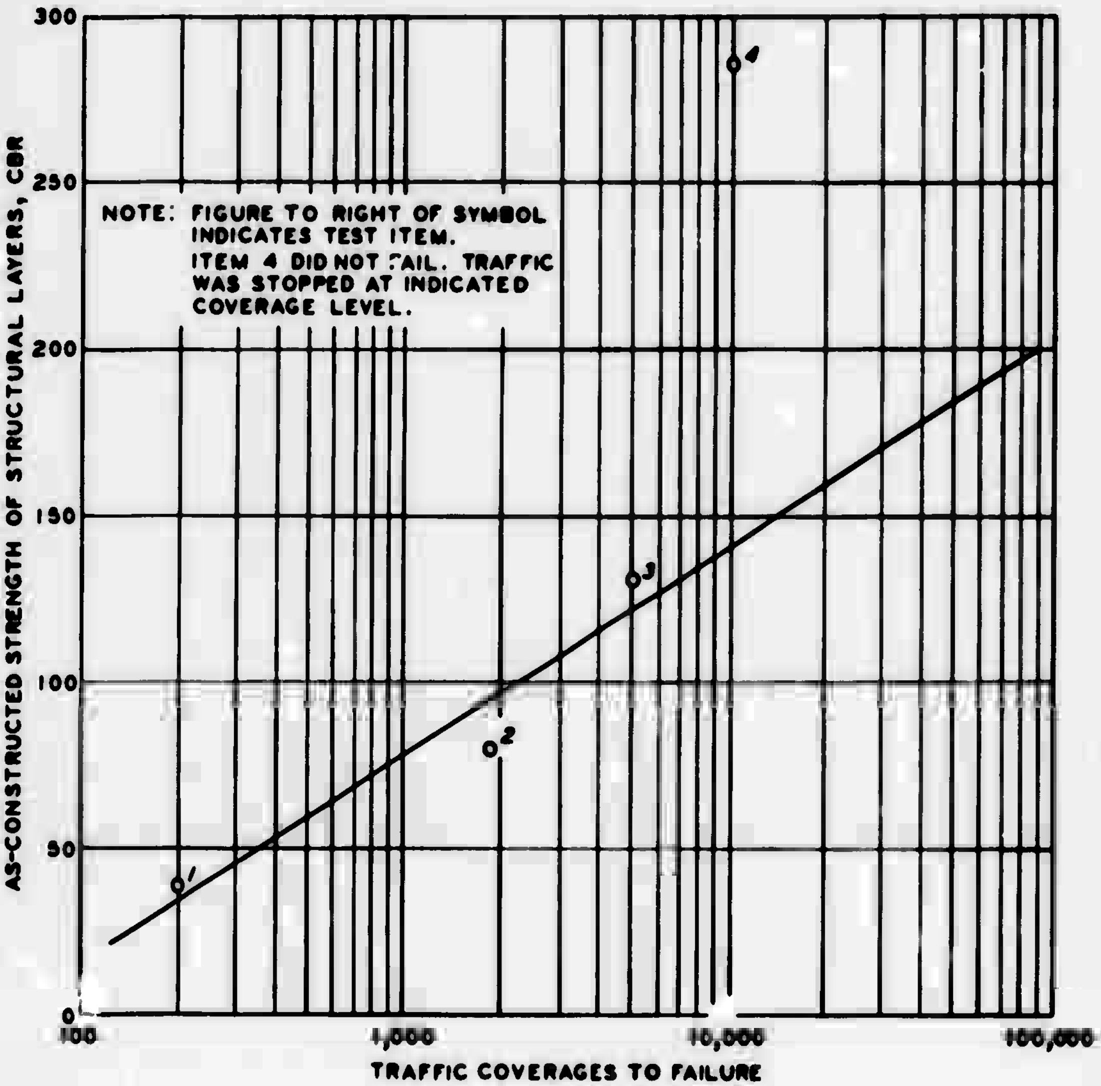
WHEEL ARRANGEMENTS


TRAFFIC PATTERNS
a. 50- OR 75-KIP SINGLE-WHEEL ASSEMBLY
b. 160- OR 200-KIP TWIN-TANDEM ASSEMBLY
c. 360-KIP 12-WHEEL ASSEMBLY
TRAFFIC APPLIED, %
COVERAGE/PATTERN
0
20
40
60
80
100
0
2
4
6
8
10
0
4
8
12
16
20
0
8
16
24
32
98"
120"
200"

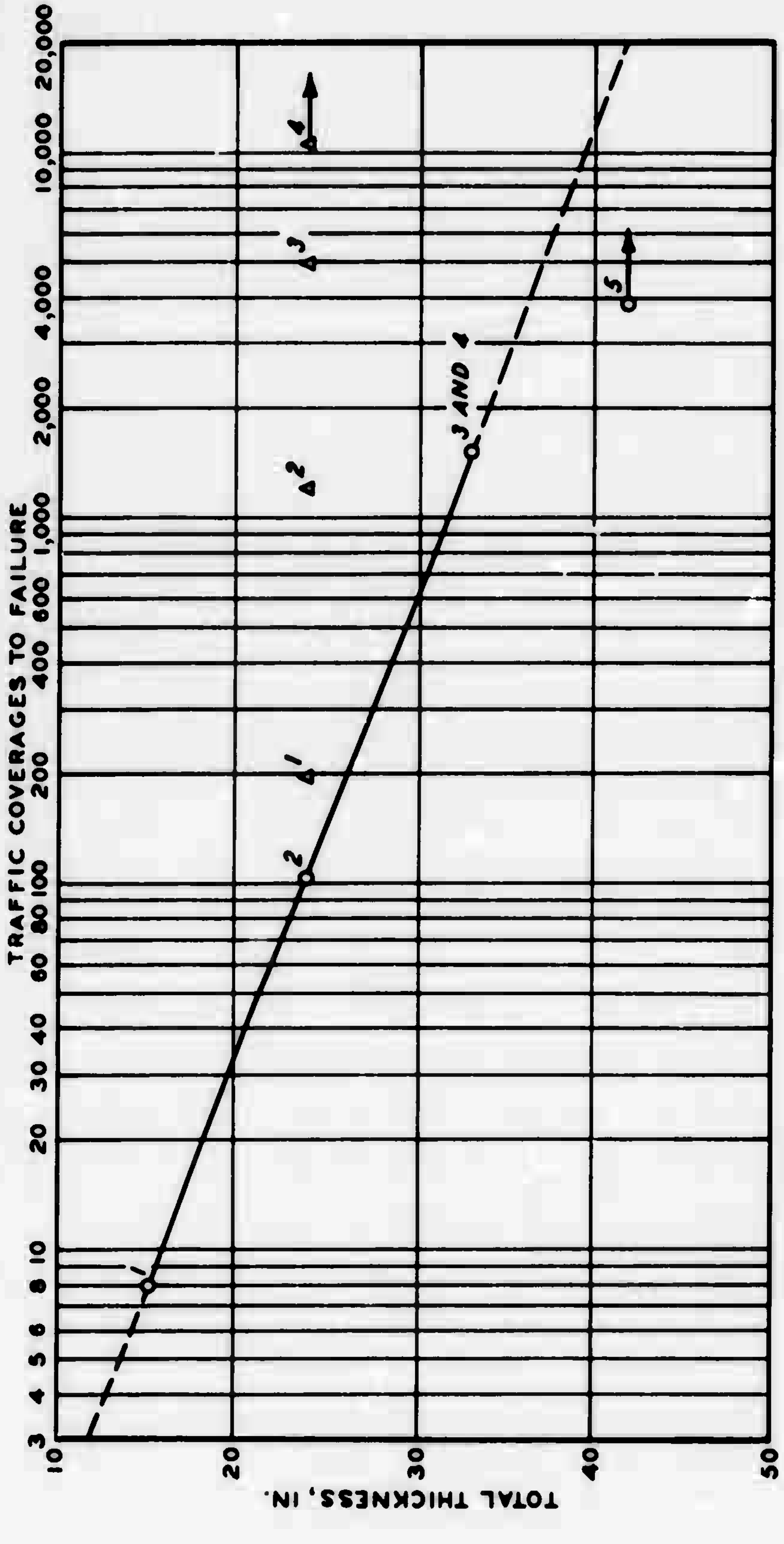


NOTE: TEMPERATURES ARE THE AVERAGE OF THOSE MEASURED AT THE TOP AND BOTTOM OF THE PAVEMENT.

TRAFFIC AND PAVEMENT TEMPERATURE DISTRIBUTION



COVERAGES VS CBR AND DEFLECTION
360-KIP, 12-WHEEL ASSEMBLY
100-PSI TIRE PRESSURE



COVERAGES VS THICKNESS

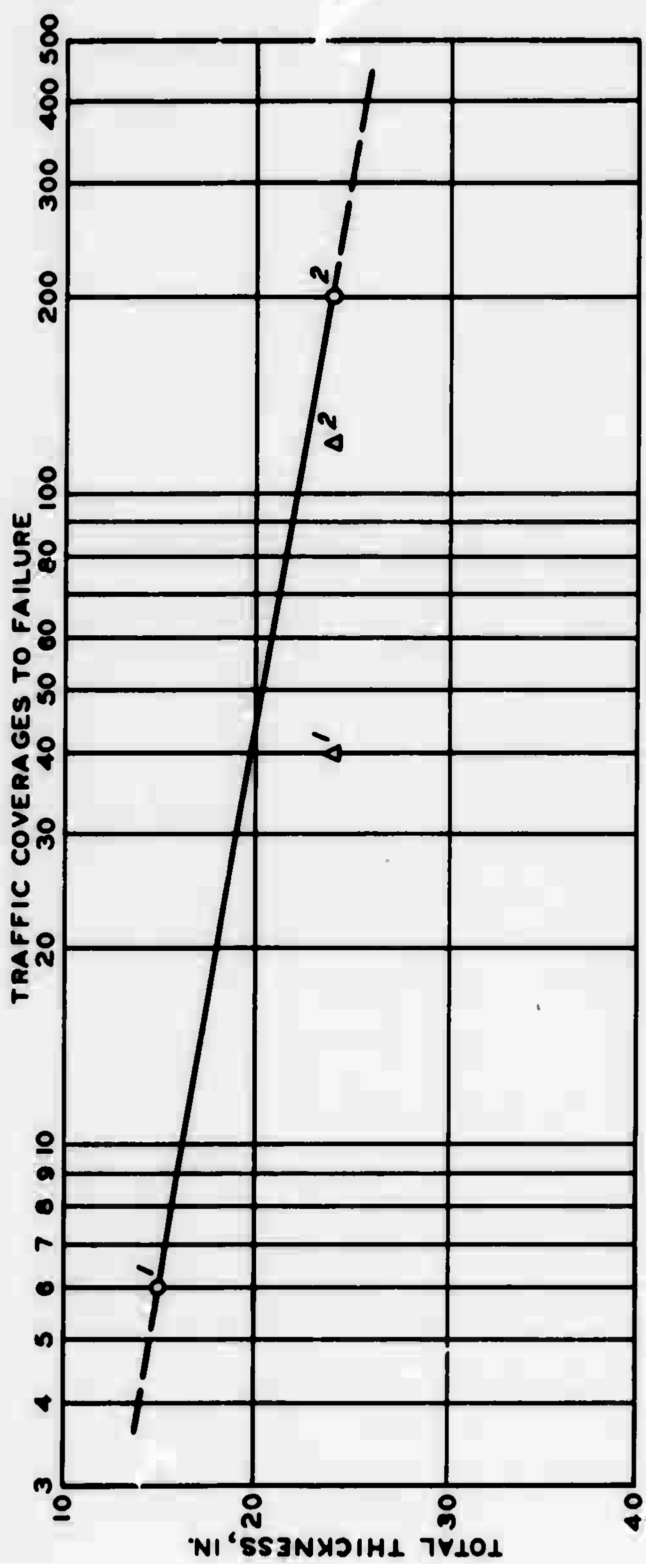
360-KIP 12-WHEEL ASSEMBLY
100-PSI TIRE PRESSURE

LEGEND

O MWHGL TEST SECTION
Δ STRUCTURAL LAYERS

NOTE: FIGURE TO RIGHT OF SYMBOL INDICATES TEST ITEM.
ITEM 5 OF MWHGL TEST SECTION AND ITEM 4 OF STRUCTURAL LAYER TEST SECTION DID NOT FAIL.

PLATE 19



LEGEND

O MWHGL TEST SECTION

Δ STRUCTURAL LAYERS TEST SECTION

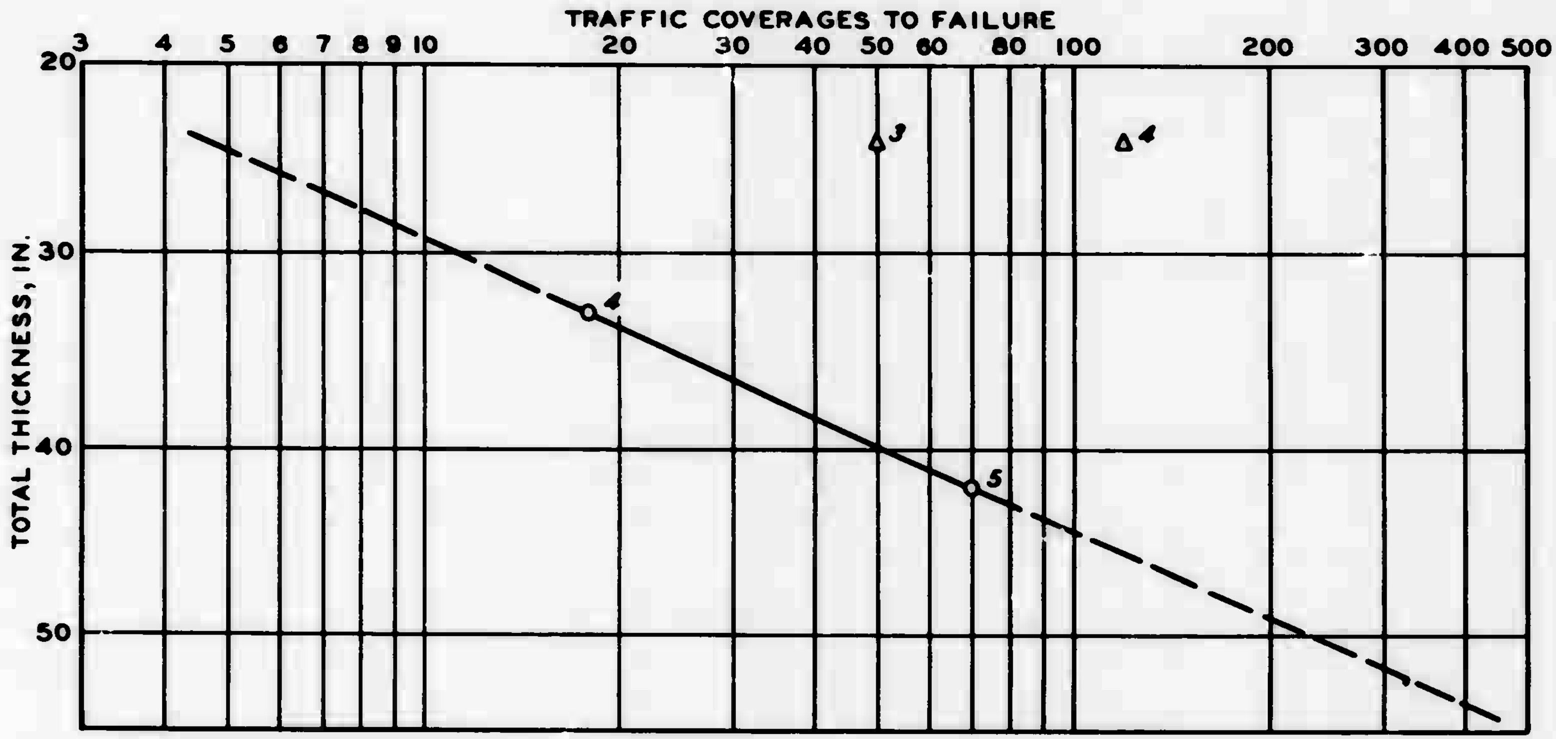
NOTE: FIGURE TO RIGHT OF SYMBOL INDICATES TEST ITEM.

COVERAGES VS THICKNESS

50-KIP SINGLE-WHEEL ASSEMBLY

170-PSI TIRE PRESSURE

PLATE 20



LEGEND

O MWHGL TEST SECTION

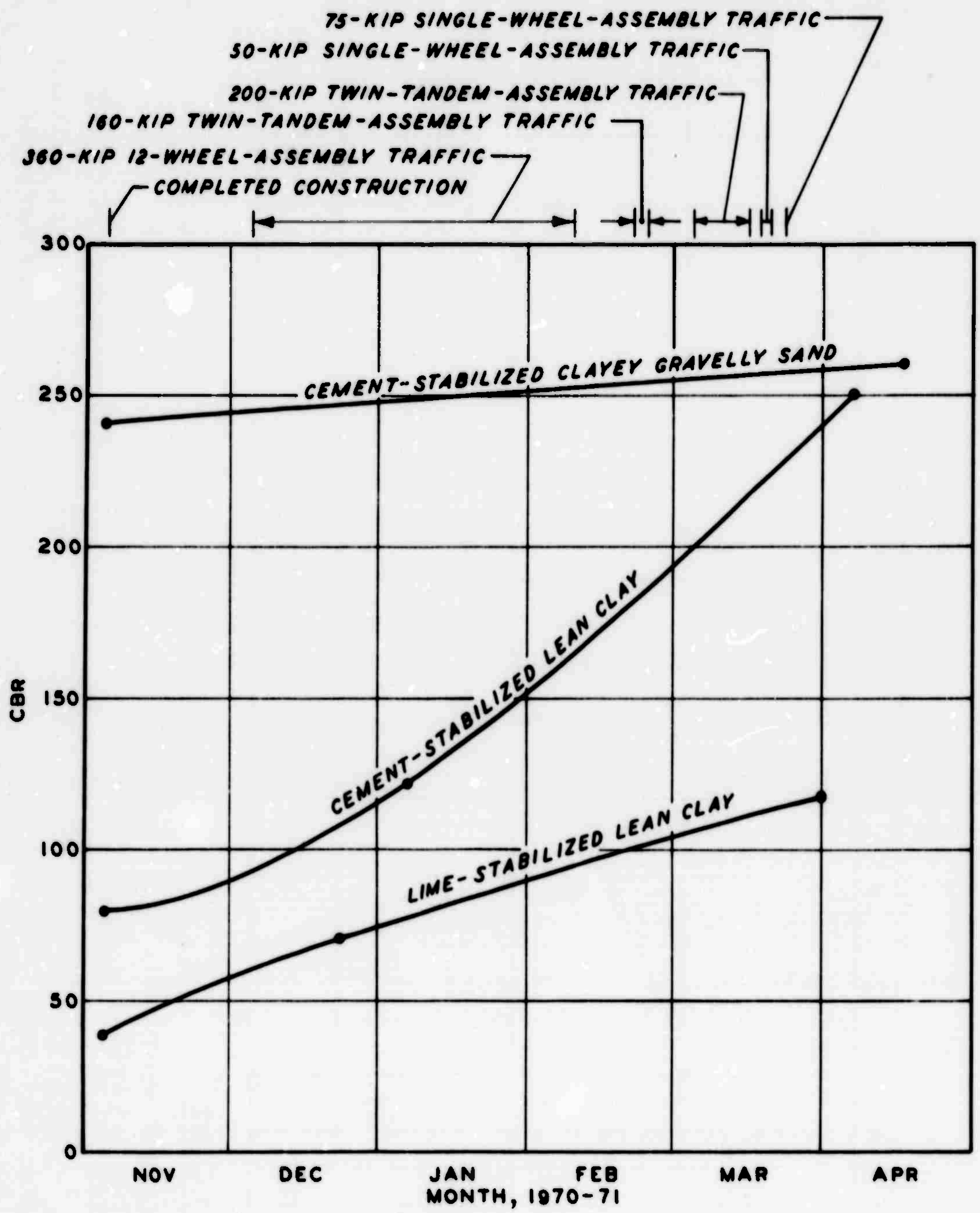
Δ STRUCTURAL LAYERS TEST SECTION

NOTE: FIGURE TO RIGHT OF SYMBOL INDICATES TEST ITEM.

COVERAGES VS THICKNESS

75-KIP SINGLE-WHEEL ASSEMBLY

290-PSI TIRE PRESSURE



NOTE: TESTS MADE IN NONTRAFFICKED AREAS.

EFFECT OF TIME ON STRENGTH OF STABILIZED SOIL

PLATE 22

APPENDIX A: INSTRUMENTATION

1. All test items were instrumented with stress gages (pressure cells) and strain sensors, as shown in plate A1. Stresses were measured only at the top of the subgrade, but movements within the soil-pavement system were measured in the subbase and subgrade of items 2 and 4. The vertical movements can be accumulated arithmetically to give partial deflection; all of the movements can be divided by their respective gage lengths to determine vertical and horizontal strains.

2. A total of 61 instruments were installed: 8 stress gages and 53 strain sensors. The stress gages and motion sensors were all installed in duplicate. The instrumentation in each item was located across the center of the 12-wheel traffic lane and in the center of each item. The locations of the instruments were 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 tires of the front six wheels would be directly over duplicate instruments. From previous work with the 12-wheel assembly (volume III, reference 3*), it has been determined that either of the back inside tires of the front six wheels is the maximum load point for the range of depths investigated in these items.

Description of Gages and Sensors

3. The stress gages were WES 50-psi earth pressure cells. These cells are accurate to ±10 percent of the indications. Detailed descriptions of the WES earth pressure cells and their installation are given in references 12 and 13.

4. The strain sensors were manufactured by Bison Instruments, Inc. A strain sensor system consisting of sensors and an external instrument package was used for the tests. The sensors were individual

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* All references refer to similarly numbered items in the Literature Cited which follows the main text.

disk-shaped coils, and their principle of operation involved the electromagnetic mutual inductance coupling of two sensors. A receiver sensor can be placed anywhere within the electromagnetic field surrounding a sensor excited by an oscillating current. Movement of one sensor with respect to the other results in a change of the electromagnetic coupling between the two. Maximum coupling change will occur when a sensor moves normal with respect to the lines of equal potential, and this indicates primarily an x-movement in a three-dimension case. A y- or z-movement or rotation is a secondary movement and is in the apparent order of 5 to 15 times the x-movement required to produce the same coupling change. There are three alignments that result in couplings normal to the lines of equal potential: vertical, lateral, or perpendicular. Vertical and lateral alignments result in movements along a line of 45 degrees on an equal potential plot, and perpendicular alignments are along a 1-to-2 sloping line. Movement in any other alignment results in a coupling change indicating the resultant of an x-, y-, and z-motion.

5. The change in electromagnetic coupling is a nonlinear function of movement; however, the change can be calibrated very accurately with resolution and repeatability of spacing change better than 0.0001 in. Once the calibration curve is established, movements smaller than 0.000001 in. can be accurately measured; however, depending on the use and application, the modes of the measured motion (y-, z-, and/or rotational motion) will establish the magnitude of the significant numbers if the motion modes have not be n included in the calibration curve.

6. The sensors have no mechanical connection between them and operate at any spacing between one and four times the nominal sensor diameter as long as there is no disturbance of the induced electromagnetic field, such as metal between or around the sensors. Magnetic or highly conductive material within the electromagnetic field has two possible effects: to act as a sink absorbing energy, thus reducing the field strength; or to act as a conductor, thus amplifying the field strength. Both of these effects have been observed.

7. The physical environment of the sensors, both constant or changing, needs to be and can be calibrated. If this is done and the

y-, z-, and rotational modes of motion are included in the calibration curves, the significant figure is less than 1×10^{-6} in. The observed effects of environment are to shift the calibration curves parallel so that the change along a curve is always constant.

8. The sensors have an AC current excitation and can be connected to the voltage readout equipment in only one manner for them to work, and the coupling either increases (movement together) or decreases (movement apart) in strength.

9. 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 as well as a self-contained power supply. Changes in sensor spacing could be 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 could detect both static and dynamic strain. Response time of the instrument was about 0.1 msec.

10. Bison strain sensors are available in 1-, 2-, and 4-in.-diam sizes; the 4-in.-diam size was used in this study. Photo A1 shows one 4-in.-diam coil. Columns of sensors can be used as shown in plate A1 with the interior coils of the columns acting as common sensors to two locations.

Installation

11. Installation of the instruments was accomplished in conjunction with the construction of the test section. Locations of the gages and sensors are shown in plate A1; the plan and layout of the test section are shown in plate A2.

12. After the subgrade was prepared and fine bladed, 4-in.-diam holes for the vertical column gages were augered to a 33-in. depth in each item. Sensors were placed, the holes were backfilled with the original material, and the material was hand tamped. In items 2 and 4, 1/4- by 4-in. slots were cut with a knife to the proper depth, and the

sensors for the horizontal measurements were installed in such a manner as to form an imaginary axis centered 4.5 in. below the subgrade. The gages at the top of the subgrade were not installed until after the first lift of subbase had been pulvimixed and compacted in order to avoid destruction of the gages during the pulvimixing operation. Immediately after the compaction, 4- and 6-in.-diam holes for the duplicate sensors and stress gages, respectively, were cut to the top of the subgrade.* The gages and sensors were placed, and the holes were backfilled before the stabilized subbase material began curing. For item 3, the gages and sensors were also installed after placement of the first lift. The stress gages were seated in the subgrade, and a layer of fines from the crushed stone base course material was placed over them for protection from abrasion by large aggregate particles. In item 4, fines from the clayey gravelly sand were used to protect the stress gages.

13. The construction operation continued to a few inches above a 16.5-in. depth, and sensors for horizontal measurements were placed in items 2 and 4, as shown in plate A1. The sensors were placed in slots using the same procedure described for the subgrade. When the construction operation reached the 9-in. depth, all sensors for this depth were placed, and the construction continued on top of them. When the 3-in. depth was reached, all sensors and cables for the pavement/base interface were placed and then covered with a thin layer of cold-mix asphalt for protection from the heat anticipated from the hot mix. The hot-mix asphaltic-concrete surface was then placed and compacted above the coils. All coil and gage cables were run in a cable trench at each installation elevation. The cable trenches ran to the north side of the

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* Had the subbase material not been mixed in place, the sensor installations would all have been made when the proper elevations were reached in the construction operations. Even though holes were cut and backfilled, the soil disturbance was believed to have been held to a minimum and not to have significantly affected the sensors. Any disturbance of the soil surrounding a sensor or soil arching action caused by a rigid sensor was negligible in comparison with the distance and area of undisturbed soil between two sensors.

test section, where the cables were carried into a central instrumentation house. This last step completed the instrumentation installation.

Collection of Instrumentation Data

14. Plate A2 gives the gear configuration, load, and type of load test conducted on each test item, and plate A3 shows the gear arrangements and spacings. As mentioned in paragraph 2 of this appendix, the gages and sensors were located so that the back inside tires of the front six wheels of the 12-wheel assembly would be in position over duplicate instruments at the time of loading. For other wheel assemblies, the gage and sensor locations were loaded independently; the stress gages were loaded first, and then the strain sensors.

15. As each item was statically loaded, the gages and 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 loading reading was taken. After loading an item, the assembly was kept off of it until approximate equilibrium of recovery was reached (volume III, reference 3). These first static load tests were made with the 30-kip-per-wheel single- and 12-wheel assemblies before any traffic was allowed on the test section. Twin-tandem and single-wheel, 50-, 60-, and 75-kip load tests were made after 10,000 coverages of the 12-wheel assembly on item 4. A second load point was used after the one under the tire, and it was located at the centroid of the back four of the front six wheels of the 12 wheels and at the centroid of the twin-tandem assembly. Data were recorded under static test loads at various coverage levels, under special moving load tests (runs about 3-5 mph down selected traffic rows), and under all of the 12-wheel traffic (3-5 mph) on each item. Recorded data for 12-wheel traffic to and past failure ranged from 201 coverages on item 1 to 10,000 coverages on item 4 and an additional 200 coverages of 75-kip single-wheel traffic on the 12-wheel lane of item 4. Table A1 shows a summary of the test schedule.

16. No gages were lost throughout the load testing and traffic;

however, prior to any tests, a strain sensor failed in item 4 at the 33-in. depth in the south column. The sensor had a shorted circuit and failure was probably due to moisture penetrating the coil housing. There was a duplicate sensor at the 33-in. depth in the north column.

Data Reduction and Analysis

17. An initial reduction of data has been performed; however, since the test section and instrumentation work under MEDECC was in progress at the time of this reporting, it has been considered appropriate to present the data and a formal analysis in a future report.[14] Listed in tables A2 and A3 are some preliminary maximum values measured in the structural layer test section.

18. The first step in data handling was the conversion of instrument indications into values of measurement in the form of pounds and inches. For the stress gages, each gage had a calibration factor to convert indications into pounds per square inch of pressure. Manually recorded indications were multiplied by the calibration factor; however, mechanically recorded indications were scaled directly into pounds per square inch. Calibration curves for the motion sensors were programmed into a computer, and the indications from both manual and mechanical recordings were also put in. The computer output was the distance in inches that each indication represented. From the above conversions, the data in the form of induced stress and movement were acquired by utilizing the hypothesis of soil behavior set forth in volume III-B of reference 3.

19. Tables A2 and A3 show indications of cyclic behavior patterns of both stress and strain, respectively. The mechanically recorded traffic data showed these patterns distinctly and indicated that they were a function of load position and history. This cyclic behavior has an amplitude that is a function of load, position, soil strength, and load repetitions, and it has a wave form that is a function of the number of gear wheels. Similar behavior patterns were recorded in the MWHGL tests (volume III-B, reference 3). Also incorporated in this behavior are

vertical strain reversals from compression to tension that are a function of load, position, and soil strength. These same behavior patterns (only isolated portions of the above-mentioned wave form) were observed within the structures of the Stockton Airfield tests in the late 1940's.[15]

Table A1

Schedule of Instrumentation Tests

Item No.	Gear Configuration	Load kips	Type of Load Test
1	Single wheel	30	Static
	C-5A twelve wheel	360	Static, traffic
2	Single wheel	30	Static
	C-5A twelve wheel	360	Static, traffic
3	Single wheel	30	Static
	C-5A twelve wheel	360	Static, traffic
4	Single wheel	30	Static
	Single wheel	50	Static, moving*
	Single wheel	60	Static, moving*
	Single wheel	75	Static, moving,* traffic
	747 twin tandem	160	Static
	747 twin tandem	200	Static
	C-5A twelve wheel	360	Static, traffic

* Runs down selected traffic rows.

Table A2

Maximum Static Vertical Elastic Subgrade Stress

(Measured Under Load Wheels)

Item No.	Coverages	Stress, psi: Single-Wheel Load, kips: 30	50	60	75	Twin-Tandem Load, kips: 160	200	12-Wheel Load 360 kips
1	0	18.1	--	--	--	--	--	20.4
	96	--	--	--	--	--	--	27.3
	201	--	--	--	--	--	--	24.2
2	0	15.3	--	--	--	--	--	18.4
	96	--	--	--	--	--	--	23.4
	1,327	--	--	--	--	--	--	12.4
3	0	25.8*	--	--	--	--	--	24.9*
	96	--	--	--	--	--	--	29.6*
	1,515	--	--	--	--	--	--	30.2*
	5,000	--	--	--	--	--	--	27.0*
4	0	7.3	--	--	--	--	--	10.0
	96	--	--	--	--	--	--	14.8
	1,515	--	--	--	--	--	--	16.2
	5,000	--	--	--	--	--	--	18.4
	10,000**	--	28.5	28.0	30.0	21.6	20.0	16.5

Note: Dashes indicate that no tests were performed.

* About 100-percent overregistration due to arching action of the crushed stone base material.

** After 10,000 coverages of the 12-wheel assembly, static tests were run in the following sequence: 12-wheel 360-kip load; twin-tandem 160- and 200-kip loads; single-wheel 50-, 60-, and 75-kip loads.

Table A3

Maximum Static Vertical Elastic Subgrade Strain

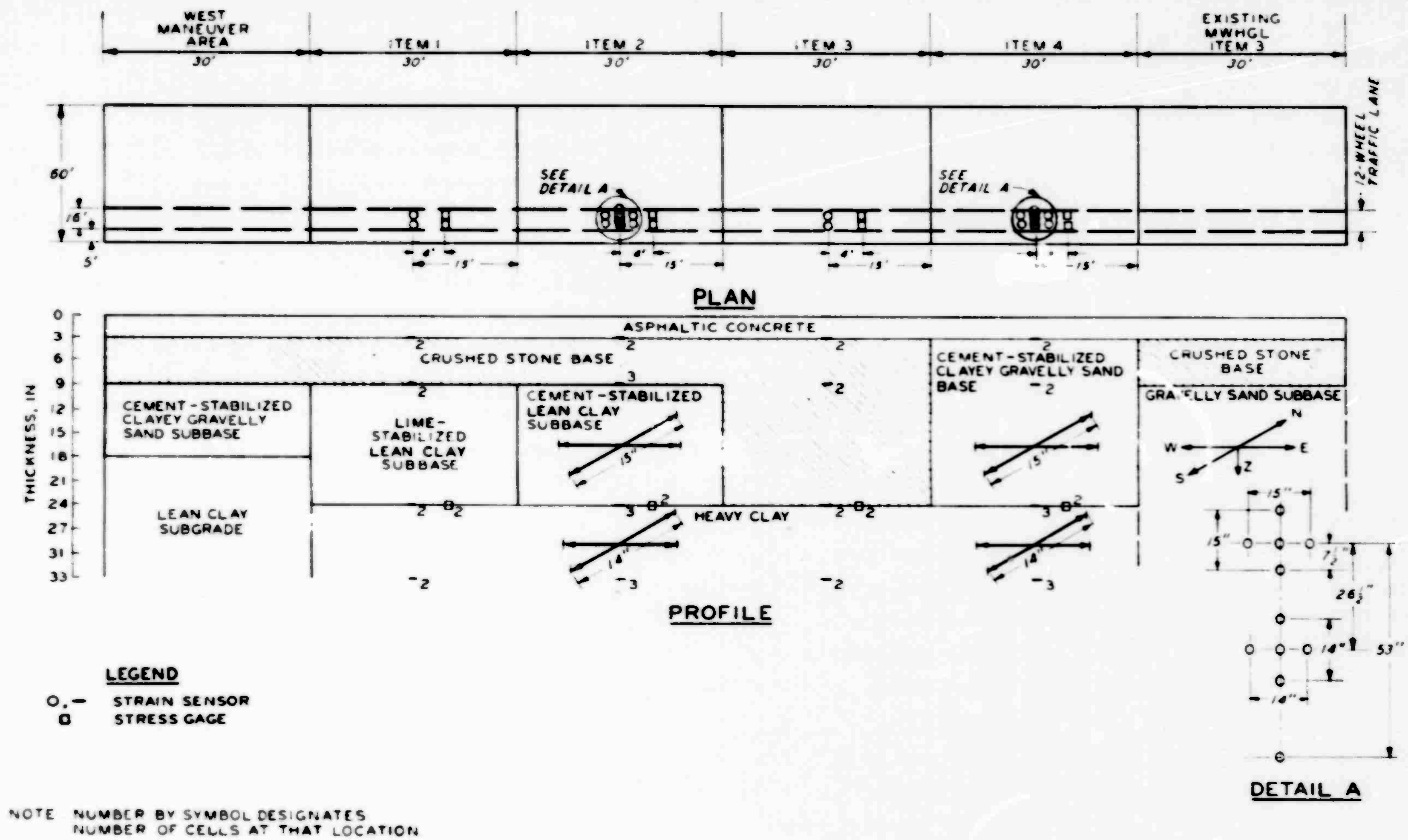
(Measured Under Load Wheels)

Item No.	Coverages	Strain, 10^{4} in./in.* Single-Wheel Load kips 30	50	60	75	Twin-Tandem Load, kips 160	200	12-Wheel Load 360 kips
1	0	+70	--	--	--	--	--	+60
	96	--	--	--	--	--	--	+69
	201	--	--	--	--	--	--	+52
2	0	+30	--	--	--	--	--	+39
	96	--	--	--	--	--	--	+47
	1,327	--	--	--	--	--	--	+00
3	0	-29	--	--	--	--	--	+38
	96	--	--	--	--	--	--	+38
	1,515	--	--	--	--	--	--	+00
	5,000	--	--	--	--	--	--	+30
4	0	+19	--	--	--	--	--	+31
	96	--	--	--	--	--	--	+101
	1,515	--	--	--	--	--	--	+18
	5,000	--	--	--	--	--	--	+24
	10,000**	--	+53	+58	+57	+34	+42	+20

* Plus symbol indicates compression; minus symbol used before value to indicate tension. Dashes indicate that no test was performed.

** After 10,000 coverages of the 12-wheel assembly, static tests were run in the following sequence: 12-wheel 360-kip load; twin-tandem 160- and 200-kip loads; single-wheel 50-, 60-, and 75-kip loads.

Photo A1. Four-in.-diam motion sensor



INSTRUMENTATION FOR STRUCTURAL LAYER STUDY

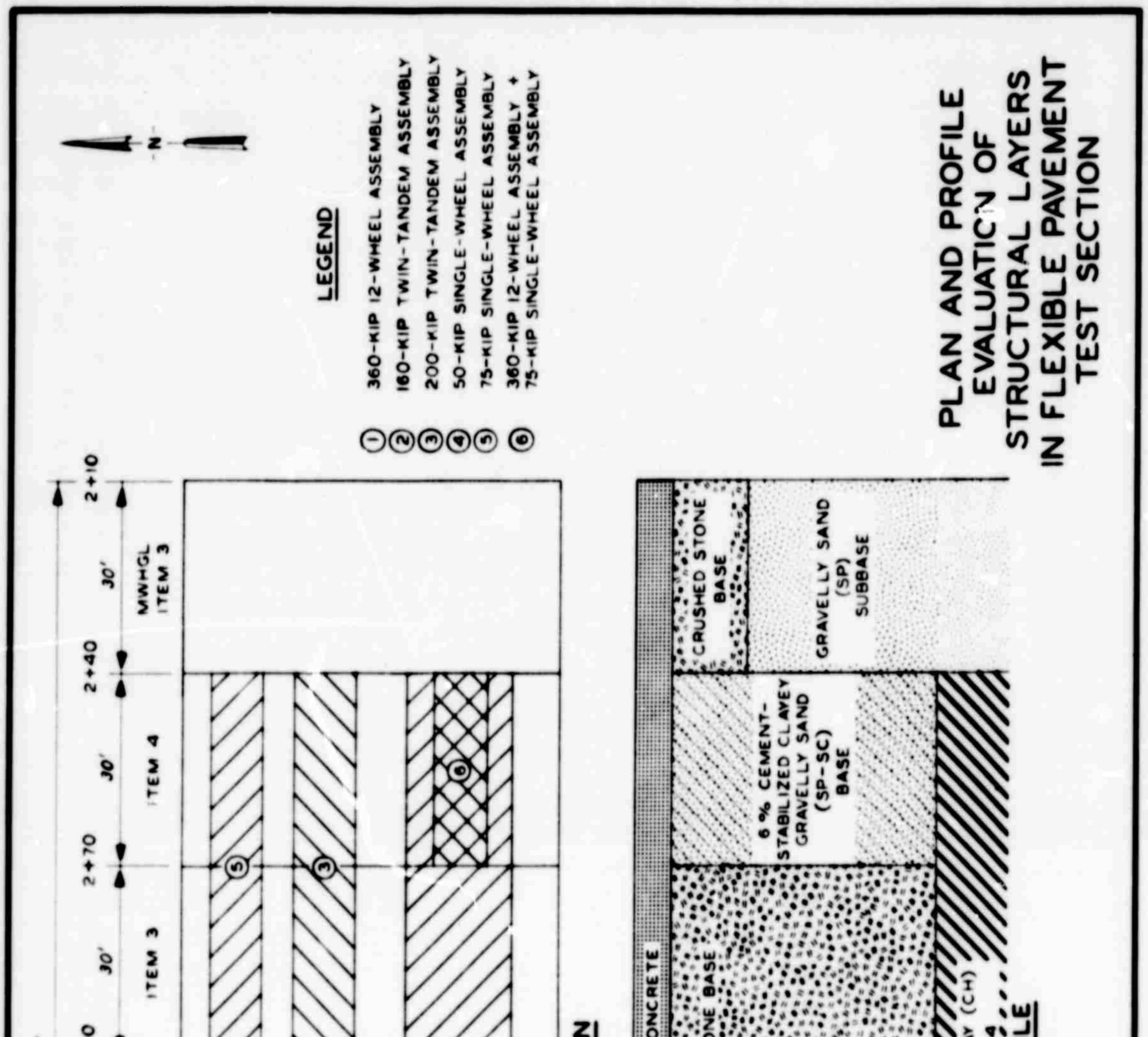
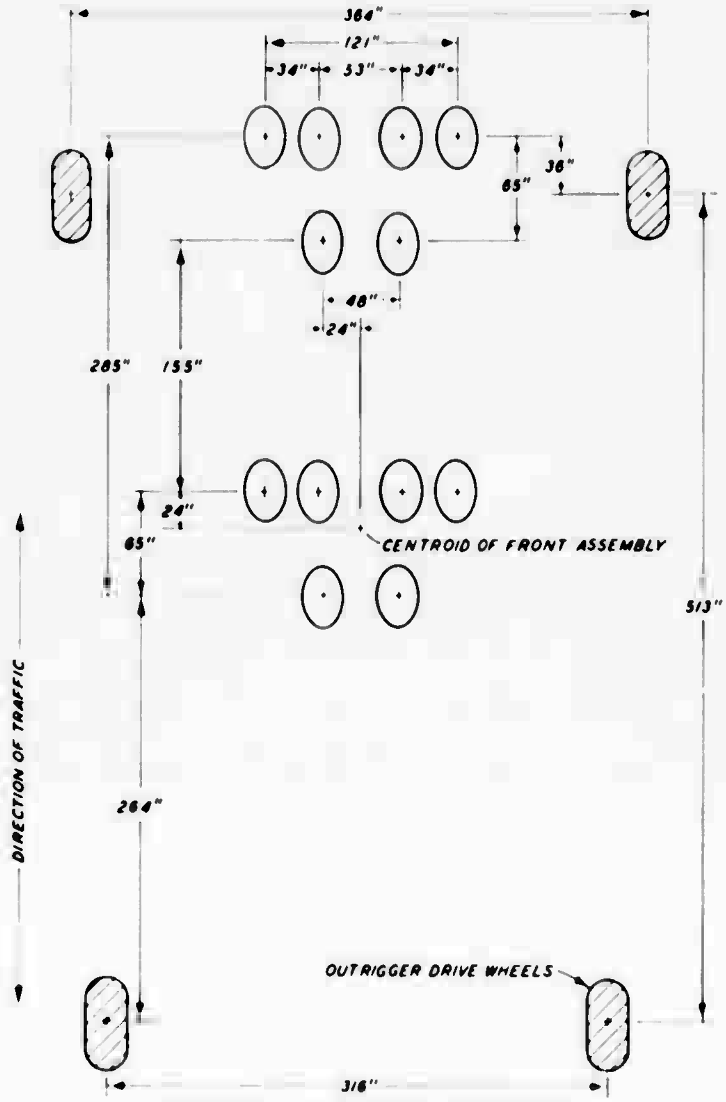
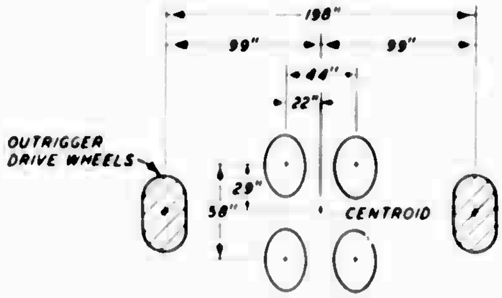

PLAN AND PROFILE
EVALUATION OF
STRUCTURAL LAYERS
IN FLEXIBLE PAVEMENT
TEST SECTION
LEGEND
① 360-KIP 12-WHEEL ASSEMBLY
② 180-KIP TWIN-TANDEM ASSEMBLY
③ 200-KIP TWIN-TANDEM ASSEMBLY
④ 50-KIP SINGLE-WHEEL ASSEMBLY
⑤ 75-KIP SINGLE-WHEEL ASSEMBLY
⑥ 360-KIP 12-WHEEL ASSEMBLY + 75-KIP SINGLE-WHEEL ASSEMBLY
N
180'
3+90
3+60
3+30
3+00
2+70
2+40
2+10
30'
WEST MANEUVER AREA
ITEM 1
ITEM 2
ITEM 3
ITEM 4
MWHGL ITEM 3
4'-10"
8'-2"
5'-0"
10'-0"
7'-8"
16'-8"
7'-8"
LANE 3
LANE 2
LANE 1
60'
PLAN
ASPHALTIC CONCRETE
CRUSHED STONE BASE
5 % CEMENT-STABILIZED CLAYEY GRAVELLY SAND (SP-SC) SUBBASE
LEAN CLAY (CL) CBR 6
3½ % LIME-STABILIZED LEAN CLAY (CL) SUBBASE
10 % CEMENT-STABILIZED LEAN CLAY (CL) SUBBASE
HEAVY CLAY (CH) CBR 4
6 % CEMENT-STABILIZED CLAYEY GRAVELLY SAND (SP-SC) BASE
CRUSHED STONE BASE
GRAVELLY SAND (SP) SUBBASE
THICKNESS, IN.
0
3
6
9
12
15
18
21
24
PROFILE

PLATE A2



a. 360-KIP 12-WHEEL ASSEMBLY
(ONE MAIN GEAR OF C-5A)



b. 160- OR 200-KIP TWIN TANDEM
(ONE TWIN-TANDEM COMPONENT
OF 747 ASSEMBLY)

WHEEL ARRANGEMENTS

APPENDIX B: BEHAVIOR OF PAVEMENT UNDER TRAFFIC

1. 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 deformation of the pavement under the assembly load for the lane being observed. After failure, a thorough investigation was made by excavating test trenches across the traffic lanes and by establishing profiles of the surface of the various layers in the structure, along with CBR measurements and other pertinent tests, to determine where failure had occurred. The data obtained during the traffic tests are presented in the following paragraphs. A layout of the test section is shown in plate B1.

360-kip 12-wheel Assembly (Lane 1)

Traffic tests

2. Item 1 (lime-stabilized lean clay subbase). A general view of item 1 prior to traffic is shown in photo B1. Initial cracking, which consisted of small hairline cracks approximately 2 ft long, was observed after 64 coverages. These cracks occurred at longitudinal paving-lane construction joints about 3 ft inside both edges of the traffic lane. During the remainder of traffic, these cracks tended to open and close as the test cart traversed the 200-in.-wide lane. After 99 coverages, the construction-joint cracks were about 20 ft in length and had a maximum width of 1/16 in. Intermittent hairline cracks located 1-1/2 ft either side of the center line of the traffic lane were also observed, throughout the length of the test item, at this time. After 198 coverages, this item was considered failed (photo B2) because of the severe cracking in the center 3 ft of the lane and along the construction joints (photo B3). The cracks at these locations went completely through the pavement and, in some instances, were about 1/2 in. wide. In addition to these cracks, alligator cracking was observed over the

entire item. After all failure data had been recorded, the area was overlaid with landing mat, and traffic was continued on items 2-4.

3. Item 2 (cement-stabilized lean clay subbase). An overall view of item 2 prior to traffic is shown in photo B4. Distress of the pavement was first noticed at 172 coverages of traffic. This distress was a continuation of the construction-joint cracking that originated in item 1. As traffic was continued, the cracks on either side of the 10-ft-wide paving lane became longer. After 201 coverages, they were approximately 1/32 in. wide and extended the length of the item. A close-up of this type of pavement cracking is shown in photo B5. At 268 coverages, hairline cracks were noticed at the transition of items 2 and 3. As traffic continued, these cracks rapidly became wider. After 783 coverages, an area between 2 to 3 ft either side of this transition had cracked completely through the pavement.

4. To prevent the cracks in the transition area between items 2 and 3 from migrating further into either item, an area 4 ft wide on either side of the transition was repaired. This maintenance consisted of removing the asphaltic concrete and then adding base course material until the surface of the base course was 1-1/2 in. below the asphaltic-concrete surface. This area was then surfaced with an aluminum landing mat. A neoprene membrane was used under the mat to prevent surface moisture from entering the base course.

5. New hairline cracks developed in the center 5 ft of the lane at about 790 coverages. After 1135 coverages, the cracks located approximately 2-1/2 ft south of the center line of the traffic lane were opening and closing as the test cart moved from one side of the lane to the other side. This item was considered failed at 1200 coverages due to cracks extending completely through the pavement. The most severe cracks were located 2-1/2 ft south of the center line and about 3 ft inside each edge of the paving lane (i.e., at the construction joints). There were also hairline longitudinal cracks and alligator cracking throughout the item, and 1/64-in.-wide cracks were evident about 2-1/2 ft north of the center line at failure. Although the item

was considered failed at 1200 coverages, traffic was continued to 1342 coverages (photo B6).

6. Item 3 (full-depth crushed limestone base). A view of item 3 prior to traffic is shown in photo B7. Pavement cracking was first detected at 274 coverages. This deterioration consisted of small hairline cracks approximately 3 ft long on either side of the paving lane (at the longitudinal construction joints). These cracks were located at the west end of the item and seemed to be a continuation of those observed in item 2. After 789 coverages, the cracks along the construction joints had increased to about 20 ft in length. No additional distress was observed at this time. The transition area between items 2 and 3 was repaired, as described in the preceding paragraph, to prevent the cracks in item 2 from migrating into item 3.

7. After repairing the transition area, traffic was continued. As traffic was applied, the construction-joint cracks increased in length, and, after 1349 coverages, the joint on either side of the 10-ft-wide paving lane had cracked the entire length of the item. Small intermittent hairline cracks were also observed throughout the item after 1349 coverages in the 100 percent coverage zone of the traffic lane. The item was rated in satisfactory condition at 1509 coverages. An overall view of this item at 1509 coverages is shown in photo B8.

8. A view of the small intermittent cracks existing in the 100 percent coverage zone at this time is shown in photo B9. Most of these cracks were longitudinal to the direction of traffic. As traffic continued, the random cracking throughout the item and the cracked construction joints became wider, which indicated an increase in crack depth. Therefore, after 2744 coverages, areas exhibiting the widest of these two types of cracks were cored to determine the depth of cracking. This investigation indicated that the cracks in the 100 percent coverage zone were approximately 1 in. in depth, and those at the construction joint were about 1-3/4 in. deep, as shown in photos B10 and B11, respectively. When the traffic coverages reached about 4500, the cracks began to open and close to a slight extent as the test cart traversed the test lane. Cores taken after 5000 coverages in the severely cracked areas,

i.e., the 100 percent coverage zone and construction joints, indicated that the cracks extended completely through the pavement. Therefore, this item 3 was considered failed at 5000 coverages. At failure, the cracks at the construction joints were long and continuous along both seams of the paving lane, and those in the 100 percent coverage zone were of the alligator type. Small alligator-type cracks were observed throughout the test lane at failure.

9. Item 4 (full-depth stabilized clayey gravelly sand base). Very little distress was observed on the pavement surface of item 4 during the 360-kip, 12-wheel-assembly traffic. A general view of this item prior to traffic is shown in photo B12. At about 1515 coverages, a very small hairline crack in the construction joint located about 3 ft south of the north edge of the traffic lane and three small cracks perpendicular to this crack were noted. The item was rated as in excellent condition at this time and was in the general condition shown in photo B13. After 5046 coverages, small hairline cracks were detected about two wheel-print widths from the north side of the traffic lane. Three 1/32- to 1/16-in.-wide cracks were perpendicular to these hairline cracks and extended to the edge of the traffic lane. Traffic was continued to 10,406 coverages, and the only change in the condition of the pavement was development of several small cracks extending out from the cracked construction joint, as shown in photo B14. Indications were that a large number of coverages would be required to produce failure; therefore, traffic was discontinued.

Pavement deflection

10. Pavement deflection measurements were made in the traffic lane at about the midpoint of each item of the test section prior to the start of traffic. The term "deflection" as used in this report indicates the total vertical movement that occurred under the static weight of the load wheels. The measurements were obtained with level instruments by reading rods (engineer scales) at prearranged positions on lines parallel and transverse to the direction of traffic. Rod readings were first taken with the load off the pavement, then the test cart was moved forward until the centroid of the front six wheels of the 12-wheel

assembly was at the desired prearranged position, and then a second series of readings was taken with the load wheels on the pavement. Readings were taken adjacent to and between the load wheels. The difference in rod readings with load on and load off indicated the vertical movement or total deflection of the pavement under load. Readings were also taken after the load was removed to determine what amount of the vertical movement was elastic deflection or rebound of the pavement.

11. Plots of the total deflection measurements taken prior to test traffic and at or near failure of each item are shown in plates B2 and B3. From these data, it can be seen that the total deflection in items 1-3 prior to traffic was about 0.25 in. and in item 4 about 0.21 in. The maximum total deflection measured at failure in the three failed items ranged from 0.30 to 0.41 in. The plots also indicate that the maximum deflection measured in item 4 was about 0.20 in., after 10,406 coverages of traffic, which was about the same or slightly less than the initial deflection. The readings taken after the load was removed indicated that 95 percent or more of the deflection measured was elastic. The deflections measured during the traffic period, tabulated in table 3 of the main text, showed that as traffic was applied deflection increased in items 1-3 and remained about the same in item 4. This determination indicates that a continued increase in deflection is a sign of a pavement structure approaching failure.

Permanent pavement deformation

12. Level readings were taken across the test lane at predetermined stations prior to traffic and at various intervals of traffic. These observations were made to determine the magnitude of pavement deformation resulting from traffic. Typical cross sections for the four test items are shown in plate B4. These data indicated that the maximum permanent deformation at failure in items 1 and 2 was approximately 1 in. and was about 2 in. in item 3. Maximum upheaval of the pavement occurred about 1 ft outside the edge of the traffic lane. After failure in items 1-3, the maximum upheaval measured was 0.36 in. After 10,406 coverages on item 4, no upheaval was apparent, and the maximum deformation detected was 0.84 in.

Failure investigation

13. After failure of an item, a trench or test pit was cut across the traffic lane to determine the extent of distortion of the various elements of the pavement structure. In-place CBR, water content, and density determinations were also made of the different elements as the trench was excavated. In addition, the thickness of each type of material and the total thickness of construction above the subgrade were measured.

14. It should be noted that the failure data measured in the test pits in items 1-3 were recorded after these items had been overlaid with landing mat and additional traffic applied. These items were overlaid with landing mat, and they were used as a maneuver area, while trafficking continued on the adjacent items. By following this procedure, time was not lost during the traffic period. However, as shown in tables 1 and 3 of the main text, the failure pits were excavated (depending on the respective item) after about 1200 to 5000 additional coverages had been applied on the landing-mat overlay and after a time lapse of 19 to 28 days after failure. Therefore, the failure data are not necessarily indicative of the strength of each item at failure due to the additional coverages and the time lapse between failure of an item and the taking of the failure data.

15. Pit profiles of the investigation trenches for items 1-3 are shown in plates B5-B7, respectively. The CBR test results, layer thicknesses, and other pertinent data for all test items are tabulated in table 1 of the main text. The as-constructed thickness measurements tabulated in table 1 (main text) were determined from level rod readings taken as each element of the pavement structure was constructed, and the after-traffic thicknesses were determined as each element of the pavement structure was removed in the various investigation pit locations.

16. Plots of the investigation trenches for items 1 and 2 (plates B5 and B6) indicate that there was very little lateral movement of the base, subbase, or subgrade during the traffic period. The data in table 1 (main text) indicate that the strength of the base course in items 1 and 2 was less after failure than it was after construction.

The strength of the base in item 1 decreased from 200 CBR to 44 CBR inside the traffic lane and to 85 CBR outside the traffic lane. The as-constructed strength of the base in item 2 was 180 CBR, as compared to 72 and 59 CBR after traffic, inside and outside the traffic lane, respectively. These data also indicate that the lime-stabilized lean clay increased from 38 to 84 CBR during traffic and that the cement-stabilized lean clay in item 2 increased from 80 to 121 CBR during traffic. The subgrades in items 1 and 2 were rated at 5.0 and 4.3 CBR, respectively.

17. The pit profile of item 3, shown in plate B7, indicated a slight amount of lateral movement of the crushed stone base material. The data summarized in table 1 (main text) indicated that the strength of the base course at the surface was about 132 CBR prior to traffic, as compared to about 86 CBR after traffic. The subgrade of this item was rated at 4.3 CBR.

18. Item 4 was in satisfactory condition when traffic was discontinued after 10,406 coverages; therefore, no investigation trenches were excavated at that coverage level.

160-kip Twin-tandem Assembly (Lane 2)

Traffic tests

19. Traffic with the 160-kip twin-tandem assembly was applied only in items 1 and 2, because it was estimated that a very large number of coverages at this load would be required to fail items 3 and 4. Item 3 was overlaid with landing mat and used as a maneuver area for the 160-kip load cart.

20. Item 1 (lime-stabilized lean clay subbase). A general view of item 1 prior to traffic is shown in photo B15. Cracks in the asphaltic concrete were first noted after 40 coverages. These were hairline to 1/32-in.-wide cracks approximately 1 ft in length located in the east half of the item in the 100 percent coverage zone of the traffic lane. Traffic was continued to 140 coverages, at which time the item was considered failed because the cracks extended through the asphaltic

concrete. These cracks were longitudinal to the direction of the traffic and were from 1/8 to 3/8 in. wide and about 12 ft long. Small hairline cracks were also detected throughout the item at this time. As can be seen in a general view of item 1 at failure (photo B16), most of the larger cracks occurred in that part of the item adjacent to item 2. A close-up of the cracks in the 100 percent coverage zone is shown in photo B17. After failure data were taken on the surface of the pavement, the traffic lane was overlaid with landing mat, and traffic was continued on item 2.

21. Item 2 (cement-stabilized lean clay subbase). A general view of item 2 prior to traffic is shown in photo B18. The first distress, alligator-type cracking located in the center 6 ft of the traffic lane and in that half of the item nearest item 1, was observed after about 600 coverages. The alligator cracks became more numerous and increased in width as traffic continued. After about 660 coverages, the alligator cracks in the west half of the item were approximately 1/8 in. wide, and small hairline cracks were observed in the center 6 ft of the lane in the east portion of the item. After 1000 coverages, this item was rated as failed due to alligator-type cracking, such as that shown in photo B19, occurring throughout most of the west half of the item (see photo B20). These cracks were about 1/4 in. wide and extended through the full thickness of the asphaltic pavement. Also at failure, hairline to 1/8-in.-wide alligator cracks were noted in the center 6 ft of the east half of the item, and a construction joint located about 1 ft from the north edge of the traffic lane was cracked for the length of the item.

Pavement deflection

22. The general procedure described in paragraph 10 of this appendix was followed to obtain deflection measurements in this lane. Plots of the total deflection taken prior to traffic and at failure are shown in plate B8. These data indicated that the total deflection in each respective item was approximately the same initially and at failure. The initial total deflection in items 1 and 2 was 0.21 and 0.23 in., respectively. At failure, the total deflection in item 1

increased to 0.42 in. and in item 2 to 0.45 in. Approximately 90 percent of the total deflection in item 1 and 76 percent of the total deflection in item 2 were elastic at 0 coverages. At failure, 83 percent of the total deflection in item 1 and 87 percent of the total deflection in item 2 were elastic.

Permanent pavement deformation

23. Level readings were taken, as described in paragraph 12, to determine the magnitude of settlement or upheaval developed in the traffic lane during traffic. Plots of typical cross-section measurements taken in items 1 and 2 are shown in plate B9. The data in this plate indicated that the maximum permanent deformation occurring at failure in items 1 and 2 was 0.96 and 1.80 in., respectively. The maximum upheaval, which was measured approximately 1 ft outside the traffic lane, was 0.24 in. in both items.

Failure investigations

24. Trenches approximately 2 ft wide and 15 ft long were excavated to the subgrade in items 1 and 2. Each trench extended from about the center line of lane 2, north to the center line of lane 3 (see photo B21).

25. These investigation trenches were excavated under the same general conditions as explained in paragraph 13 of this appendix. Prior to excavating, each item had been overlaid with landing mat and then used as a maneuver area for the adjacent unfailed item. The trench in item 1 was excavated 36 days after failure, and 41 days elapsed after failure before the trench was made in item 2. Level rod readings were taken as each layer of the system was removed to give profiles of the different layers shown in plates B10 and B11. CBR, density, and water content determinations were obtained for each type of material as the trenches were dug, and the results are summarized in table 1 of the main text. The profiles (plates B10 and B11) indicated a slight deformation in the asphaltic concrete and crushed stone base in item 1 and a very pronounced deformation of the asphaltic concrete and crushed stone base in item 2. There was also a slight amount of deformation of the cement-stabilized lean clay inside the twin-tandem assembly lane. The data,

summarized in table 1 (main text), showed a loss of strength in the crushed stone base of both items after traffic. In item 1, the base course material decreased in strength from about 200 CBR to 30 and 58 CBR inside and outside the traffic lane, respectively. The strength of the base course measured in item 2 after construction was 180 CBR, and, after traffic, the strength had decreased to 78 CBR inside the traffic lane and to 58 CBR outside the traffic lane. CBR measurements taken on the lime-stabilized lean clay indicated an increase in strength after traffic. A strength of 38 CBR was measured on this subbase material after construction, and, after traffic, the strength inside the traffic lane was 68 CBR and outside the lane was 118 CBR. These data also indicated that the cement-stabilized lean clay in item 2 was stronger after traffic than when it was constructed. A strength of 80 CBR was measured after construction, and 200 and 253 CBR were measured inside and outside the traffic lane, respectively.

200-kip Twin-tandem Assembly (Lane 2)

Traffic tests

26. Item 3 (full-depth crushed limestone base). A general view of item 3 prior to traffic is shown in photo B22. Before the trafficking of items 1 and 2 with the 160-kip twin-tandem assembly, item 3 was overlaid with landing mat to be used as a maneuver area. After the mat was removed and prior to tracking item 3, measurements were made that showed 1.68-in. maximum permanent deformation had occurred in the traffic lane while the item was used as a maneuver area (photo B23). The first distress, which consisted of a small crack about 15 in. long near the center of the traffic lane, was noticed after 180 coverages. After 200 coverages, hairline cracks in the center 4 ft of the traffic lane and a hairline crack at a construction joint about 1 ft inside the north edge of the traffic lane had developed. The cracks located in the 100 percent coverage zone and at the construction joint had increased in width to about 1/64 in. after about 500 coverages. The cracks located in the 100 percent coverage zone were parallel to the direction of

traffic. At 890 coverages, these cracks extended through the asphaltic-concrete layer, and this item was evaluated as failed. A general view of item 3 at failure is shown in photo B24. The cracks in the foreground of this photograph were not considered in the evaluation of this item, because they originated from the cracks in item 2 and were considered to be in a transition zone between the two items. A close-up of the cracked pavement evaluated as failed in the 100 percent coverage zone is shown in photo B25. In addition to these cracks, small alligator-type cracks had developed throughout the entire item at failure.

27. Item 4 (full-depth stabilized clayey gravelly sand base). Distress of the pavement in item 4 was first observed at about 500 coverages. This distress consisted of a few small hairline cracks, transverse to the direction of traffic, located in the 100 percent coverage zone of the traffic lane. After 560 coverages, small cracks running transverse to traffic, such as those shown in photo B26, were noticed throughout the 100 percent coverage zone of the item. Very little additional distress was observed until about 1300 coverages. At this time, small cracks, like those shown in photo B26, were located throughout the item. Several of these cracks in the center of the traffic lane were about 1/16 in. wide. After 1810 coverages, the cracks in the 100 percent coverage zone had opened up to a width of about 1/8 in., and core samples cut at this time revealed that the pavement was cracked completely through; therefore, this item was rated as failed at 1810 coverages. The general condition of this item at failure is shown in photo B27. Also, at failure, a construction joint about 1 ft south of the north edge of the traffic lane was cracked the length of the item, and small intermittent cracks were located throughout the lane.

Permanent pavement deformation

28. Level readings were taken at prearranged positions to determine the amount of settlement or upheaval developed during traffic. Plots of the cross-section measurements taken for items 3 and 4 are shown in plate B12. The as-constructed and 0-coverage data are plotted for item 3 to show the deformation (maximum 1.68 in.) that occurred when

this item was used as a maneuver area during the trafficking of items 1 and 2 with the 160-kip twin-tandem assembly. The deformation and upheaval data, summarized in table 3 (main text), for item 3 were computed from the 0-coverage measurements. After failure of item 3, the maximum permanent deformation and upheaval measured were 2.52 and 0.36 in., respectively. The maximum deformation in item 4 was 1.32 in. at failure, and an upheaval of about 0.24 in. was measured about 2 ft outside the traffic lane.

Failure investigations

29. A 2-ft-wide trench, such as that described in paragraph 24 of this appendix, was excavated to the subgrade in item 3. The profile of the different layers in the test pit is shown in plate B13. These data indicated a deformation of about 1.92 in. at the center line of the traffic lane and an upheaval of 0.72 in. located 2 ft outside the traffic lane. This deformation was due to consolidation and lateral movement of the base material, which resulted in surface upheaval. A summary of the soil data taken as this pit was excavated is included in table 1 of the main text. These data indicated that the average strength of the base course material decreased during traffic from 132 to 63 CBR, and the strength of the subgrade remained about the same. After failure, the percent of CE 55 density for the base course material was 106 inside the traffic lane and 99 outside the traffic lane, as compared to 100 percent for the as-constructed values. The heavy clay subgrade of this item was rated at 4.3 CBR.

50-kip Single-wheel Assembly (Lane 3)

Traffic tests

30. Single-wheel traffic was applied on each item of the test section in lane 3: a 50-kip load was applied in items 1 and 2, and a 75-kip load was used on items 3 and 4. To prevent deformation from occurring in item 3 while tracking items 1 and 2, as was the case in the twin-tandem-assembly lane, the load cart was stopped on the backward

pass in item 2 prior to the test wheel reaching item 3; then the test cart continued forward.

31. Item 1 (lime-stabilized lean clay subbase). A view of item 1 prior to traffic is shown in photo B28. It was apparent during the initial application of traffic that this test item would withstand very little traffic. As the test vehicle made the first pass, the pavement seemed to form a bow wave in front of the test wheel, and small cracks appeared in the pavement alongside the test wheel. After 40 coverages, the item was in the condition shown in photo B29 and was rated as failed. At failure, there was a crack in the center of the traffic lane, which extended completely through the pavement and was about 3/8 in. wide. Also at failure, there were cracks from 1/16 to 1/8 in. wide parallel to traffic in four other locations and hairline cracks throughout the 80 and 100 percent coverage zones of the traffic lane. After failure, the item was overlaid with landing mat, and traffic was continued on item 2 using the overlaid item 1 as a maneuver area.

32. Item 2 (cement-stabilized lean clay subbase). A general view of item 2 before traffic is depicted in photo B30. The first pavement distress, which was observed at 50 coverages, consisted of a 10-ft-long hairline crack in a paving-lane construction joint located in the center of the traffic lane. After 90 coverages, this crack was 1/4 to 3/8 in. wide, and it extended completely through the asphaltic-concrete layer for the length of the item (see photo B31). Since this crack extended through the pavement, the item was considered failed, but traffic was continued until failure occurred at some location other than a construction joint. After an additional 30 coverages, two other cracks in the 100 percent coverage zone and one crack near the north edge of the traffic lane were noticed that were from 1/4 to 3/8 in. wide and extended through the pavement. A general view of the item at 120 coverages is shown in photo B32. Photo B33 is a close-up of the severely cracked pavement in the 100 percent coverage zone after 120 coverages.

Pavement deflection

33. Total deflections of the pavement surface prior to and after

traffic are shown in plate B14. The maximum initial total deflection in item 1 was about 0.13 in.; 80 percent of this was elastic. The maximum total deflection measured in item 2 at 0 coverages was 0.15 in. After removing the load assembly from item 2 at 0 coverages, the pavement surface rebounded to its original elevation; therefore, the total and elastic deflections were the same in item 2 at 0 coverages. The greatest total deflection in item 1, 0.53 in., and in item 2, 0.69 in., occurred at failure. Approximately 83 percent of the total deflection measured at failure in items 1 and 2 was elastic.

Permanent pavement deformation

34. Permanent deformation measurements, as determined from level readings taken prior to, during, and at the end of traffic, are shown in plate B15. The greatest deformation, about 0.60 in., and upheaval, 0.48 in., occurred in item 1 at failure. After failure of item 2, the maximum deformation measured was 1.44 in., and the maximum upheaval was 0.96 in. The greatest upheaval occurred at the edge of the traffic lane in both of these items.

Failure investigations

35. Test trenches were excavated in items 1 and 2 of this lane from about the center line of the traffic lane south to the center line of the twin-tandem traffic lane. Prior to excavating these trenches, each item was overlaid with mat and used as a maneuver area for traffic on the unfailed adjacent item. The profiles of each layer of these items after traffic are shown in plates B10 and B11. These data indicated deformation in the base and subbase course of each item. A slight upheaval of the pavement surface between the two lanes was detected from these data.

36. A summary of the after-traffic water content, density, CBR, and thickness data obtained from the test pit investigations is shown in table 1 (main text). The data for item 1 were obtained 12 days after failure and for item 2, 18 days after failure. These data indicated that, during traffic, the strength of the base courses for both items decreased and the strength of the stabilized lean clay subbases increased. The CBR of the crushed stone base in item 1 decreased from

200 to 53; the CBR of the lime-stabilized lean clay increased from 38 to 53. In item 2, CBR's of 180 and 42 were measured for the crushed stone base prior to and after traffic, respectively. The CBR of the cement-stabilized lean clay subbase increased from 80 to 109. These data were obtained from 12 to 18 days after failure and after additional traffic had been applied on the landing mat overlying each item; therefore, as discussed in paragraph 25 of this appendix, these data do not necessarily represent the characteristics of each element at failure.

75-kip Single-wheel Assembly (Lane 3)

Traffic tests

37. Traffic commenced on items 3 and 4 with the 75-kip single-wheel assembly, after item 2 was overlaid with landing mat.

38. Item 3 (full-depth crushed limestone base). After 10 coverages, small hairline cracks parallel to the direction of traffic were observed in the 100 percent coverage zone of item 3. After 50 coverages, item 3 was rated as failed and was in the condition shown in photo B34. At failure, there were 1/4- to 3/8-in.-wide cracks extending through the asphaltic concrete in the 100 and 80 percent coverage zones, and several 1/8-in.-wide cracks were observed in the 20 percent coverage zone. Photo B35 shows a close-up view of the severely cracked pavement in the 100 percent coverage zone at failure.

39. Item 4 (full-depth stabilized clayey gravelly sand base). Pavement distress in item 4 was first observed at 50 coverages. This distress consisted of small hairline cracks located in the center of the traffic lane perpendicular to the direction of traffic. After about 100 coverages, cracks 1/32 to 1/16 in. wide and from 3 to 4 ft long were detected in the center of the traffic lane. All of these cracks were transverse to the direction of traffic. As traffic was continued, the pavement began cracking in a longitudinal direction, which resulted in an alligator-type cracking pattern when these cracks connected with the transverse cracks. After 120 coverages, several cracks were 1/8 in. wide and extended through the asphaltic-concrete layer; therefore, this

item was evaluated as failed. A general view of this item at failure is shown in photo B36, and a close-up of the cracking pattern is shown in photo B37. Also, at failure, there were small intermittent cracks throughout the lane.

40. After item 4, lane 3, failed, the center 98 in. of item 4, lane 1, was trafficked with the 75-kip single-wheel assembly. It was decided to apply the single-wheel traffic in the unfailed 12-wheel-assembly lane of item 4 to determine the performance of the pavement structure under mixed-traffic conditions. A general view of this item after 10,406 coverages of the 360-kip 12-wheel-assembly traffic is shown in photo B38. The general condition of this item prior to the single-wheel-assembly traffic is discussed in paragraph 9 of this appendix. After 60 coverages of the single-wheel traffic, a small hairline crack was observed in a construction joint located about 1 ft inside the north edge of the traffic lane. Hairline alligator-type cracking appeared in the 100 percent coverage zone of the traffic lane at approximately 170 coverages. The crack in the construction joint had opened up to about 1/32 in. at this time. After 200 coverages, the cracks in the center portion of the lane extended through the pavement, and the item was rated as failed. Several of the cracks in the 100 percent coverage zone were 1/2 to 1 in. wide. A close-up view of the most severe cracks at failure is shown in photo B39, and a general view of the item at failure is shown in photo B40. Other distress of the asphaltic concrete noted at failure included hairline to 1/8-in.-wide alligator cracks throughout the item and a 1/8-in.-wide crack running down the construction joint located inside the north edge of the traffic lane.

<u>Pavement deflection</u>

41. Total deflection measurements taken prior to and after traffic are shown in plate B14. In lane 3, the maximum deflection measured in items 3 and 4 prior to traffic was 0.39 and 0.18 in., respectively; in lane 1, 0.26 in. was the maximum deflection measured in item 4 prior to the 75-kip single-wheel traffic. After failure, the maximum deflection measurements in lane 3 were 0.74 in. in item 3 and 0.65 in. in item 4; 0.55-in. maximum deflection was measured in the portion of the

12-wheel-assembly lane of item 4 trafficked with the single-wheel assembly.

Permanent pavement deformation

42. Cross sections, taken to determine the amount of settlement or upheaval developed during single-wheel traffic, are shown in plates B15 and B16. In lane 3, the greatest deformation, about 2.88 in., occurred in item 3 at failure. The maximum deformation measured in item 4 was 1.44 in. After failure, the greatest upheaval measured in item 3 was 0.48 in. and in item 4 was 0.60 in.

Failure investigation

43. Test pits were excavated inside the lane in item 3 and outside the lane in item 4. A summary of the soil data taken as these pits were excavated is included in table 1 of the main text. The data indicated that the strength of the crushed stone base course in item 3 decreased during traffic from 132 to 94 CBR and that the field density was 104 and 99 percent of the laboratory density inside and outside the traffic lane, respectively, as compared to 101 percent of the laboratory density after construction. The cement-stabilized clayey gravelly sand base in item 4 had an initial strength of 202 CBR, and, after traffic, 159 and 261 CBR were measured inside and outside the traffic lane, respectively. There was a time lapse of 16 days between failure and excavating the test pit in item 4.

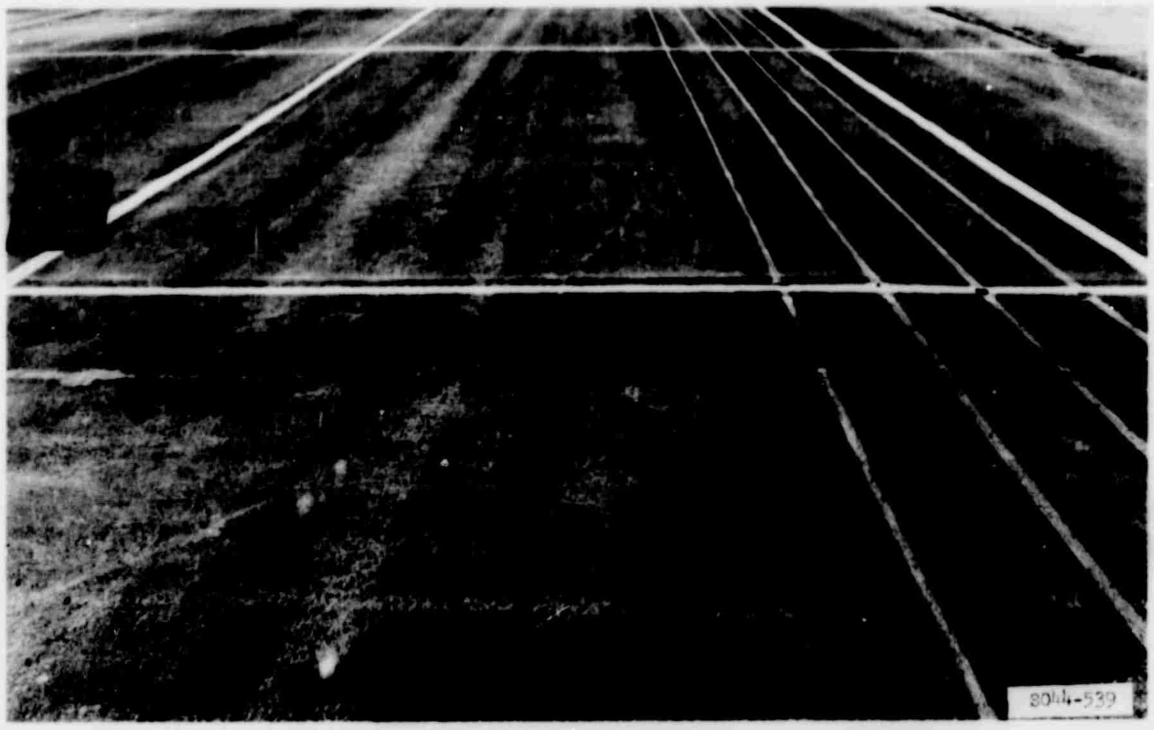


Photo B1. Item 1, lane 1, prior to traffic

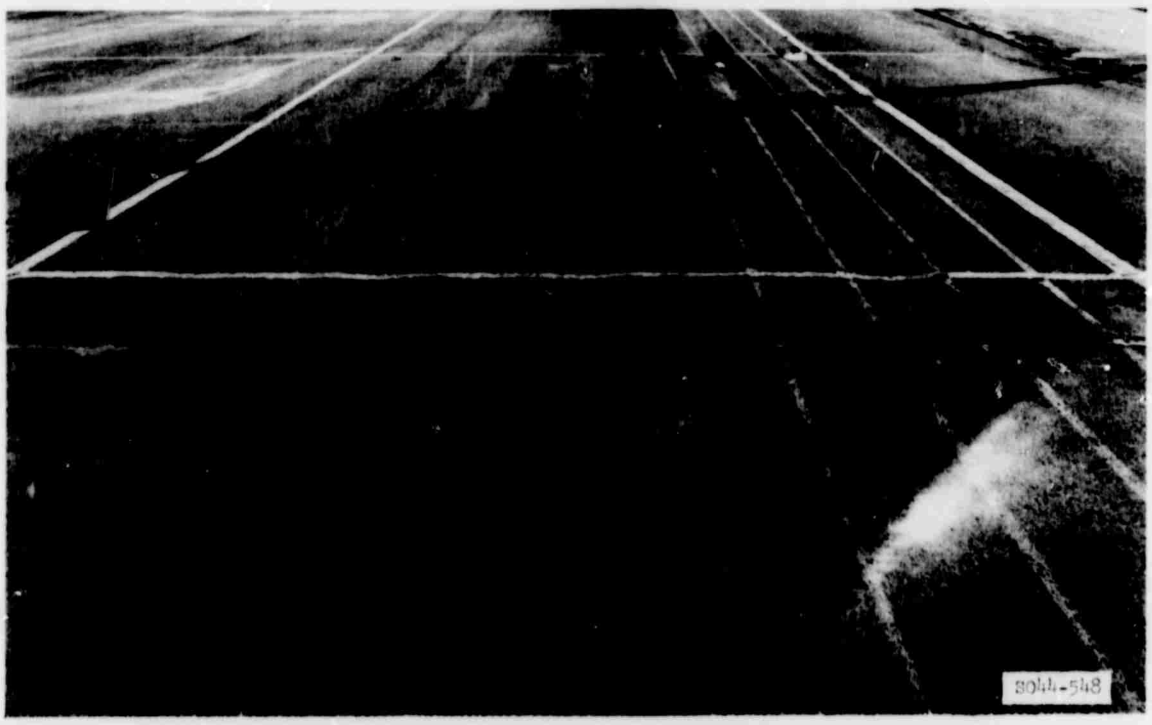


Photo B2. General view of item 1, lane 1, at failure (after 198 coverages of 360-kip, 12-wheel assembly)

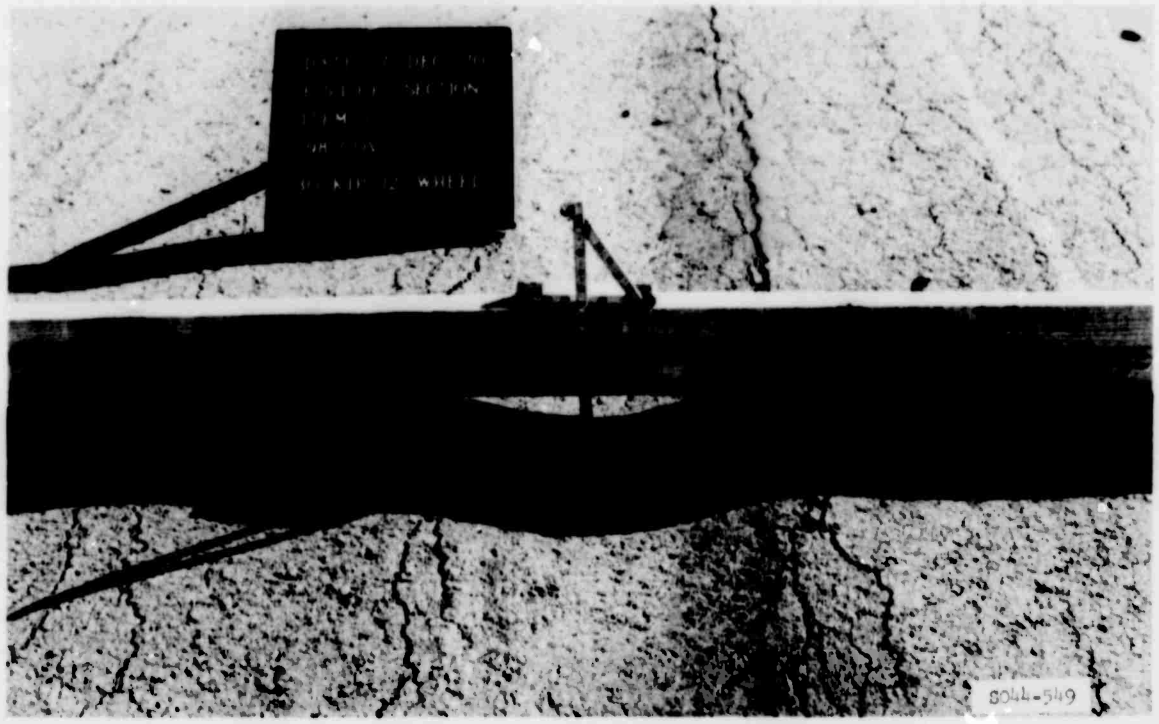


Photo B3. View of the severe cracking in item 1, lane 1, at failure

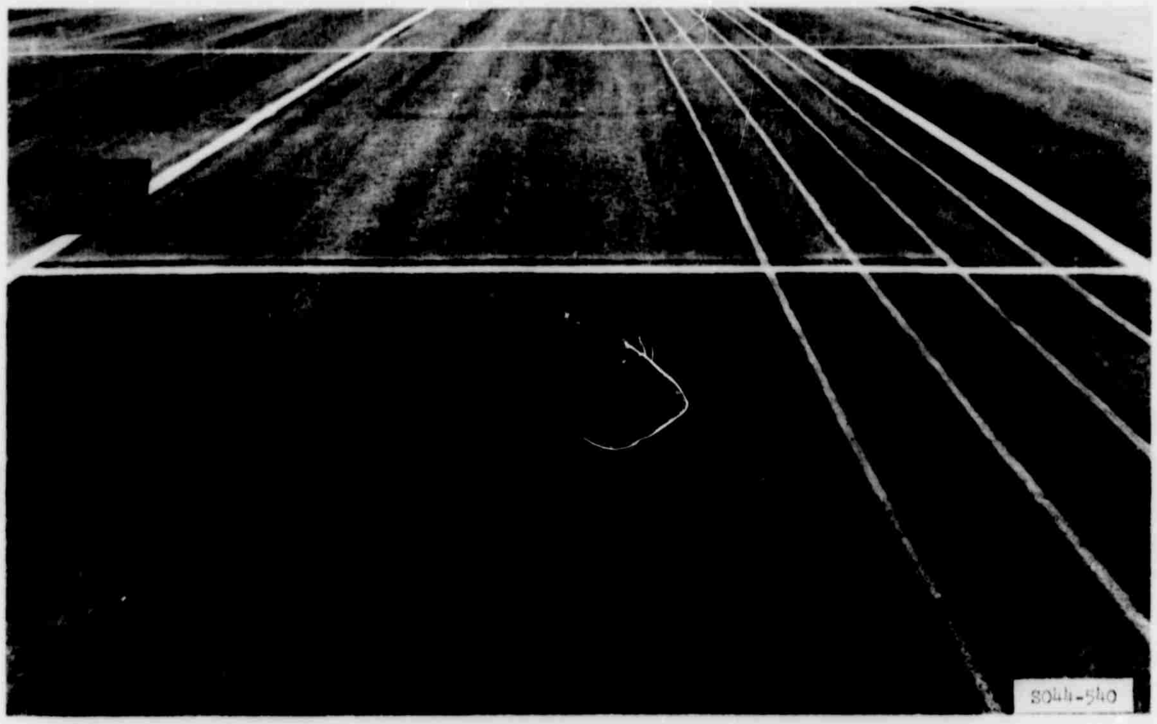


Photo B4. Item 2, lane 1, prior to traffic

Photo B5. Construction-joint crack in item 2, lane 1 (after 201 coverages of 360-kip, 12-wheel assembly)

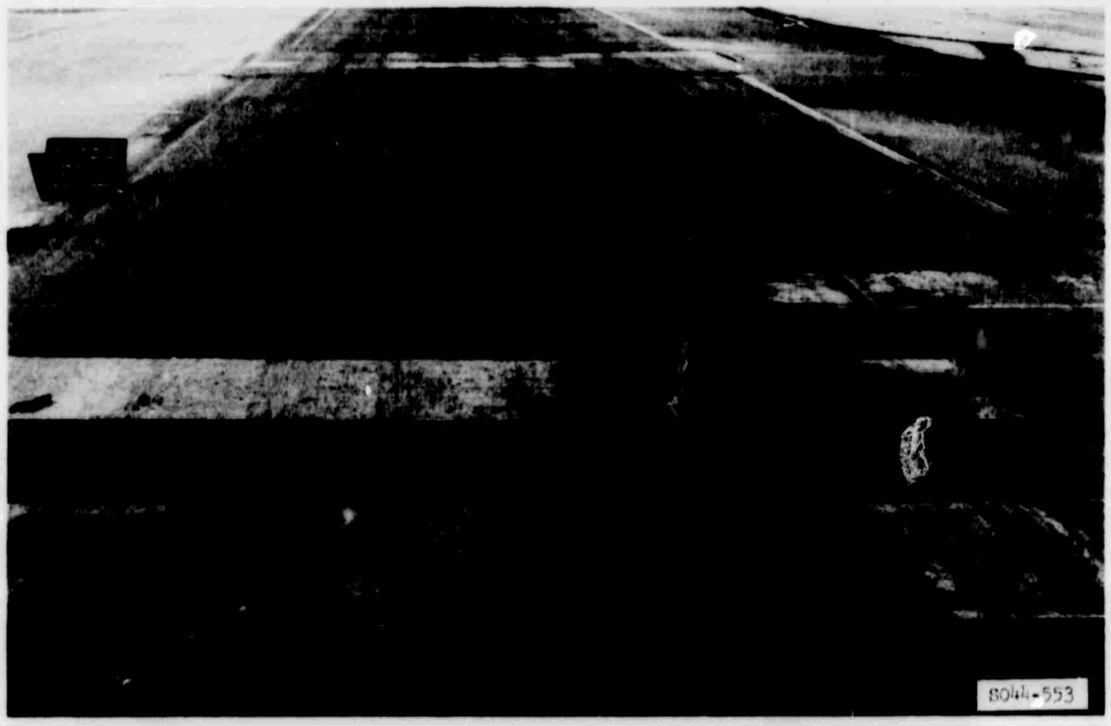


Photo B6. General view of item 2, lane 2, after 1327 coverages of 360-kip, 12-wheel assembly (failed at 1200 coverages)

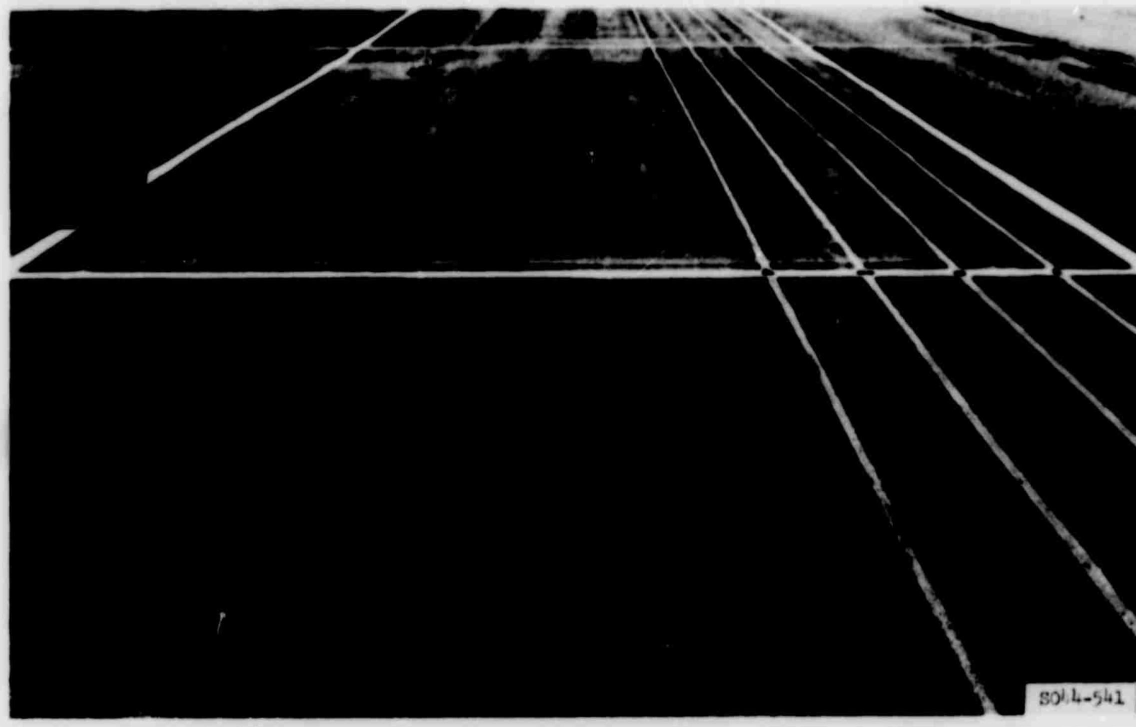


Photo B7. Item 3, lane 1, prior to traffic

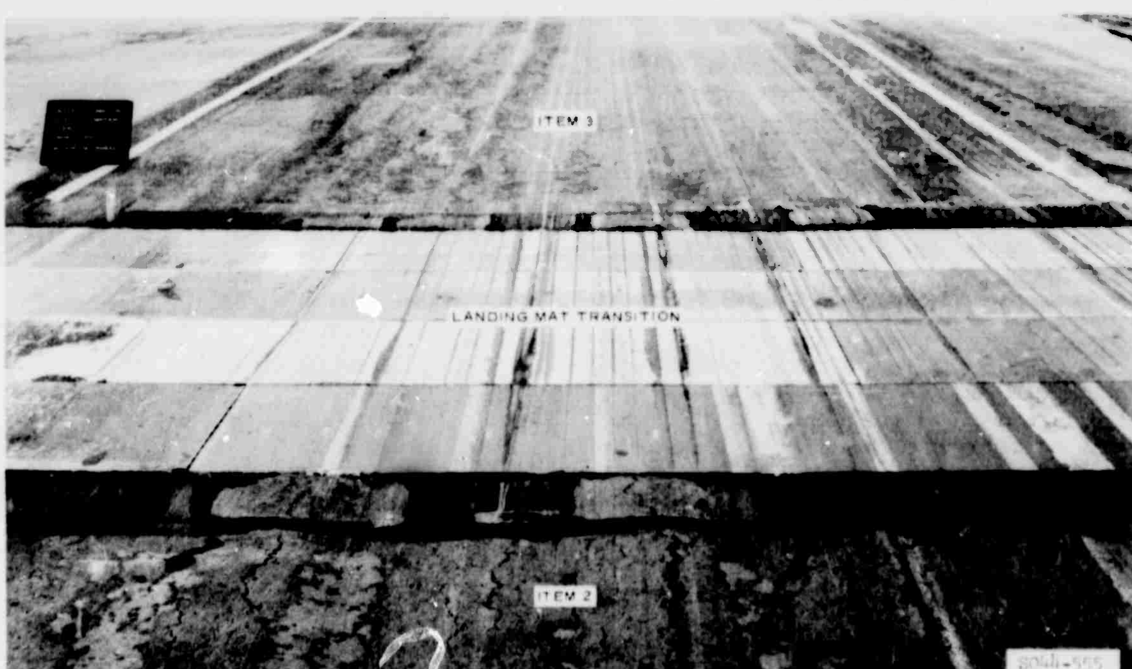


Photo B8. General view of item 3, lane 1, after 1509 coverages of 360-kip, 12-wheel assembly. Note cracks in item 2 in foreground

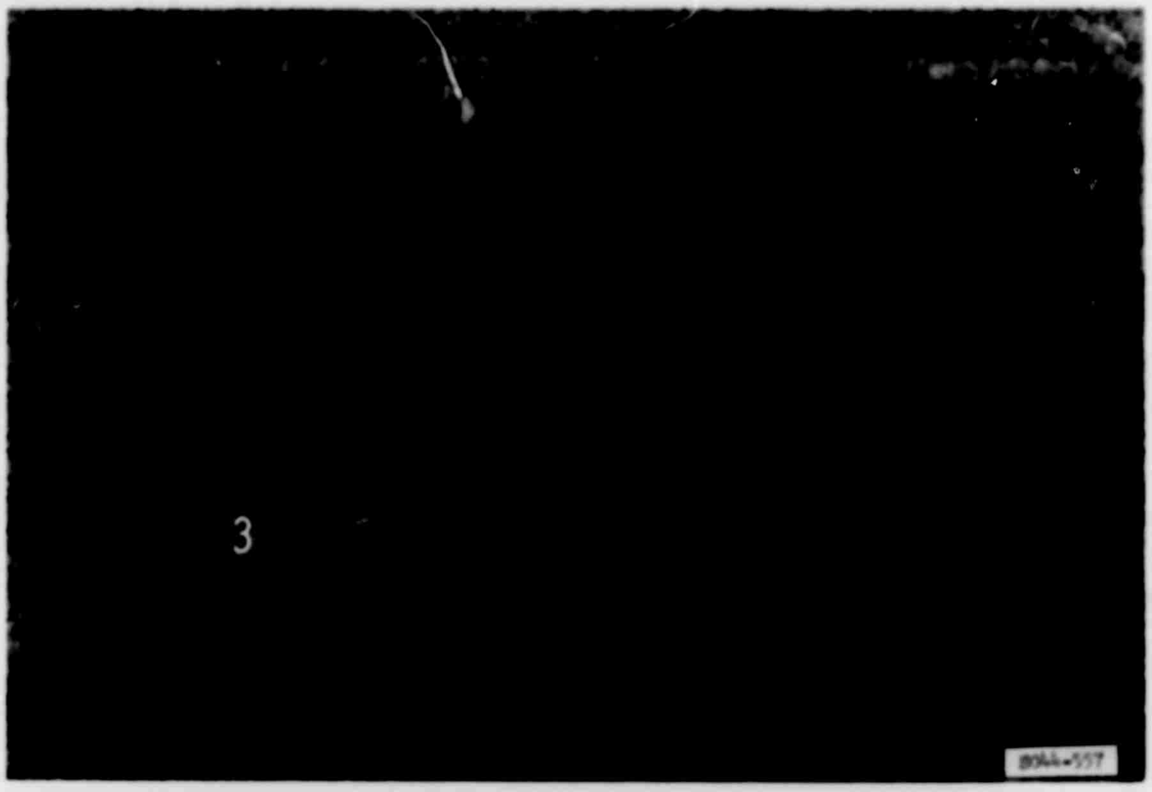


Photo B9. Small intermittent cracks in 100 percent coverage zone of item 3, lane 1, at 1509 coverages of 360-kip, 12-wheel assembly

Photo B10. One-in.-deep crack in core from 100 percent coverage zone of item 3, lane 1 (after 2744 coverages of 360-kip, 12-wheel assembly)

Photo B11. Core taken at construction joint in item 3, lane 1 (after 2744 coverages of 360-kip, 12-wheel assembly). Crack depth about 1-3/4 in.

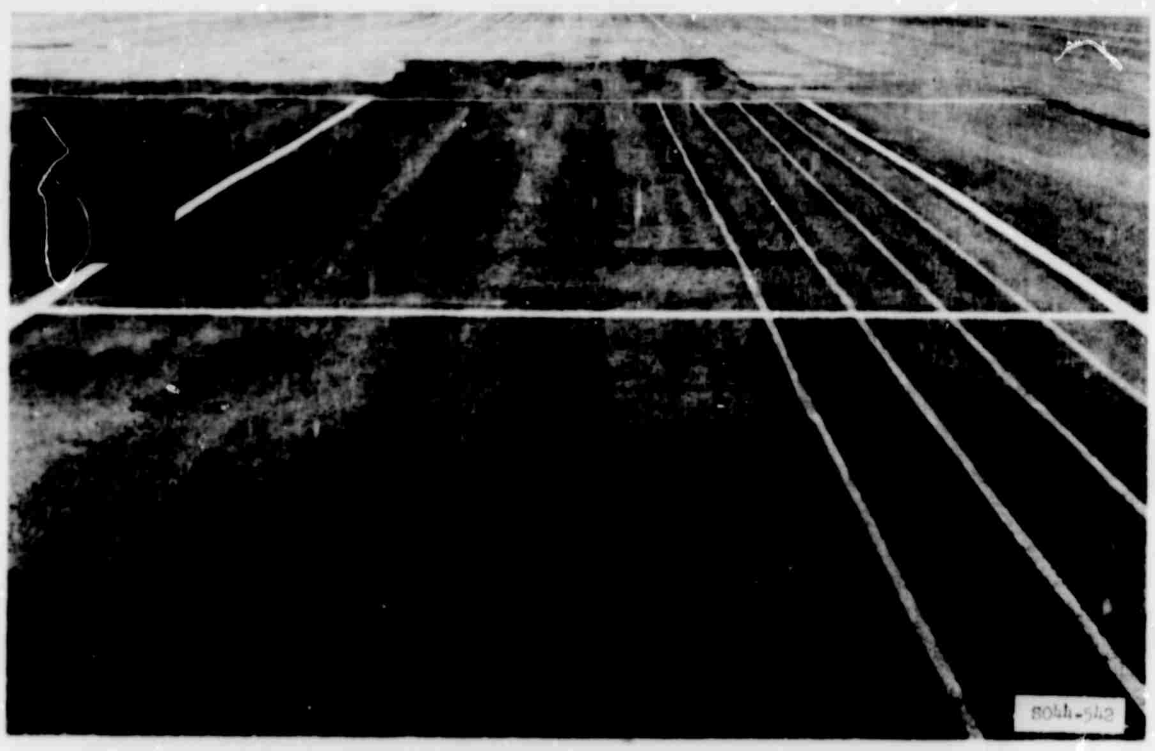


Photo B12. Item 4, lane 1, prior to traffic

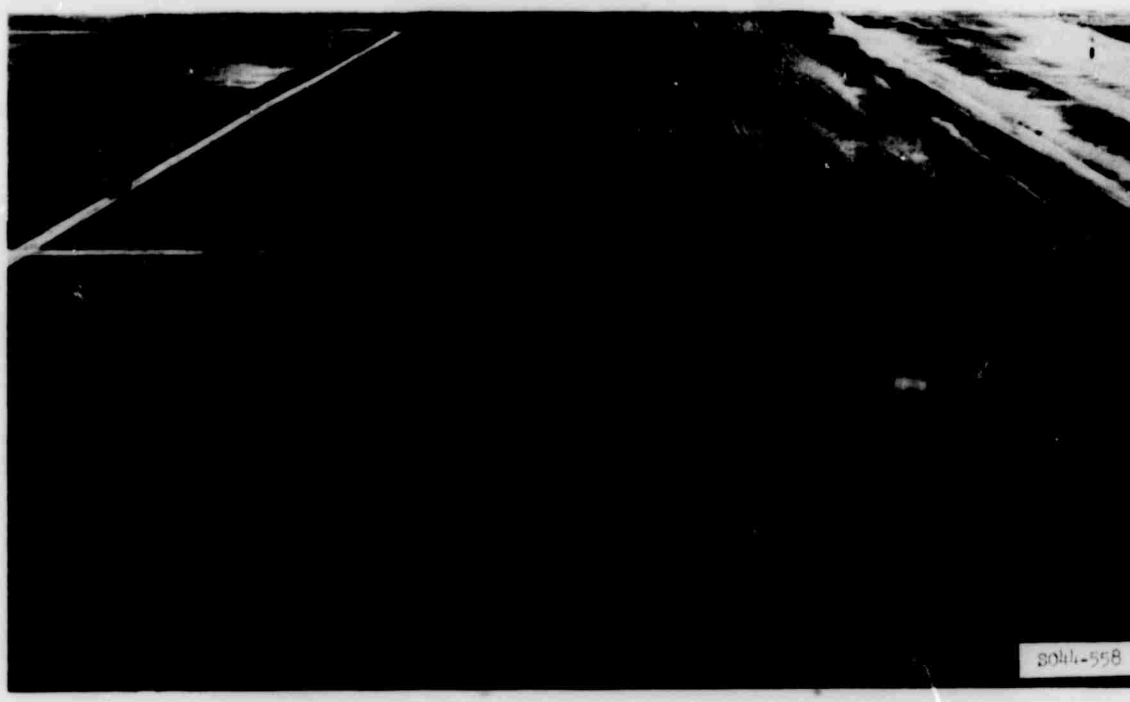


Photo B13. General view of item 4, lane 1, after 1515 coverages of 360-kip, 12-wheel assembly

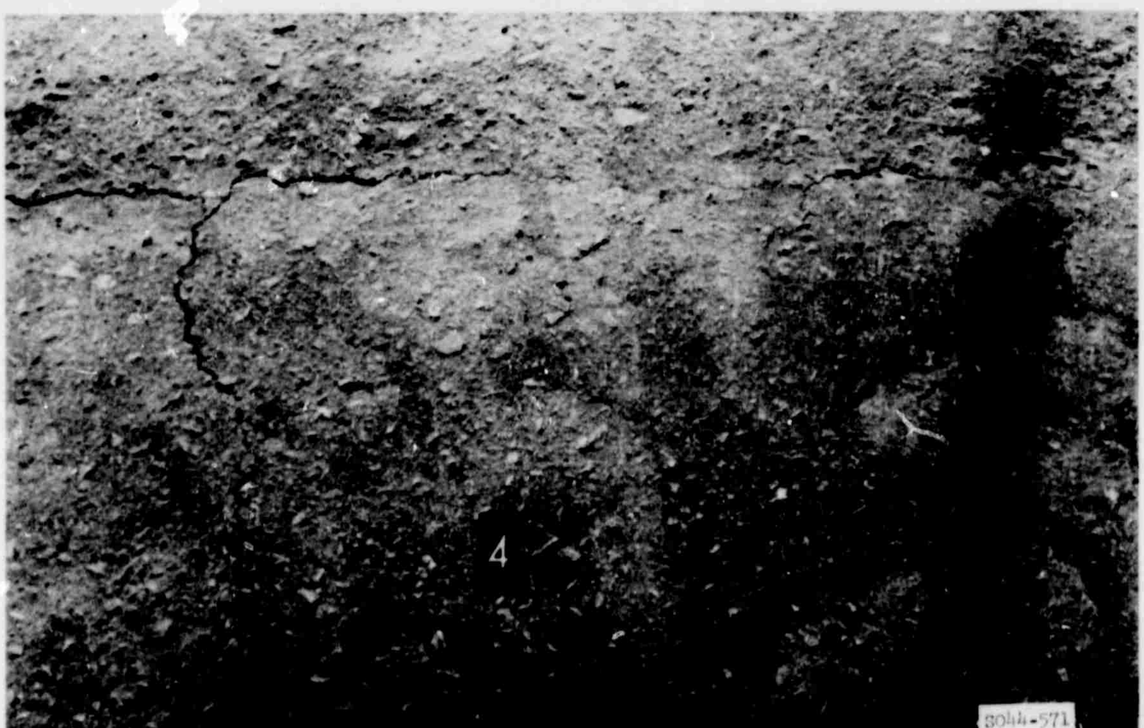


Photo B14. Small cracks extending from construction-joint crack in item 4, lane 1, after 10,406 coverages of 360-kip, 12-wheel assembly

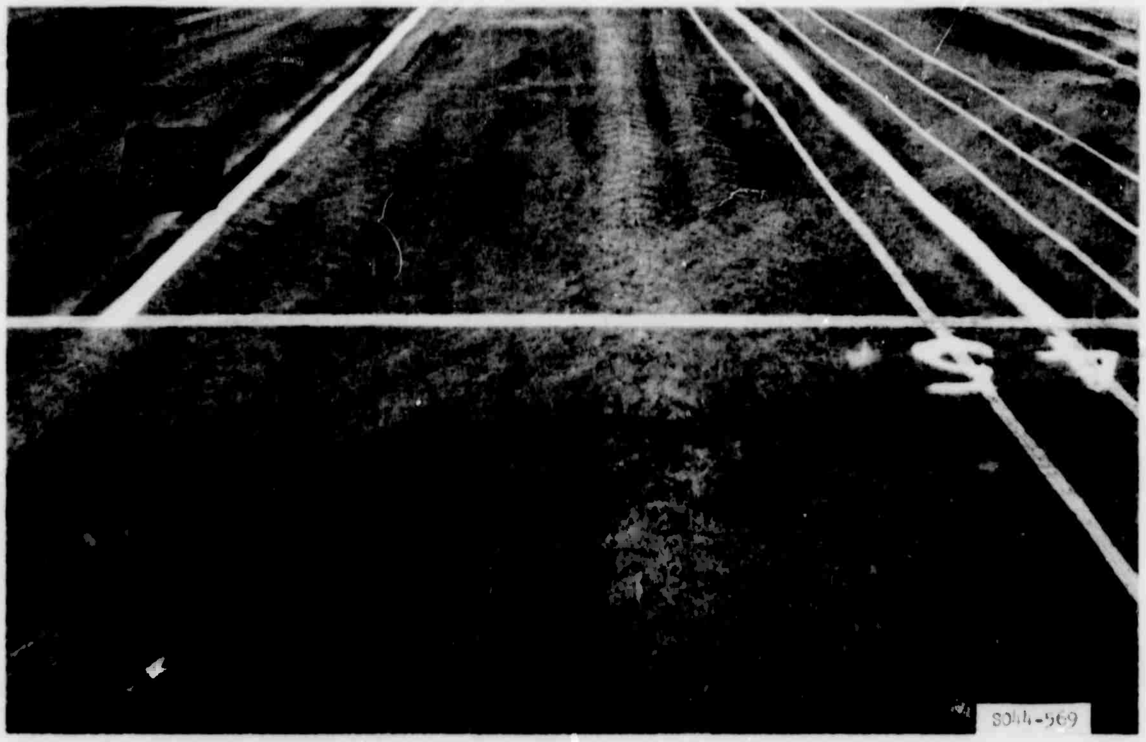


Photo B15. Item 1, lane 2, prior to traffic

Photo B16. General view of item 1, lane 2, at failure (after 140 coverages of 160-kip, twin-tandem assembly)

Photo B17. Close-up of cracks in 100 percent coverage zone in item 1, lane 2, at failure (after 140 coverages of 160-kip, twin-tandem assembly)

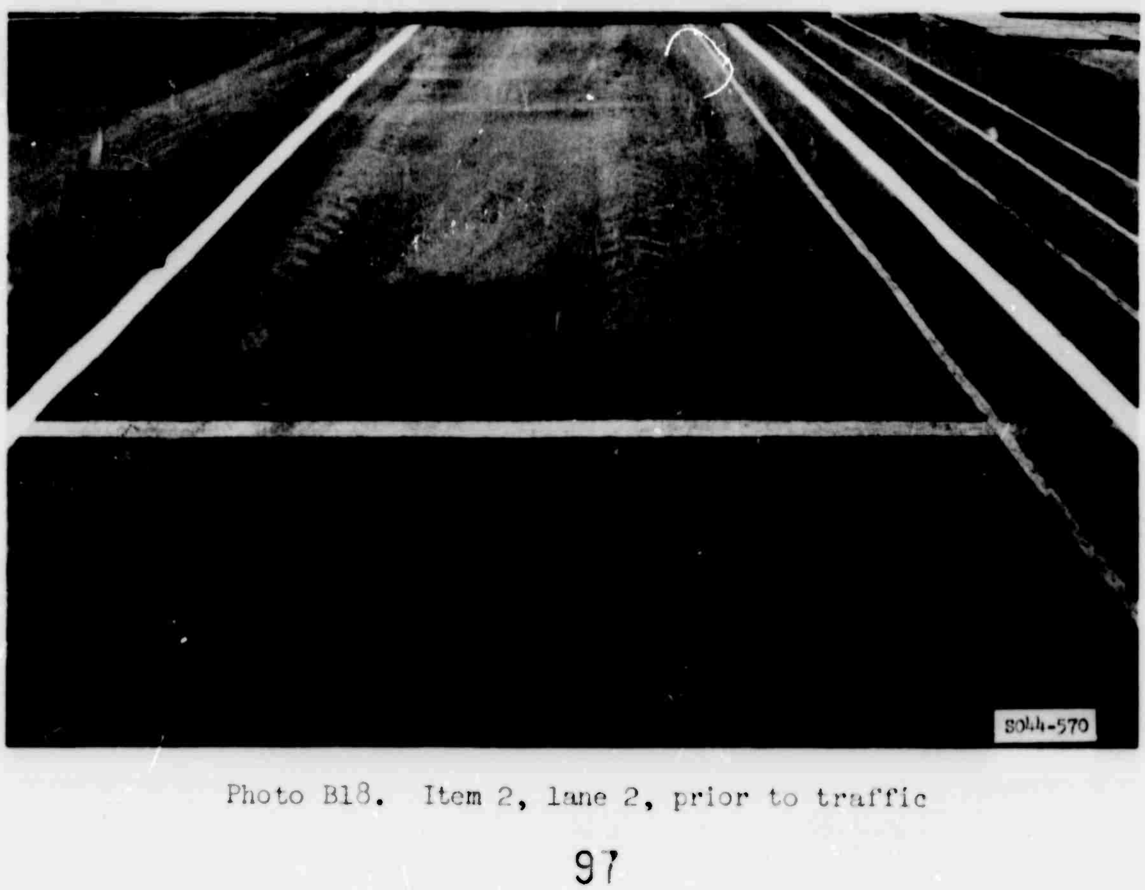


Photo B18. Item 2, lane 2, prior to traffic

Photo B19. 1/4-in.-wide alligator-type cracking in 100 percent coverage zone of item 2, lane 2, at failure (after 1000 coverages of 160-kip, twin-tandem assembly)

Photo B20. General view of item 2, lane 2, at failure (after 1000 coverages of 160-kip, twin-tandem assembly)

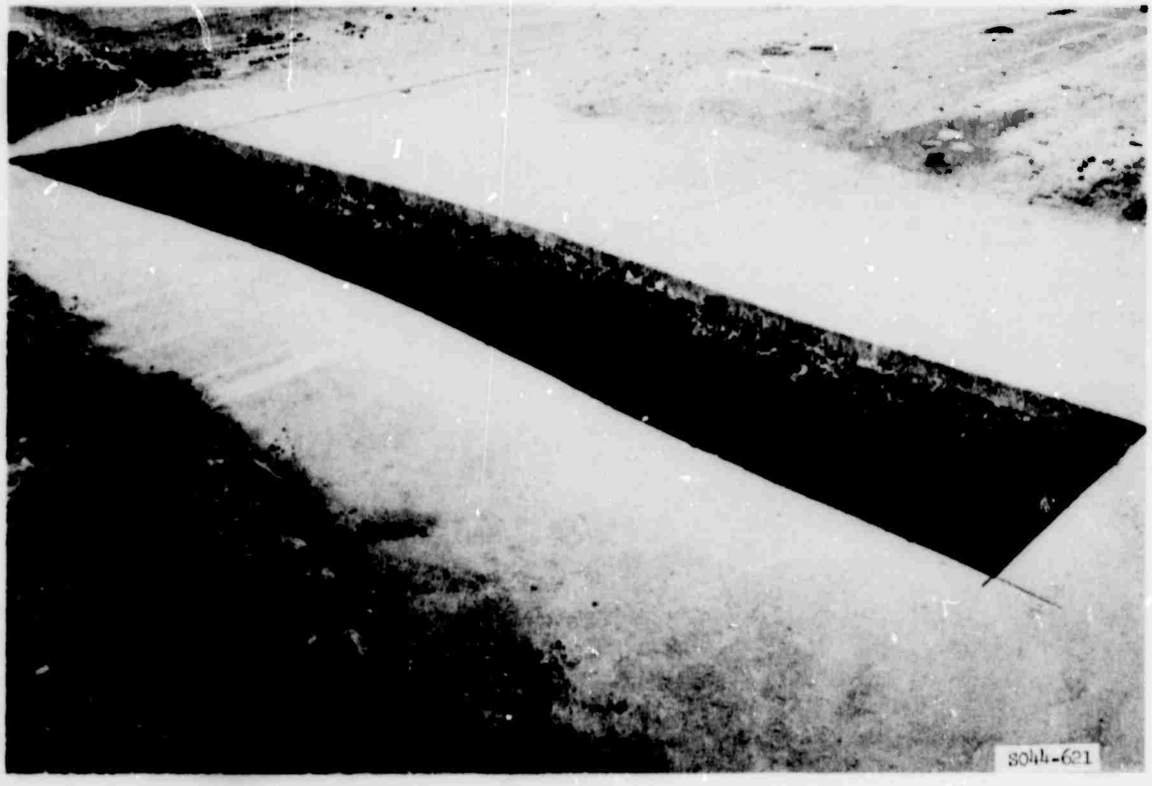


Photo B21. Test trench across item 1, lanes 2 and 3

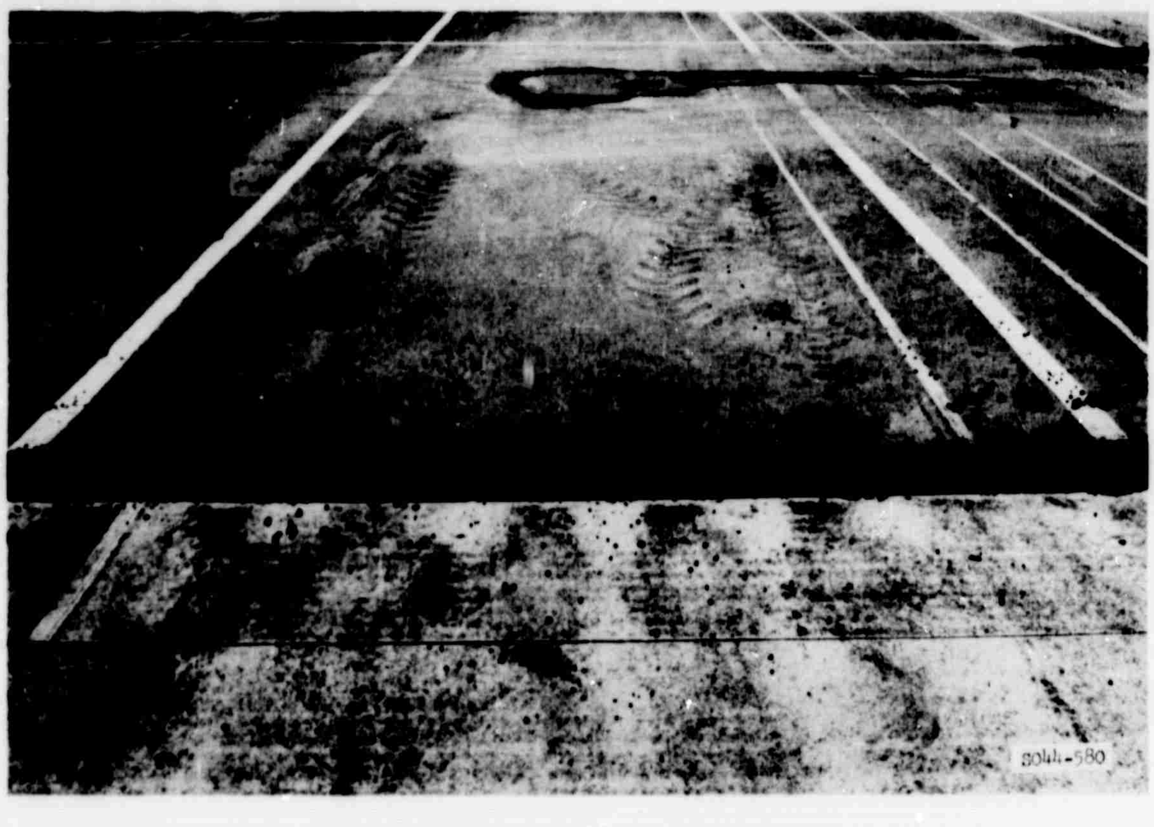


Photo B22. Item 3, lane 2, prior to traffic

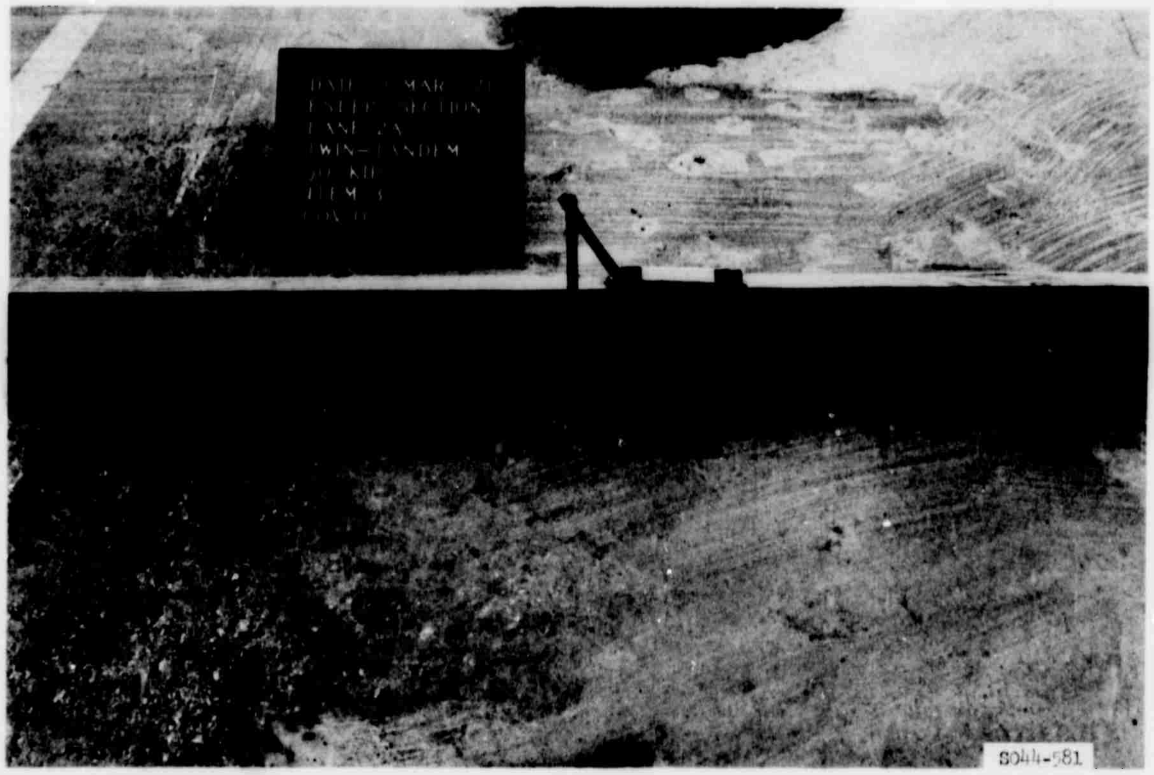


Photo B23. Deformation in item 3, lane 2, prior to 200-kip, twin-tandem-assembly traffic

Photo B24. General view of item 3, lane 2, at failure (after 890 coverages of 200-kip, twin-tandem assembly)

Photo B25. Close-up of cracks in 100 percent coverage zone in item 3, lane 2, at failure

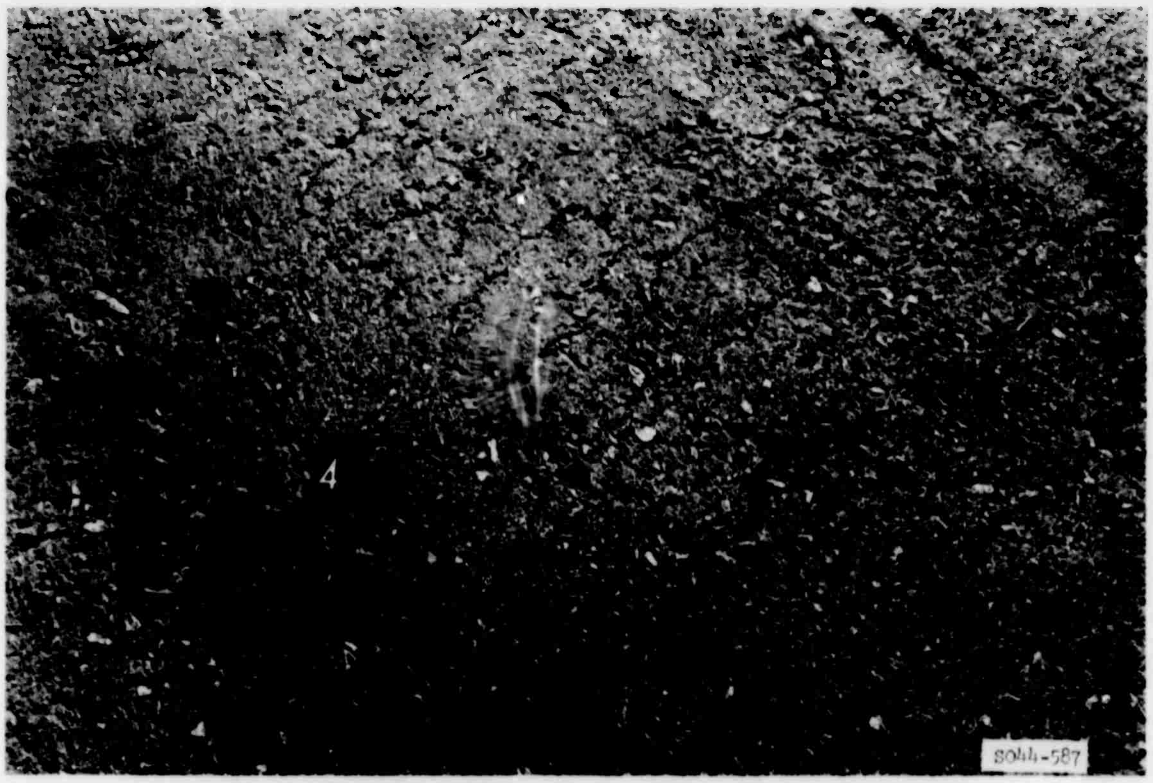


Photo B26. Small hairline cracks transverse to direction of traffic in item 4, lane 2, after 560 coverages of 200-kip, twin-tandem assembly

Photo B27. General view of item 4, lane 2, at failure (after 1810 coverages of 200-kip, twin-tandem assembly)

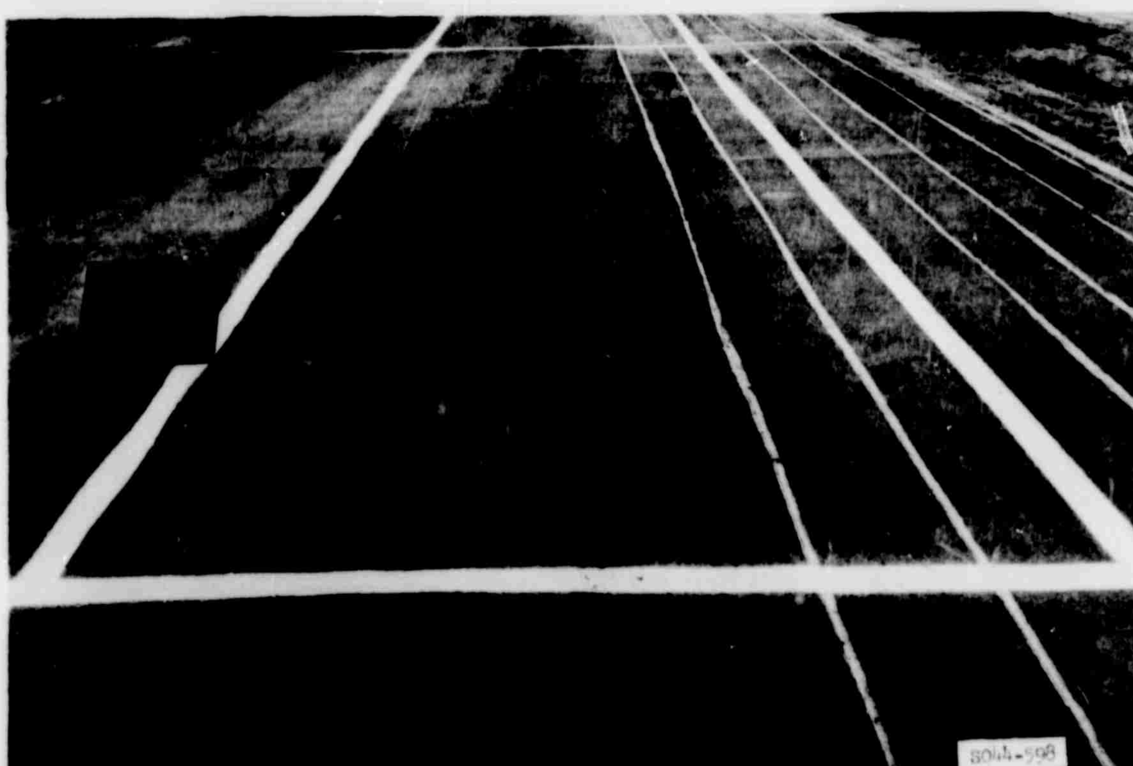


Photo B28. Item 1, lane 3, prior to traffic

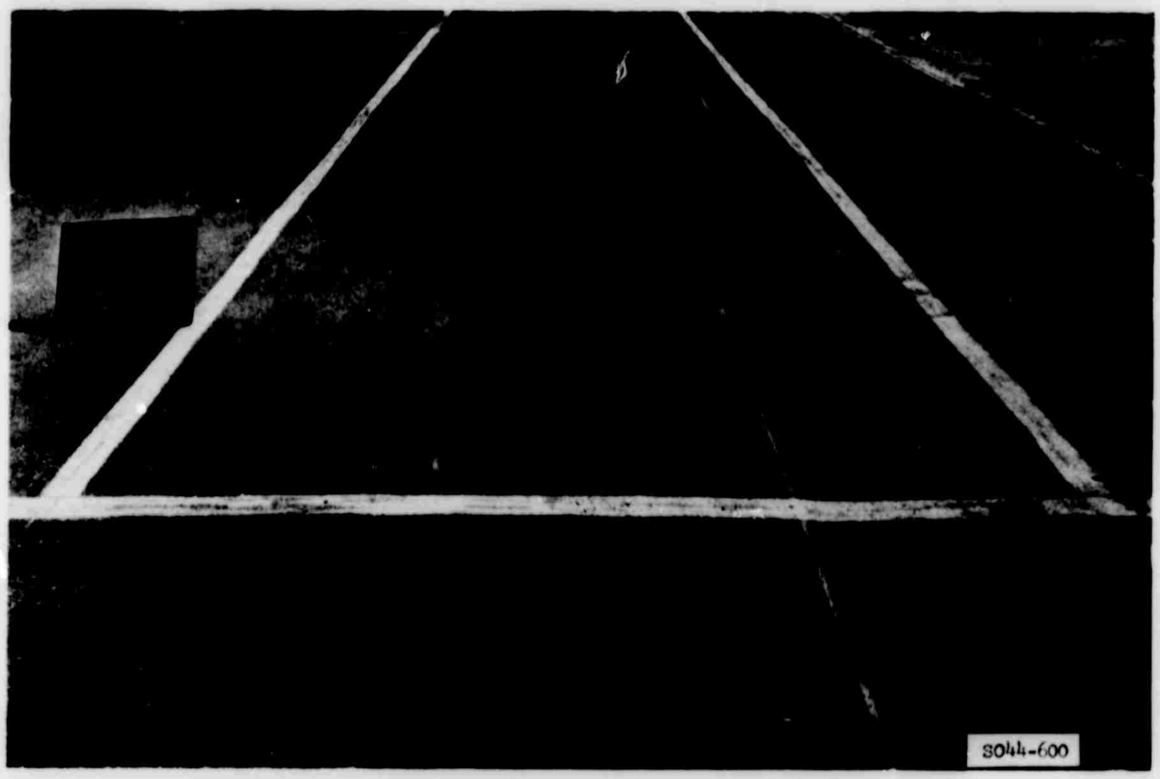


Photo B29. Item 1, lane 3, at failure (after 40 coverages of 50-kip, single-wheel assembly)

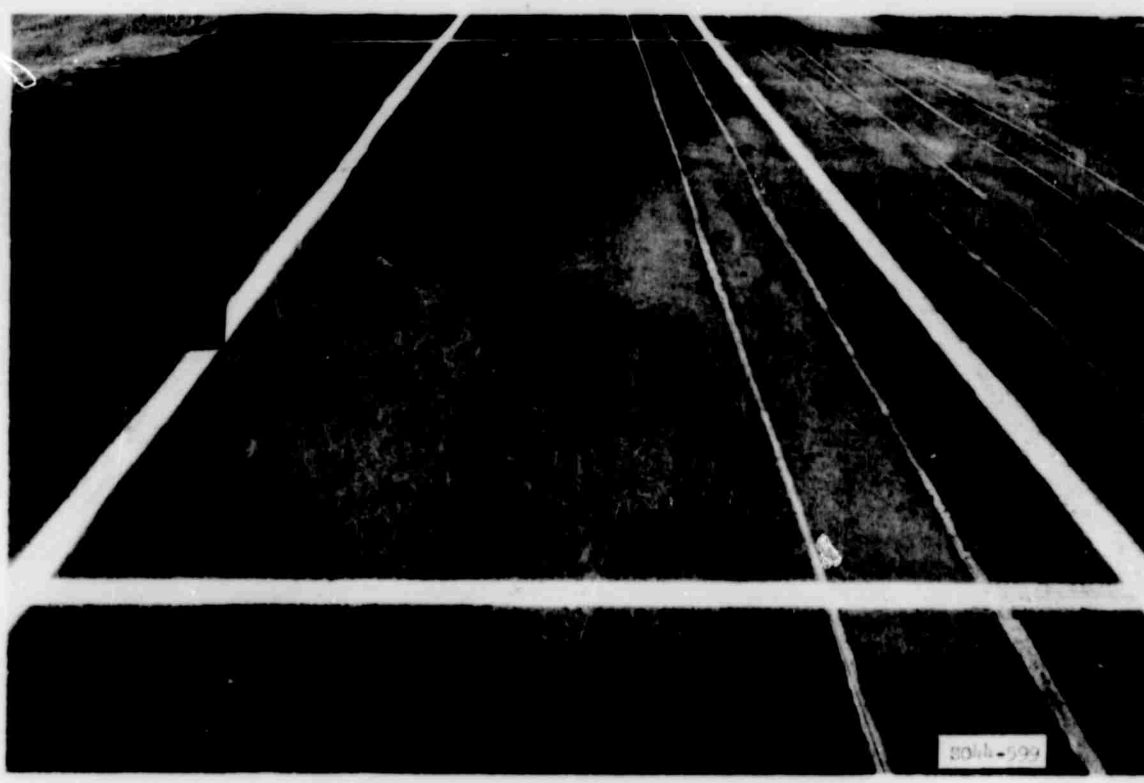


Photo B30. Item 2, lane 3, prior to traffic

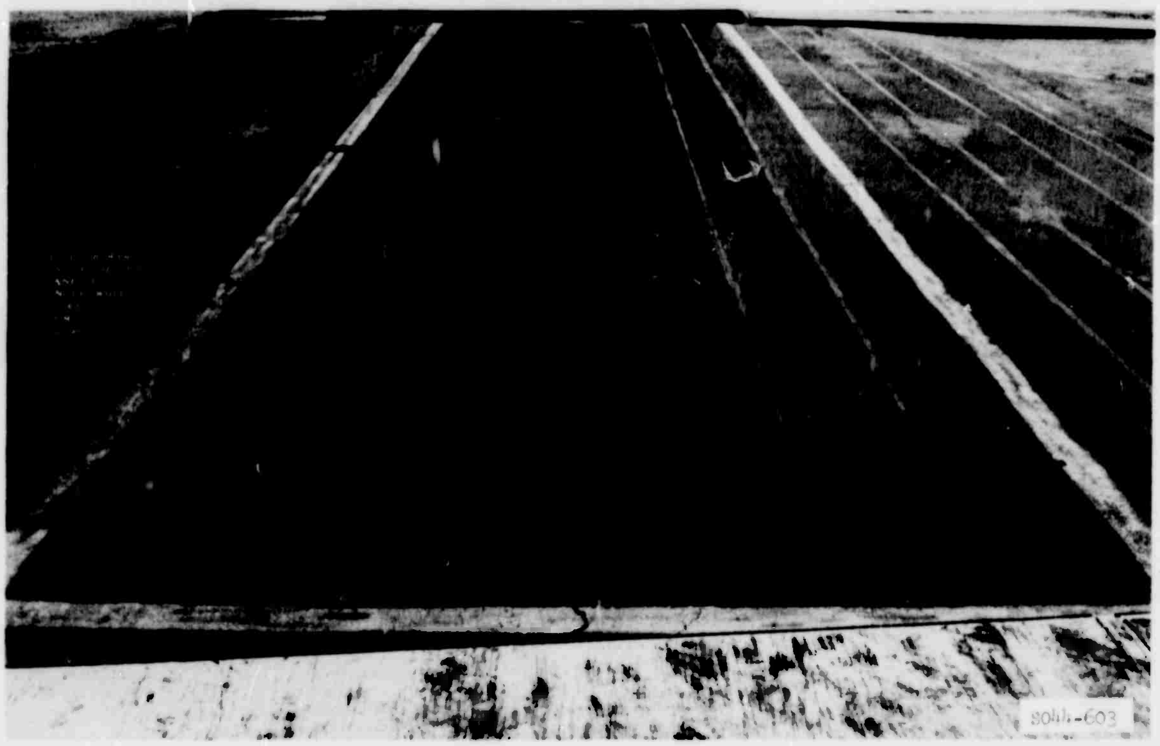


Photo B31. Failure of construction joint in item 2, lane 3 (after 90 coverages of 50-kip, single-wheel assembly)

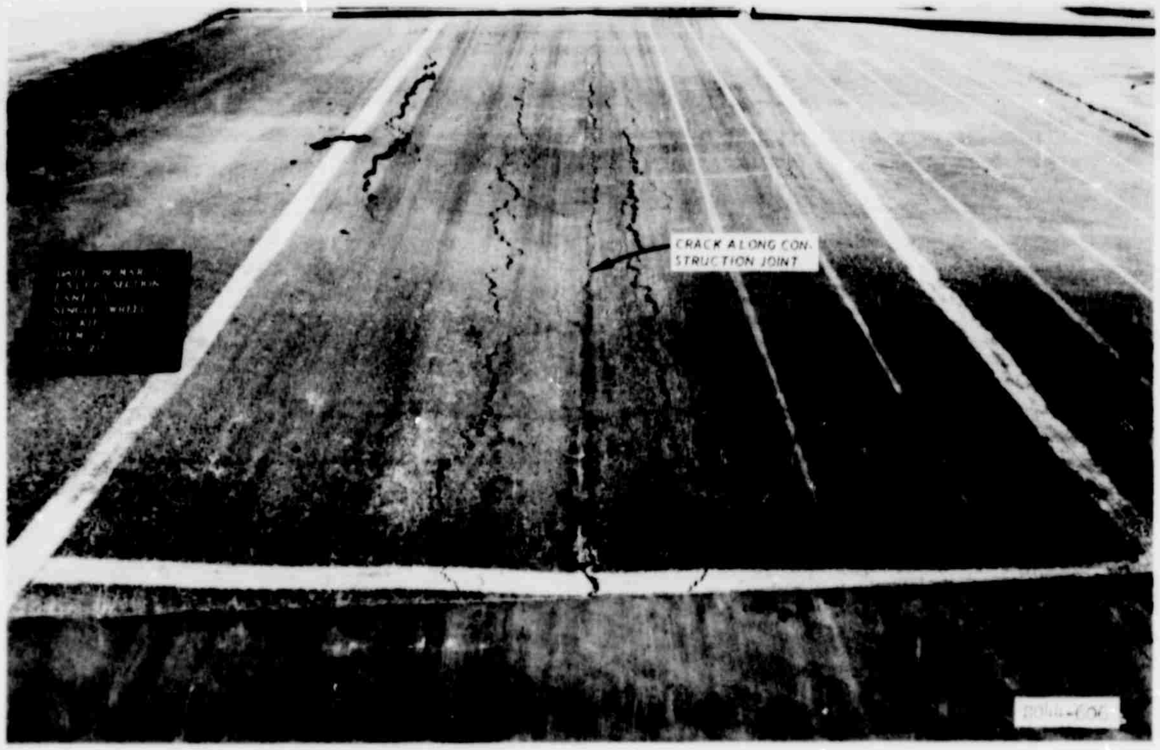


Photo B32. General view of item 2, lane 3, after 120 coverages of 50-kip, single-wheel assembly

Photo B33. Close-up of severely cracked pavement in 100 percent coverage zone of item 2, lane 2, after 120 coverages of 50-kip, single-wheel assembly

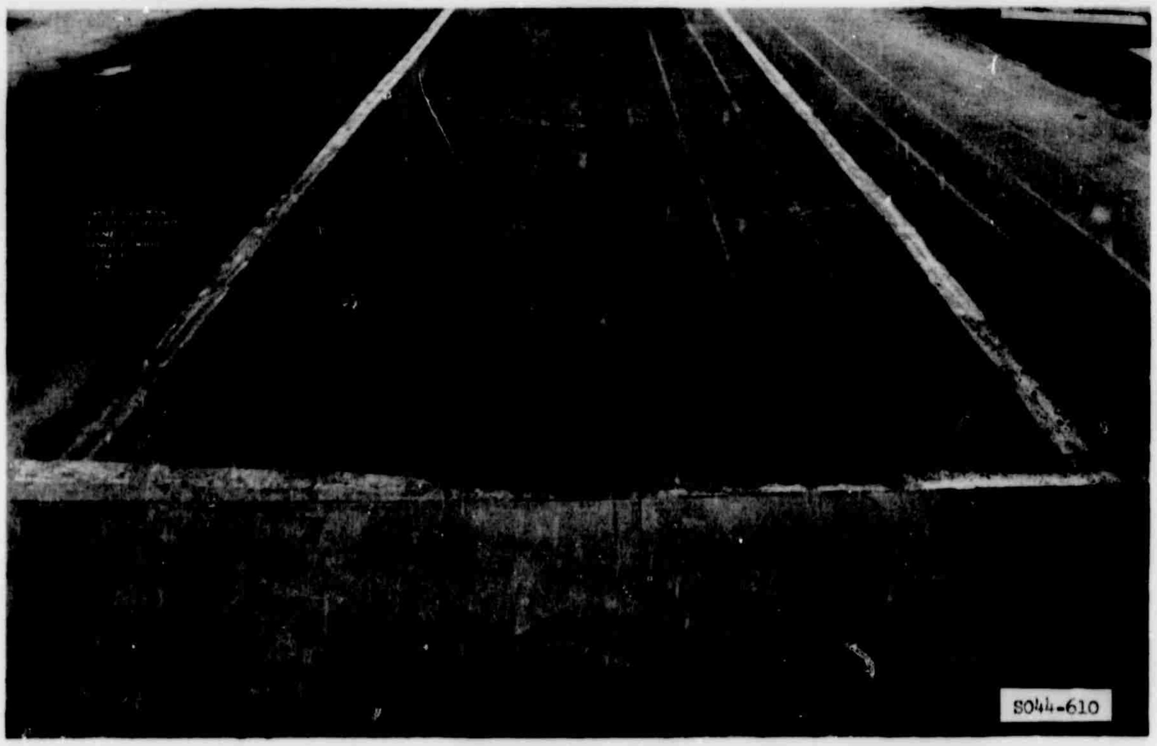


Photo B34. General view of item 3, lane 3, at failure (after 50 coverages of 75-kip, single-wheel assembly)

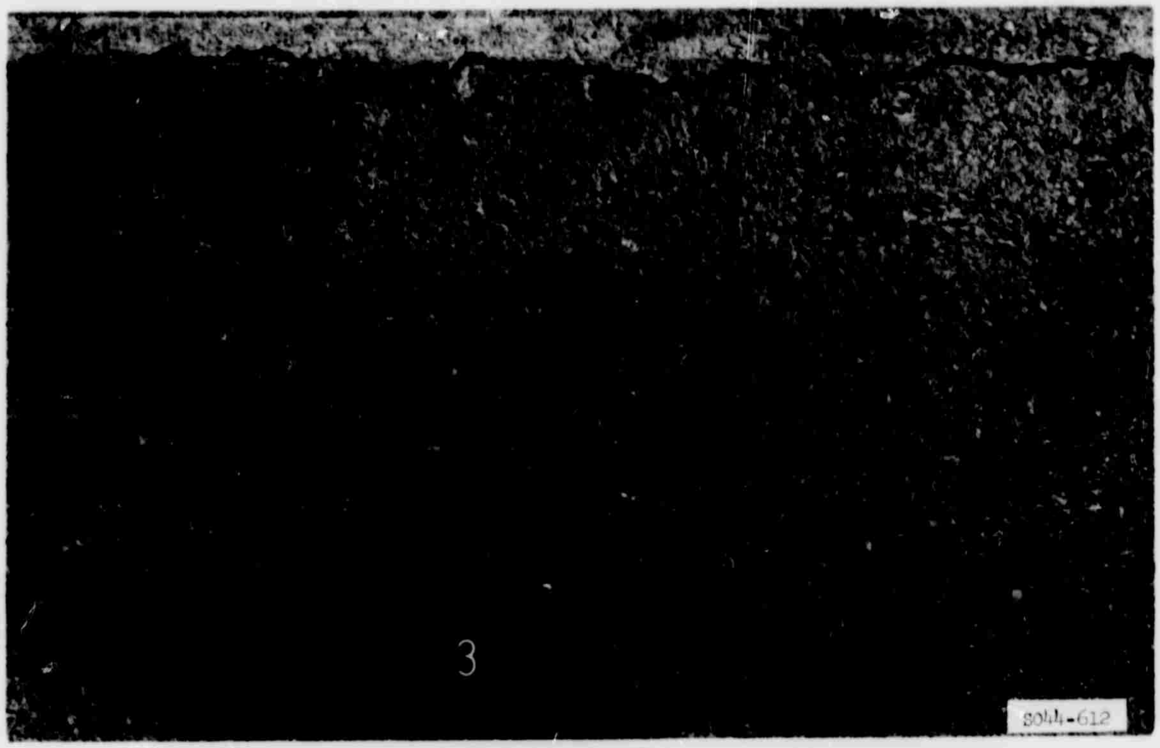


Photo B35. Severely cracked pavement in 100 percent coverage zone of item 3, lane 3, at failure (after 50 coverages of 75-kip, single-wheel assembly)

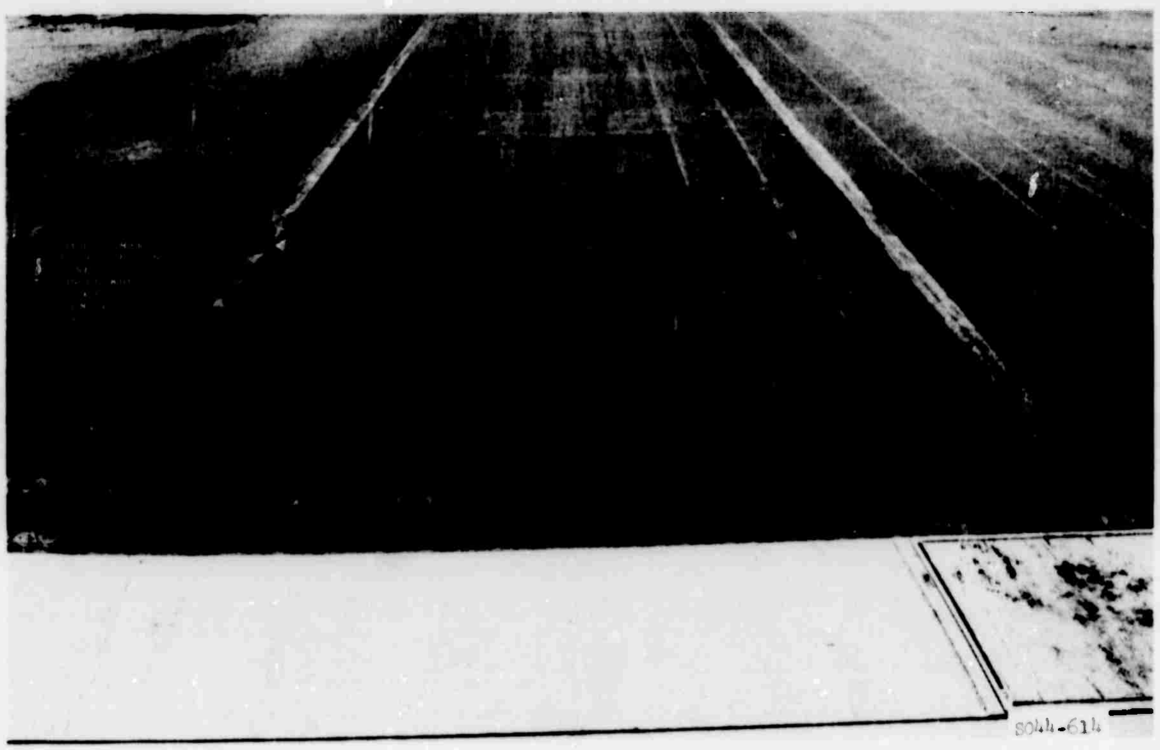


Photo B36. General view of item 4, lane 3, at failure (after 120 coverages of 75-kip, single-wheel assembly)

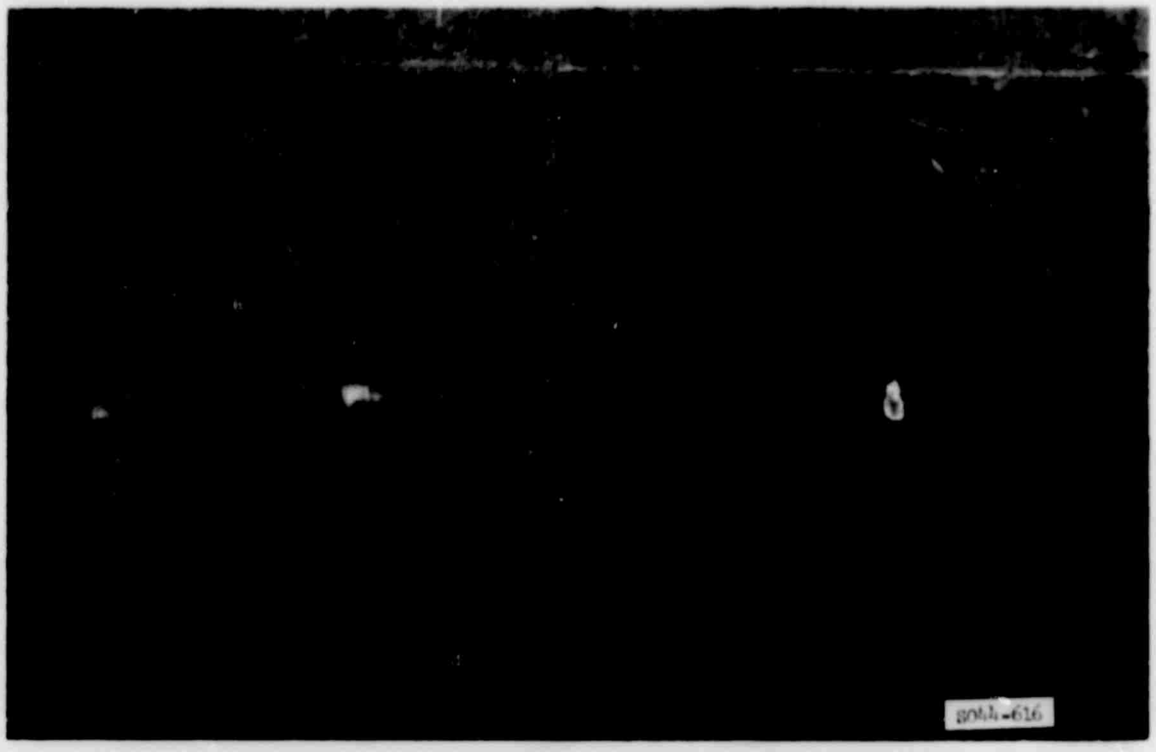


Photo B37. Close-up of cracking pattern in 100 percent coverage zone in item 4, lane 3, at failure

Photo B38. General view of item 4, lane 1, after 10,406 coverages of the 360-kip, 12-wheel assembly and prior to trafficking with the 75-kip, single-wheel assembly

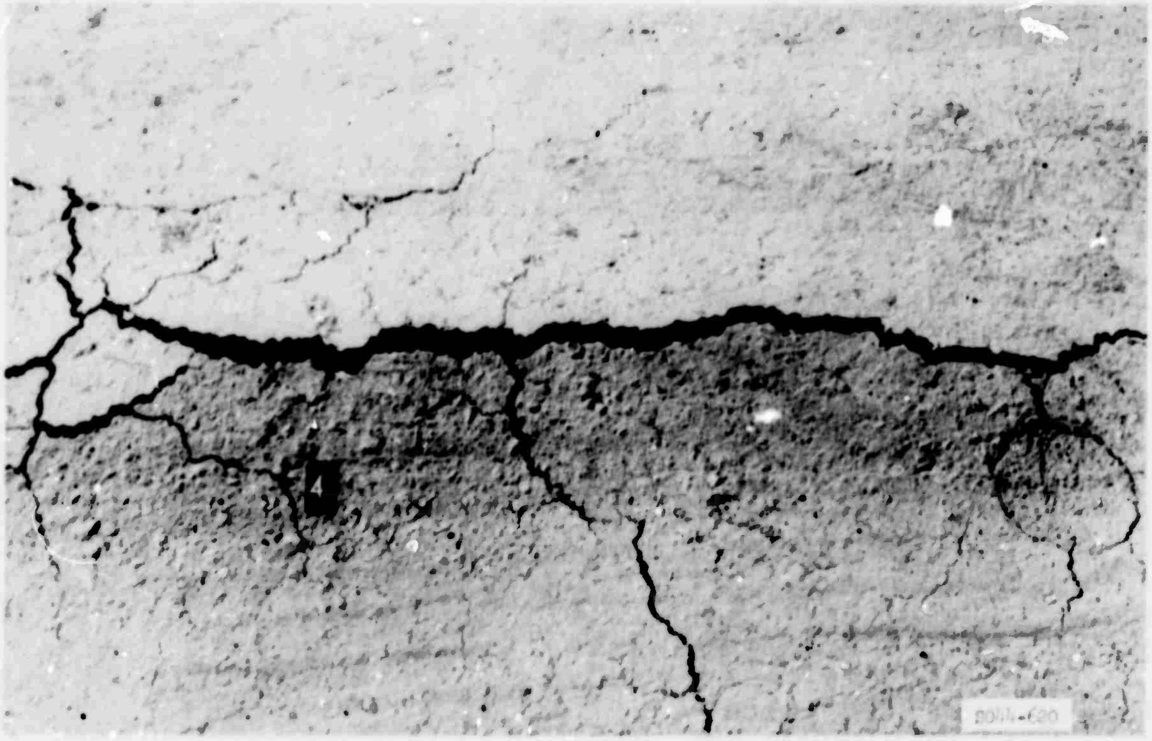


Photo B39. Severe cracking in the 100 percent coverage zone of item 4 at failure (after mixed traffic)

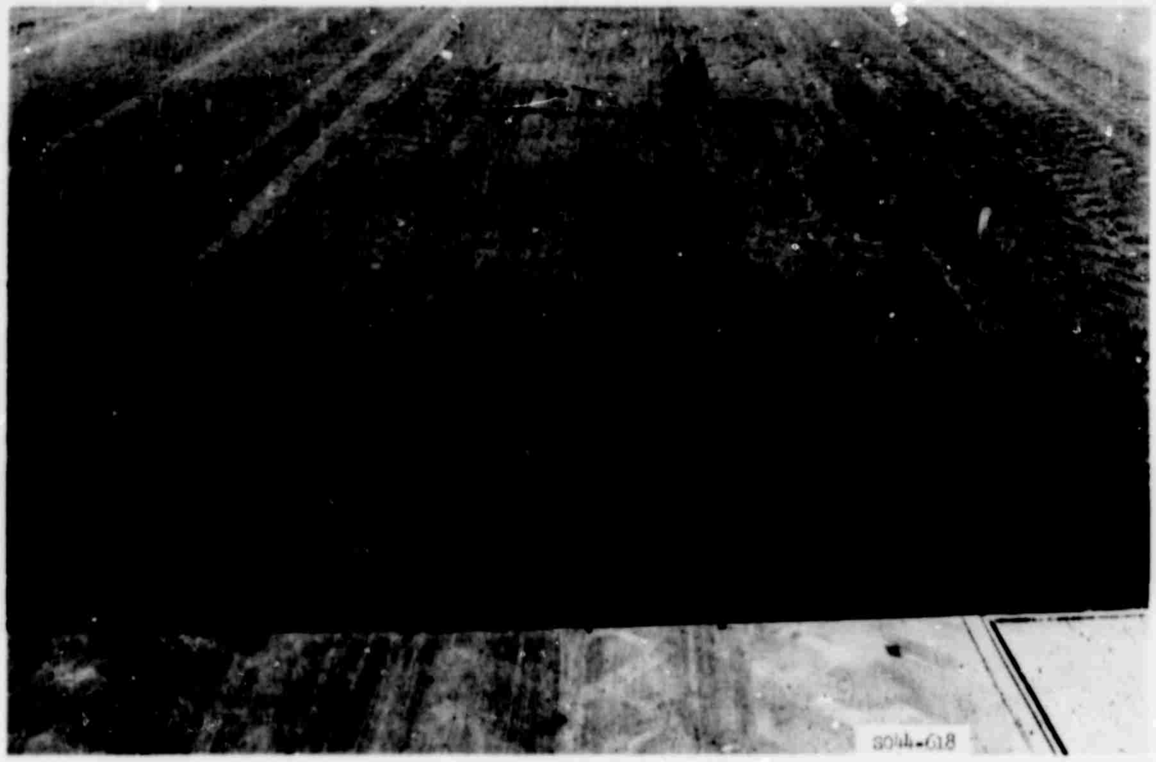
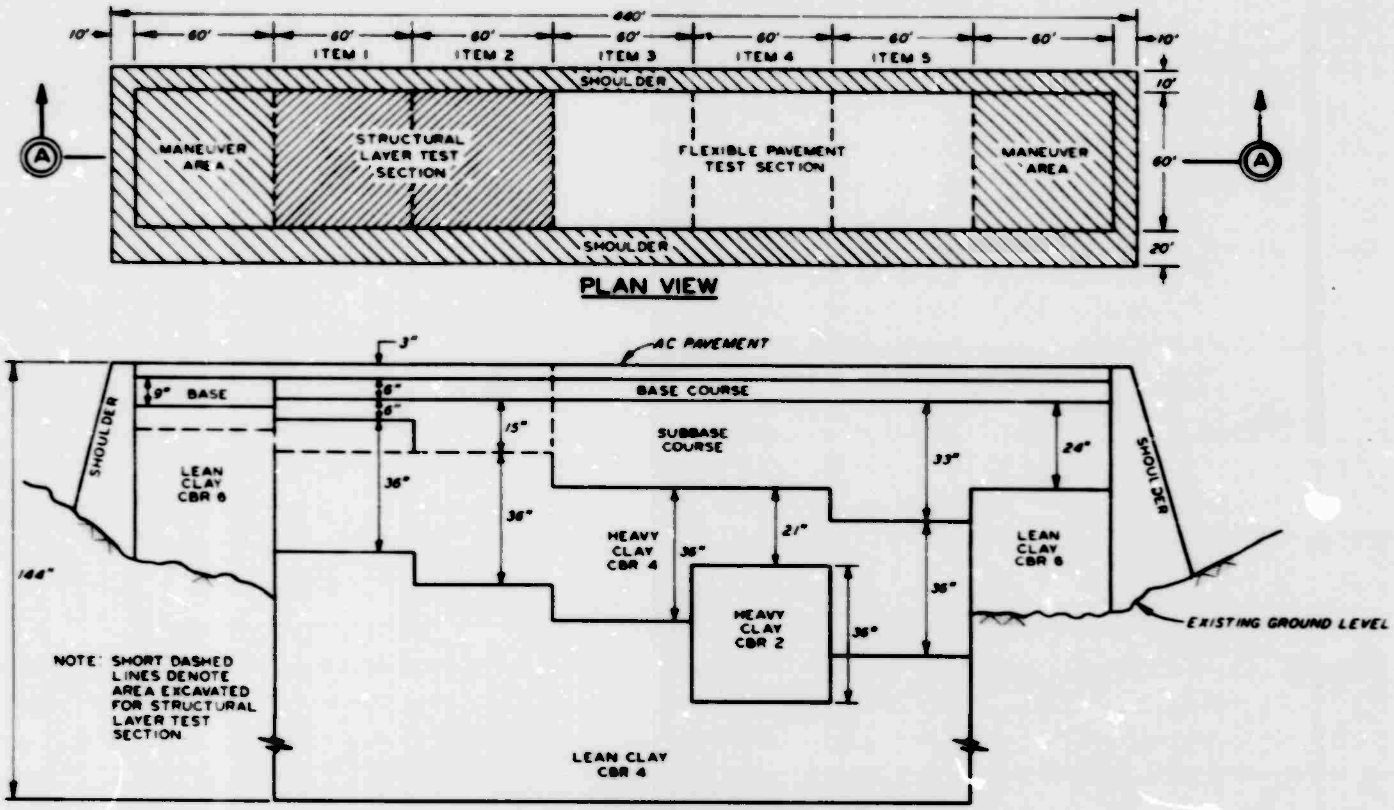


Photo B40. General view of item 4, lane 1, at failure (after mixed traffic)



SECTION A-A
FLEXIBLE PAVEMENT TEST SECTION

LOCATION OF STRUCTURAL LAYER TEST SECTION
EXISTING MWHGL TEST SECTION

PLATE B1

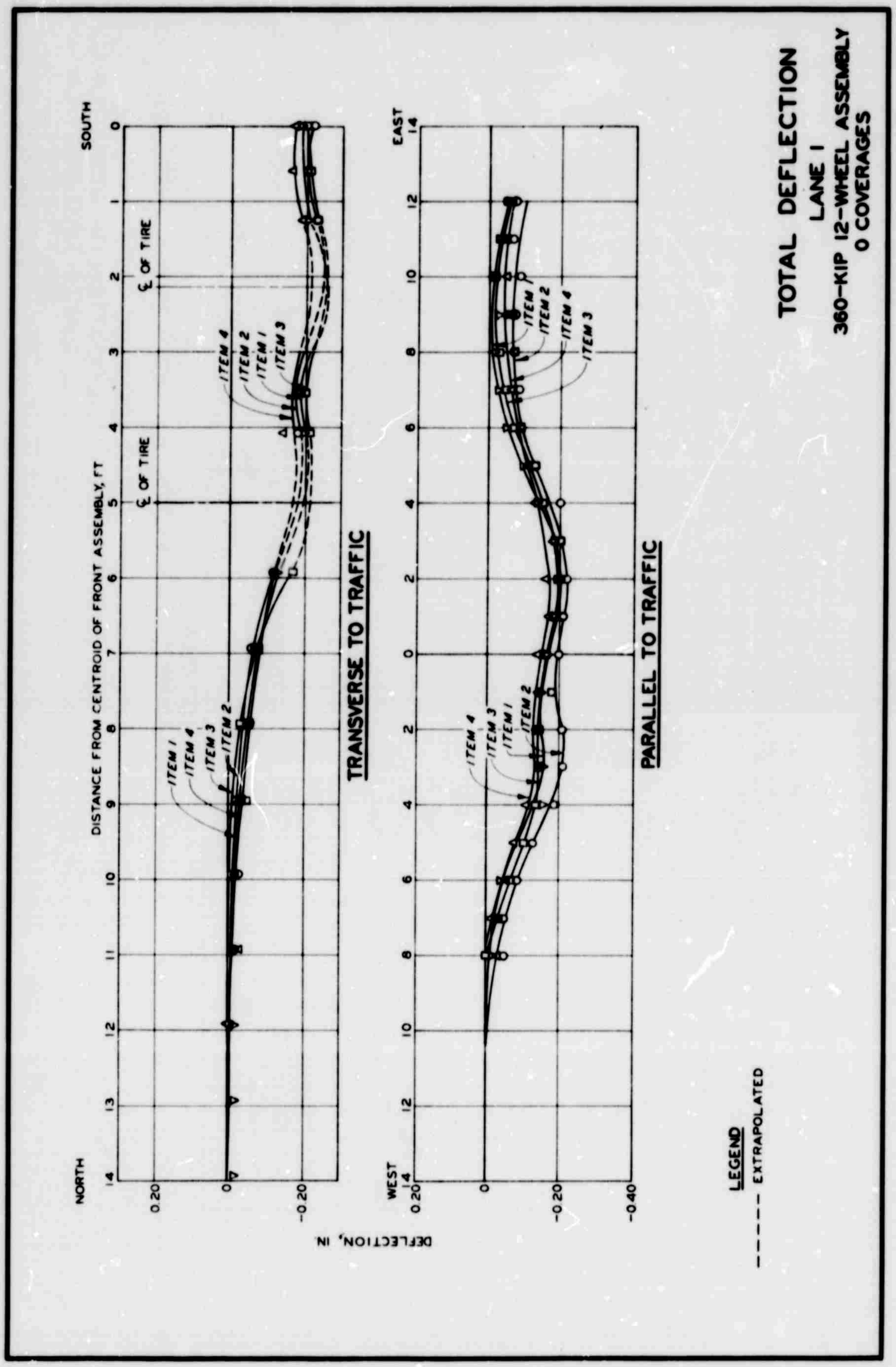

TOTAL DEFLECTION
LANE 1
360-KIP 12-WHEEL ASSEMBLY
0 COVERAGES
TRANSVERSE TO TRAFFIC
PARALLEL TO TRAFFIC
DISTANCE FROM CENTROID OF FRONT ASSEMBLY, FT
DEFLECTION, IN.
NORTH
SOUTH
WEST
EAST
℄ OF TIRE
ITEM 1
ITEM 2
ITEM 3
ITEM 4
LEGEND
EXTRAPOLATED

PLATE B2

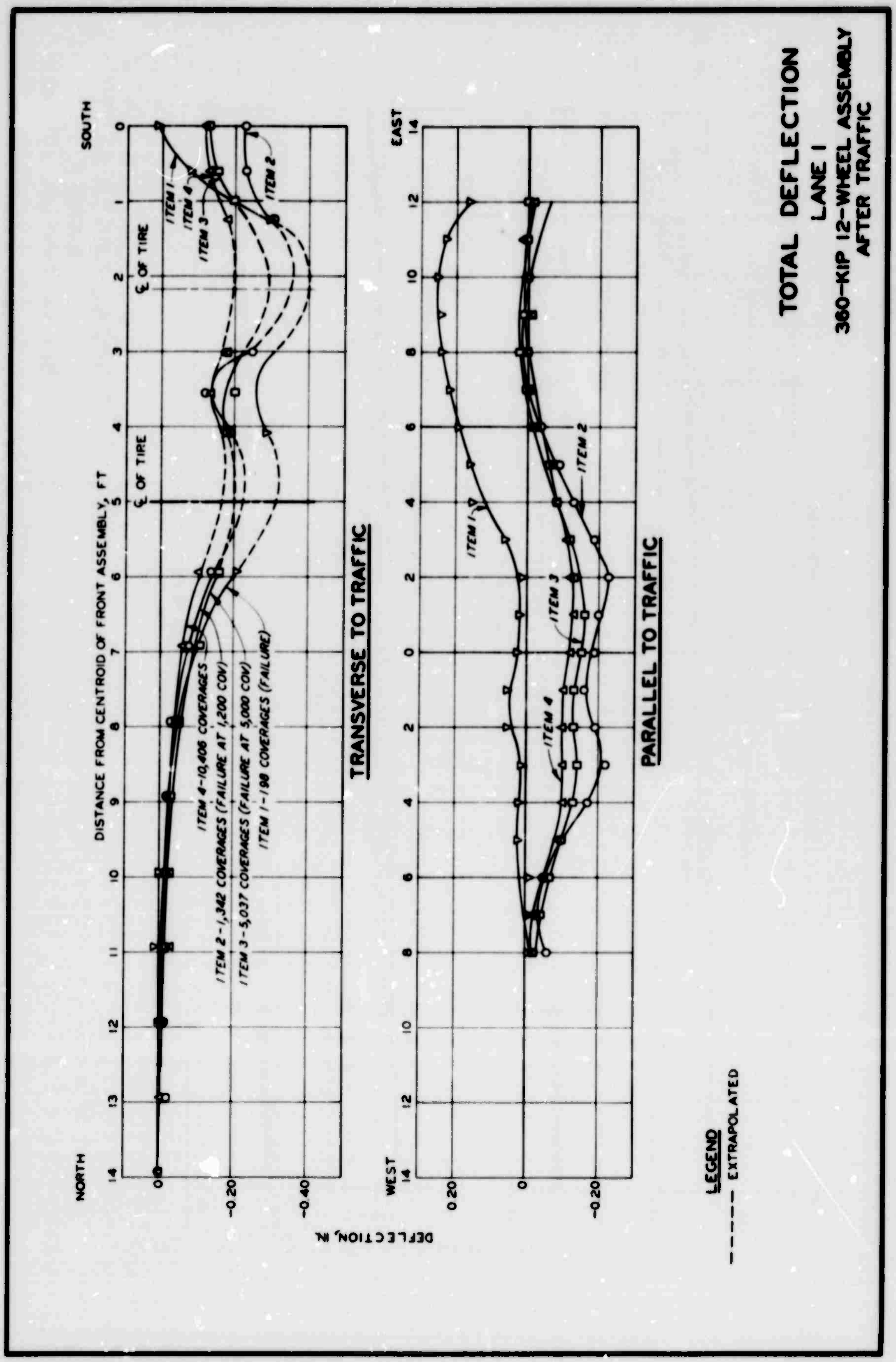
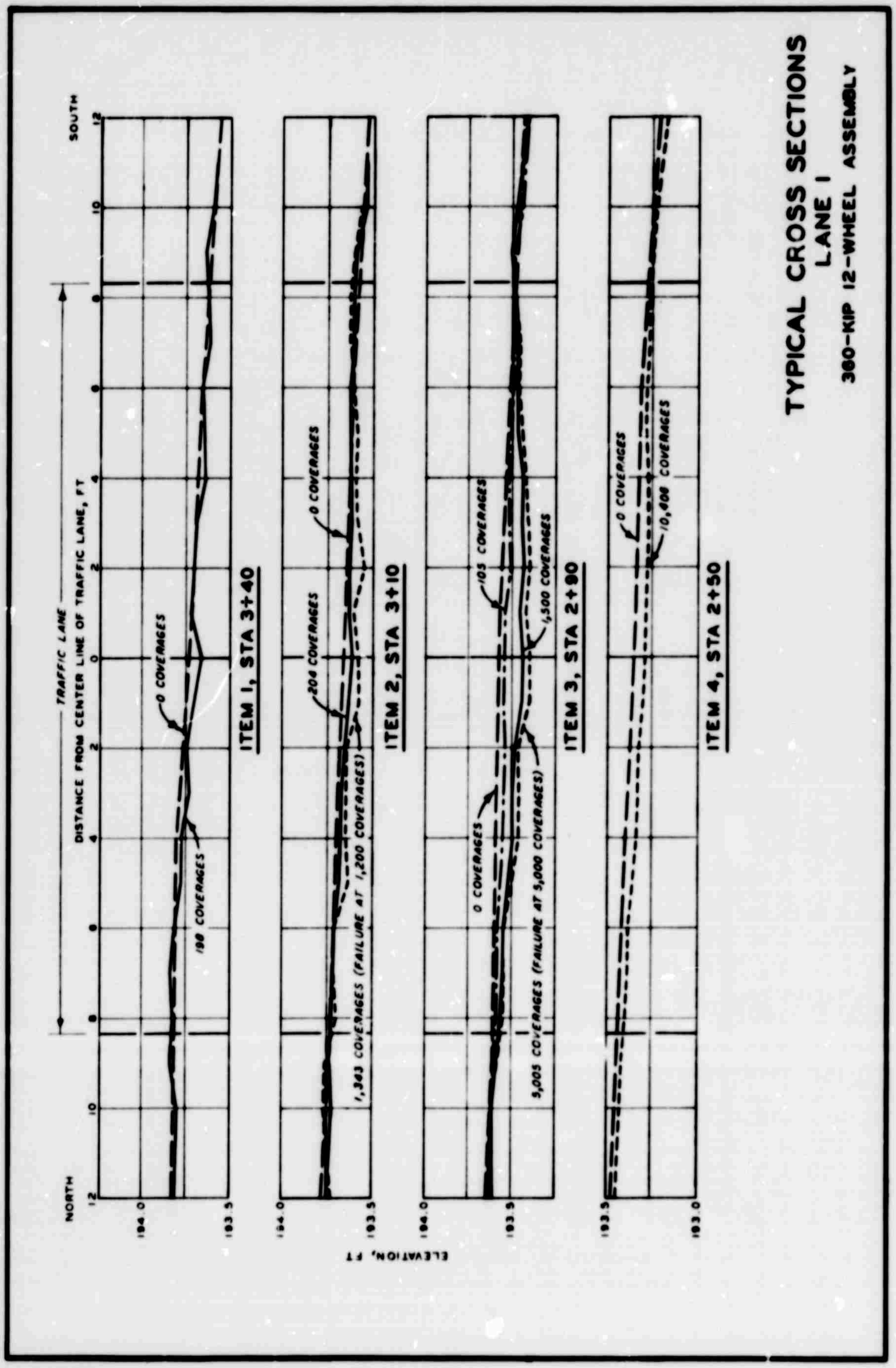
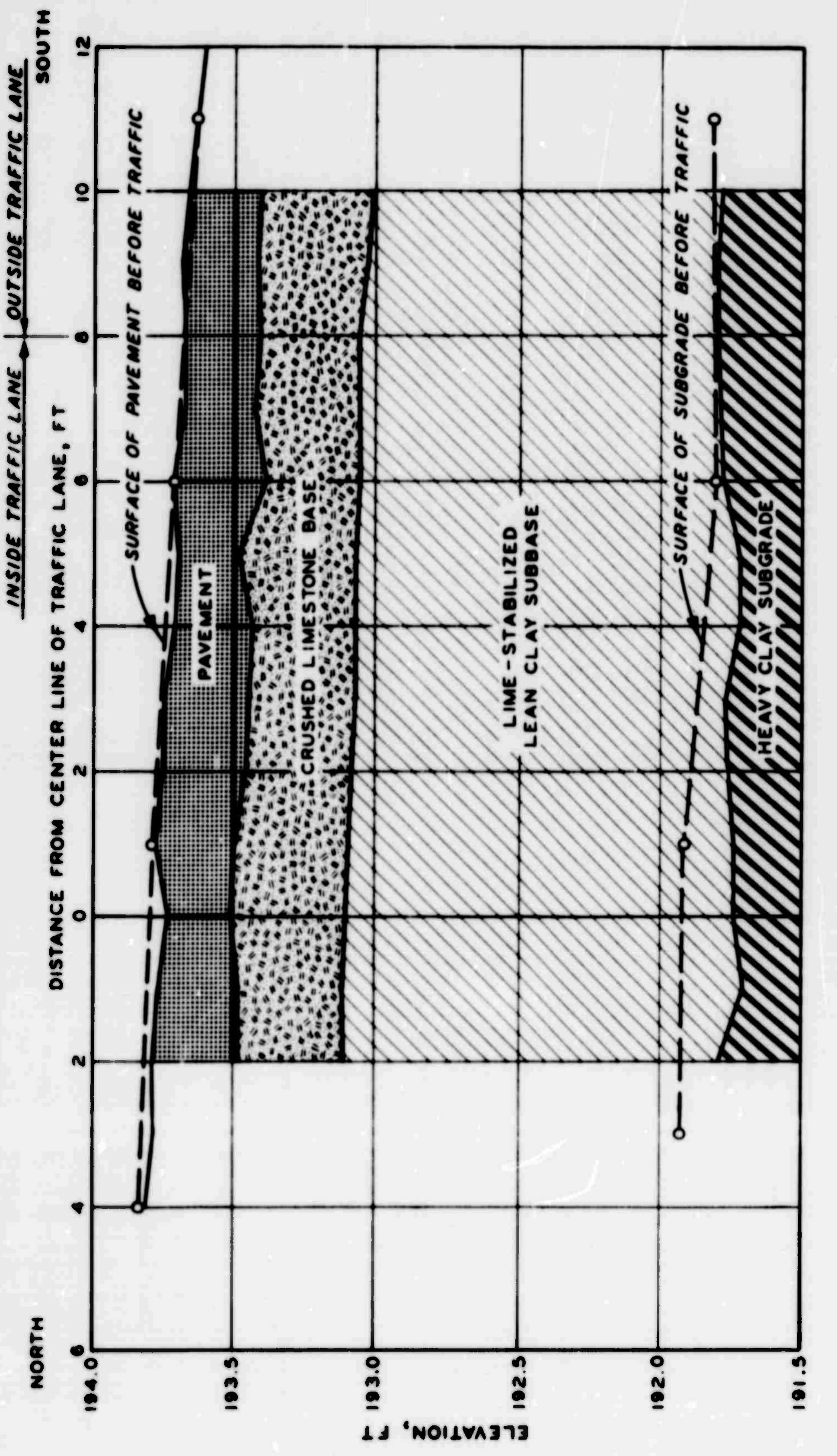

TOTAL DEFLECTION
LANE I
360-KIP 12-WHEEL ASSEMBLY
AFTER TRAFFIC
TRANSVERSE TO TRAFFIC
PARALLEL TO TRAFFIC
DISTANCE FROM CENTROID OF FRONT ASSEMBLY, FT
DEFLECTION, IN.
NORTH
SOUTH
WEST
EAST
℄ OF TIRE
ITEM 4-10,406 COVERAGES
ITEM 2-1,342 COVERAGES (FAILURE AT 1,200 COV)
ITEM 3-5,037 COVERAGES (FAILURE AT 5,000 COV)
ITEM 1-198 COVERAGES (FAILURE)
ITEM 1
ITEM 2
ITEM 3
ITEM 4
LEGEND
EXTRAPOLATED

PLATE B3

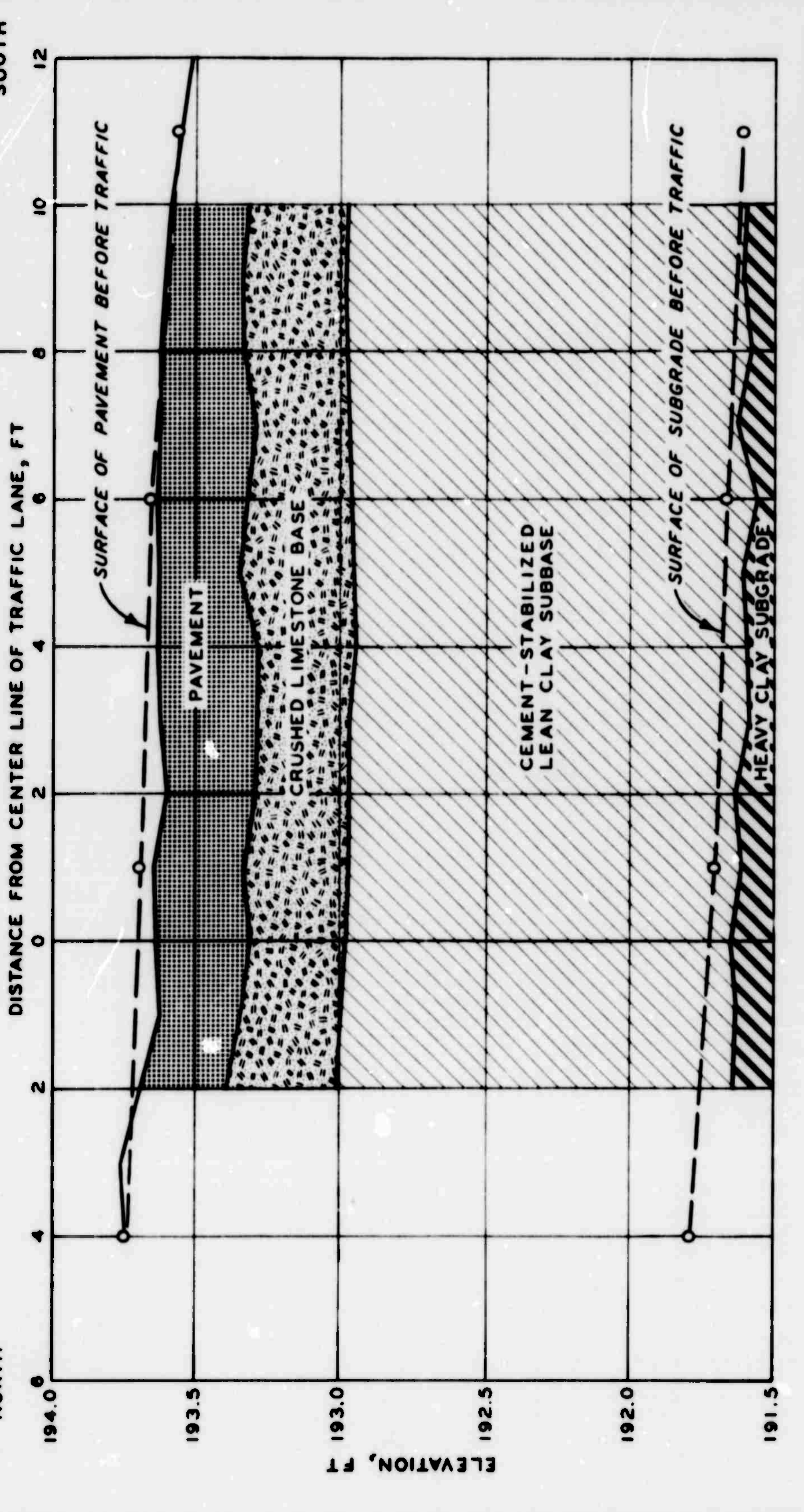


TYPICAL CROSS SECTIONS
LANE 1
360-KIP 12-WHEEL ASSEMBLY
ITEM 1, STA 3+40
ITEM 2, STA 3+10
ITEM 3, STA 2+90
ITEM 4, STA 2+50
0 COVERAGES
198 COVERAGES
204 COVERAGES
1,343 COVERAGES (FAILURE AT 1,200 COVERAGES)
105 COVERAGES
1,500 COVERAGES
5,005 COVERAGES (FAILURE AT 5,000 COVERAGES)
10,408 COVERAGES
TRAFFIC LANE
DISTANCE FROM CENTER LINE OF TRAFFIC LANE, FT
NORTH
SOUTH
ELEVATION, FT
194.0
193.5
193.0

PLATE B4

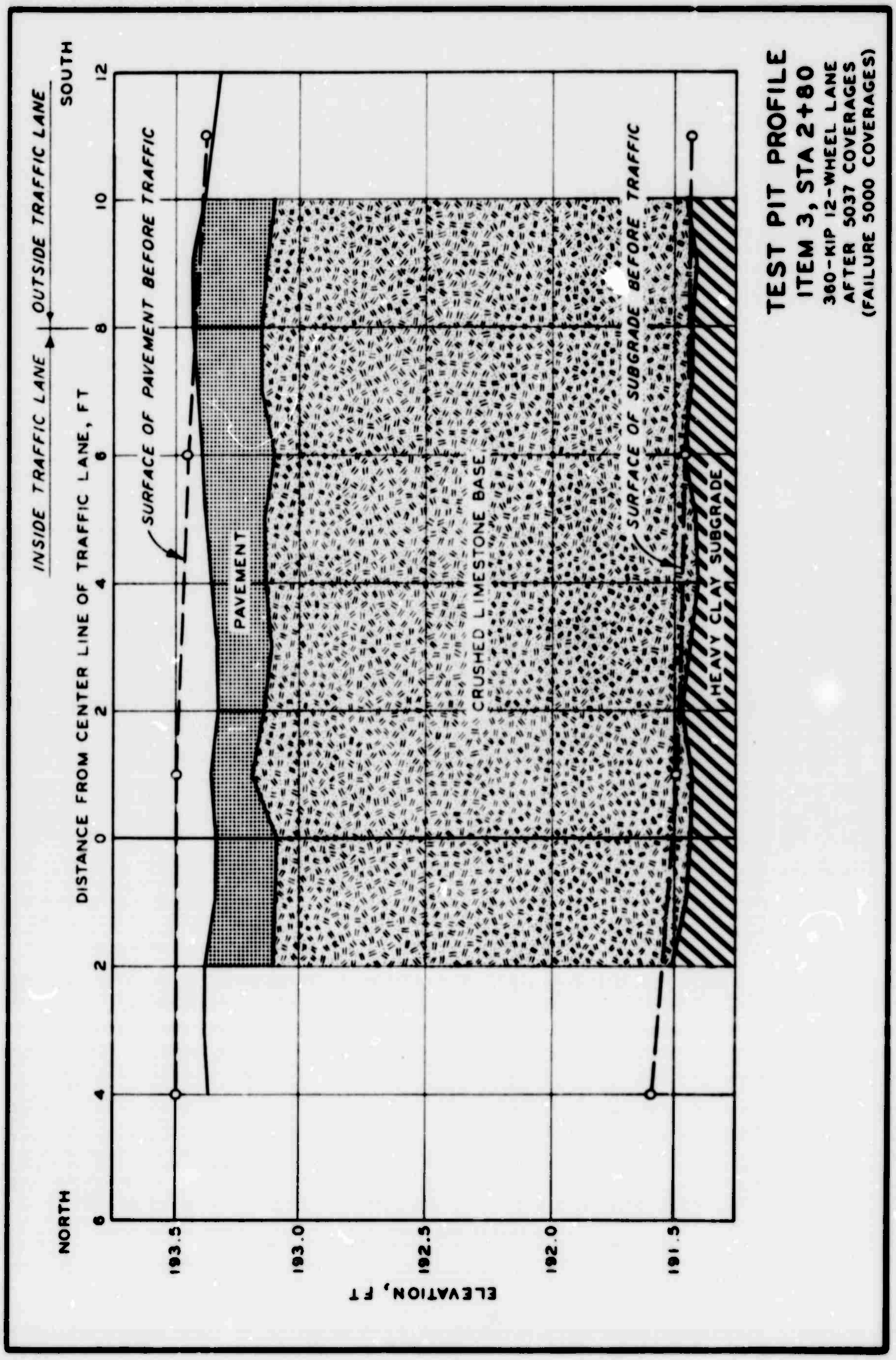


TEST PIT PROFILE
ITEM 1, STA 3+50
360-KIP 12-WHEEL LANE
AFTER 198 COVERAGES



TEST PIT PROFILE
ITEM 2, STA 3+20
360-KIP 12-WHEEL LANE
AFTER 1871 COVERAGES
(FAILURE 1200 COVERAGES)

PLATE B6



TEST PIT PROFILE
ITEM 3, STA 2+80
360-KIP 12-WHEEL LANE
AFTER 5037 COVERAGES
(FAILURE 5000 COVERAGES)
INSIDE TRAFFIC LANE
OUTSIDE TRAFFIC LANE
NORTH
SOUTH
DISTANCE FROM CENTER LINE OF TRAFFIC LANE, FT
6
4
2
0
2
4
6
8
10
12
ELEVATION, FT
193.5
193.0
192.5
192.0
191.5
SURFACE OF PAVEMENT BEFORE TRAFFIC
PAVEMENT
CRUSHED LIMESTONE BASE
SURFACE OF SUBGRADE BEFORE TRAFFIC
HEAVY CLAY SUBGRADE

PLATE B7

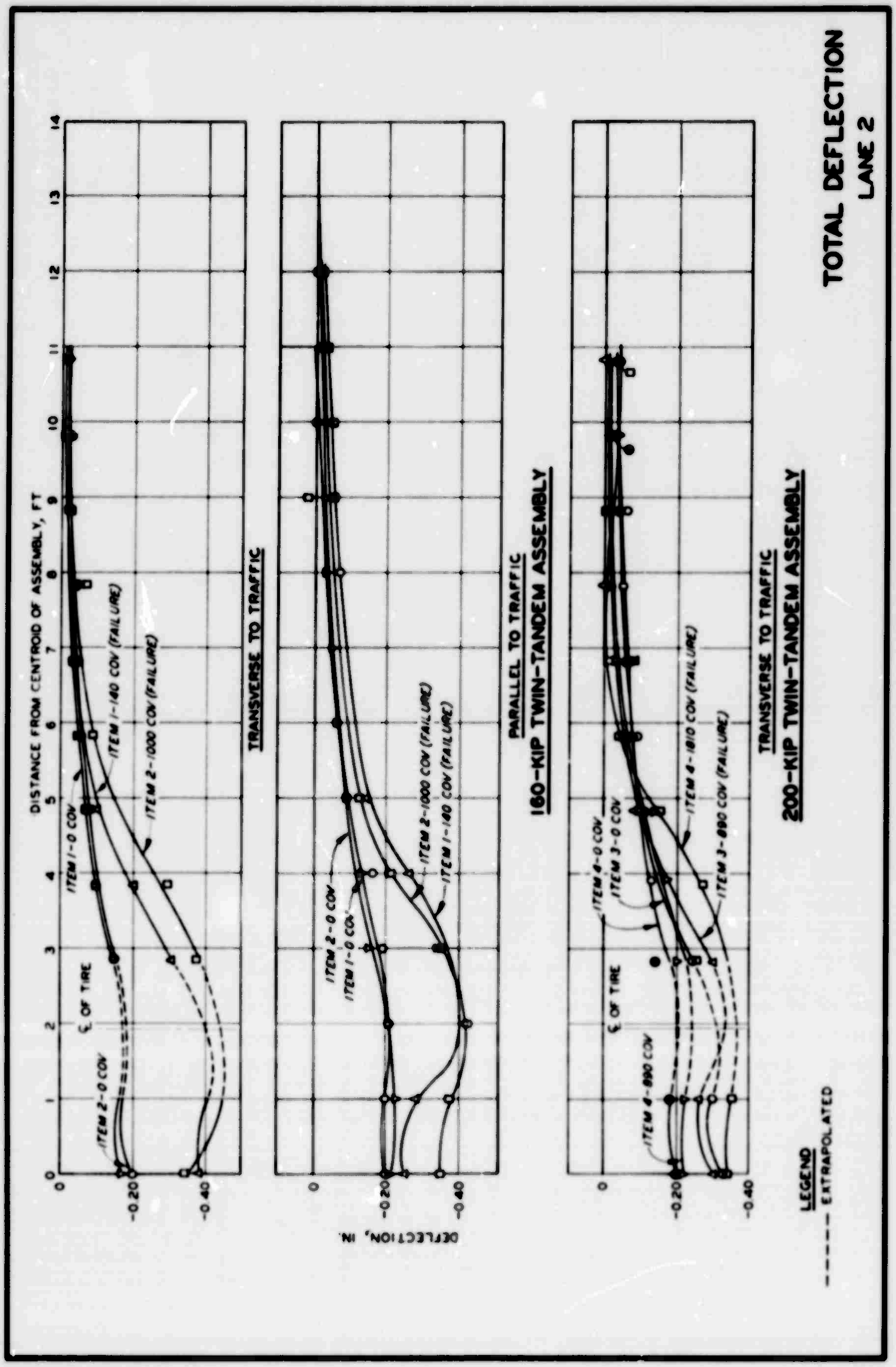
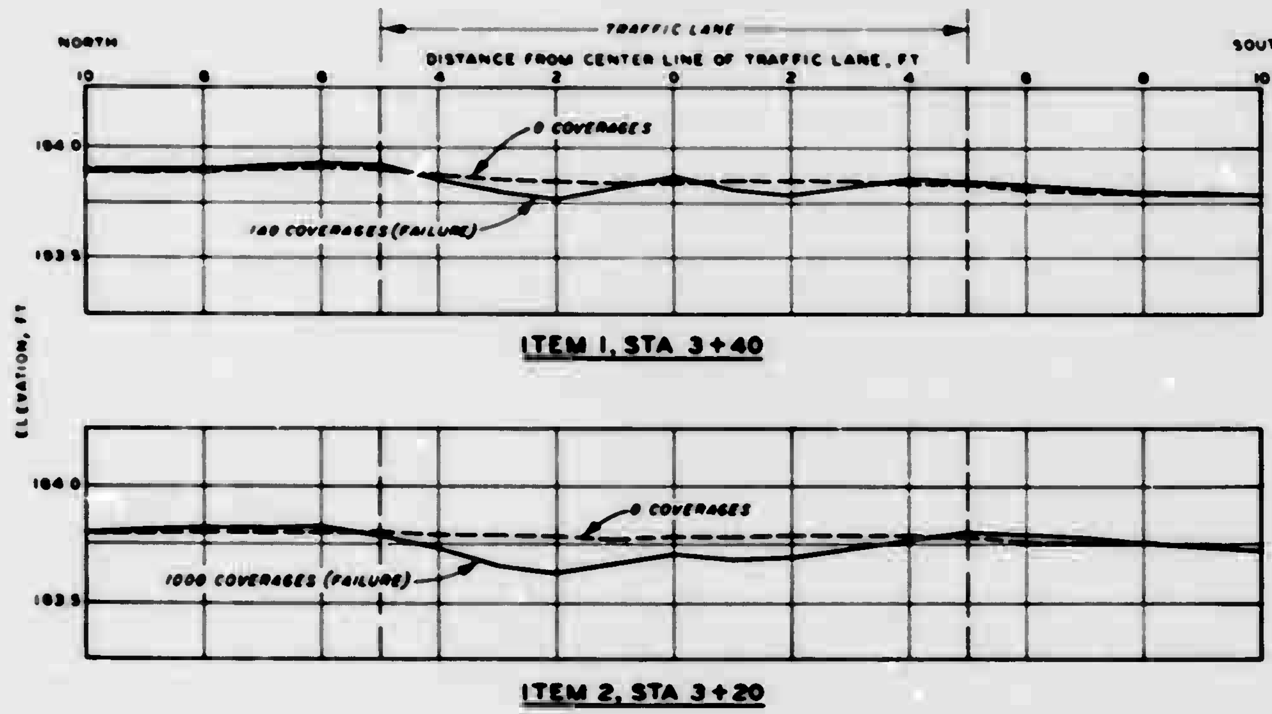
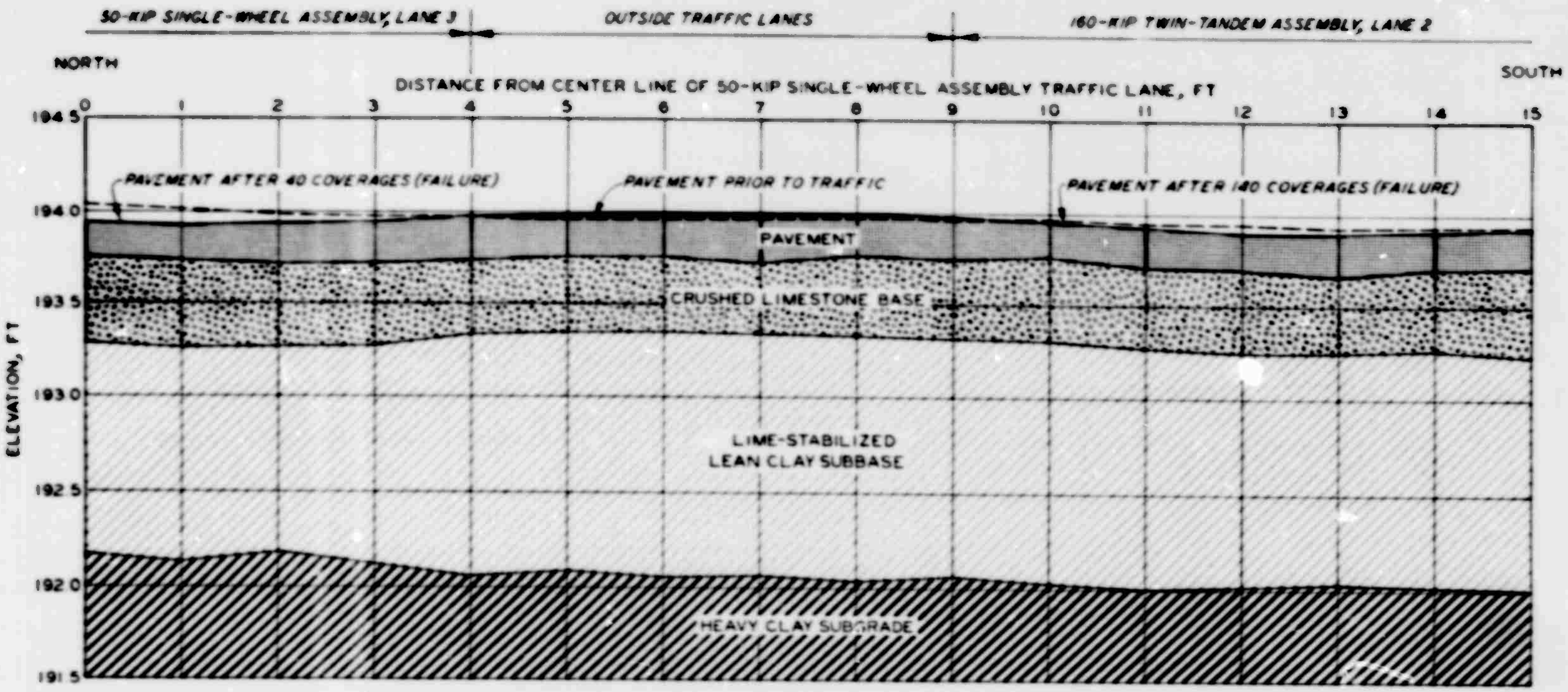

TOTAL DEFLECTION
LANE 2
DISTANCE FROM CENTROID OF ASSEMBLY, FT
DEFLECTION, IN.
TRANSVERSE TO TRAFFIC
PARALLEL TO TRAFFIC
160-KIP TWIN-TANDEM ASSEMBLY
TRANSVERSE TO TRAFFIC
200-KIP TWIN-TANDEM ASSEMBLY
ITEM 1-0 COV
ITEM 2-0 COV
ITEM 1-140 COV (FAILURE)
ITEM 2-1000 COV (FAILURE)
ITEM 2-1000 COV (FAILURE)
ITEM 1-140 COV (FAILURE)
ITEM 4-0 COV
ITEM 3-0 COV
ITEM 4-1810 COV (FAILURE)
ITEM 3-890 COV (FAILURE)
ITEM 4-890 COV
℄ OF TIRE
LEGEND
EXTRAPOLATED

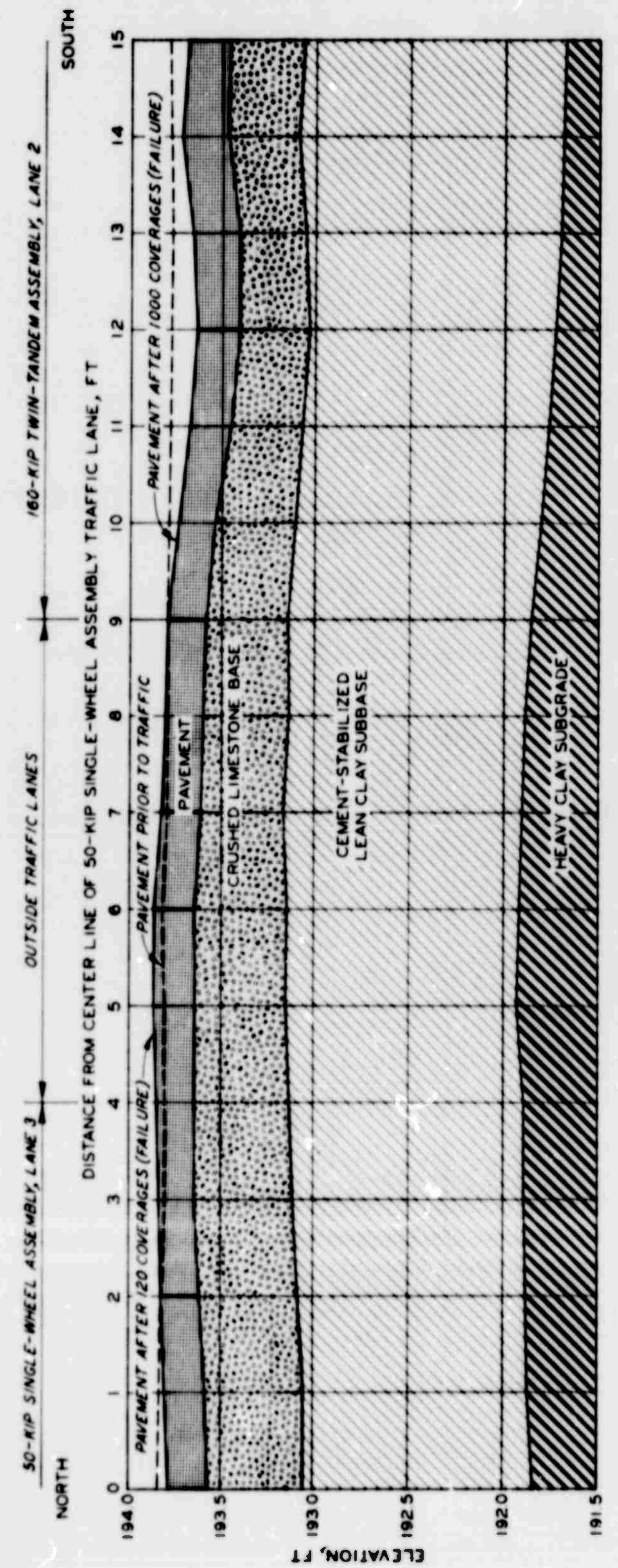
PLATE B8



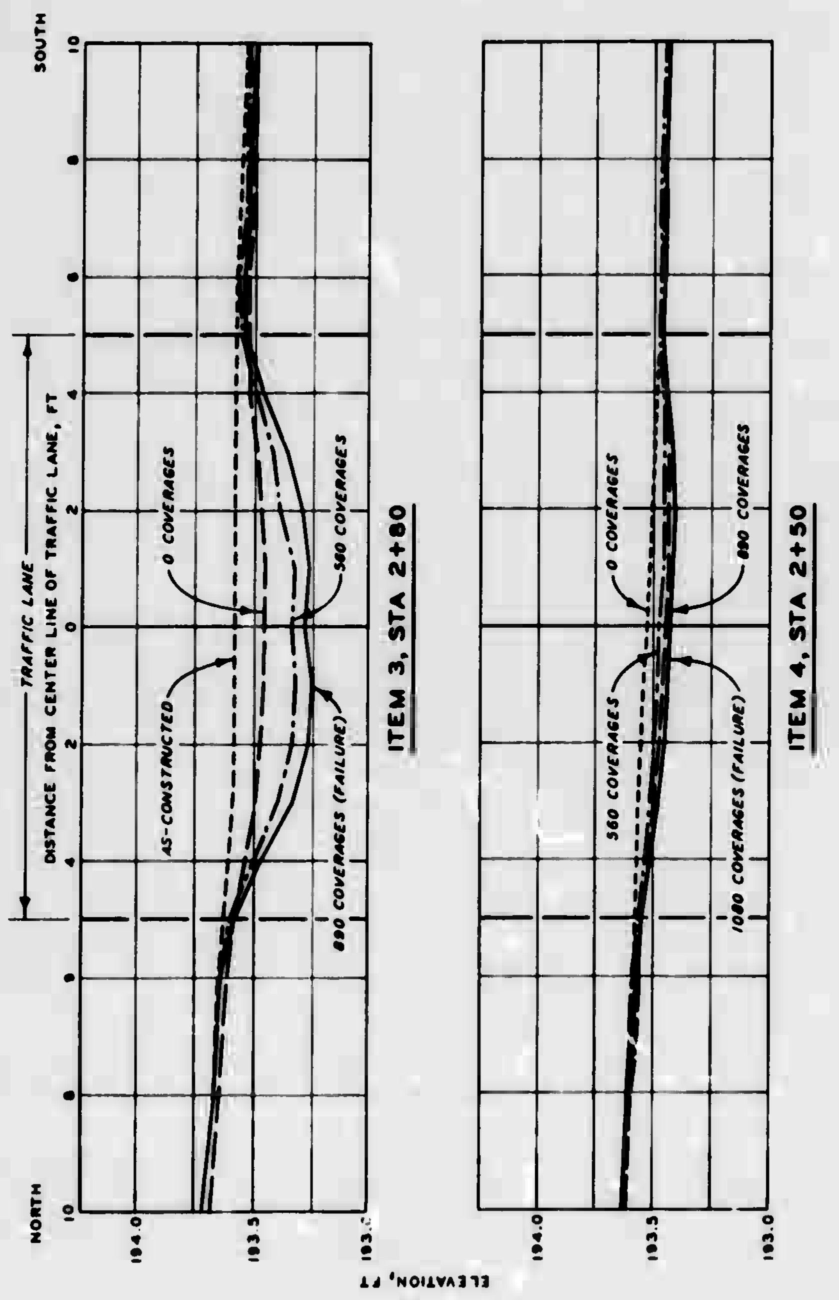
TYPICAL CROSS SECTIONS
LANE 2
160-KIP TWIN-TANDEM ASSEMBLY



TEST PIT PROFILE
ITEM 1
STA 3+50

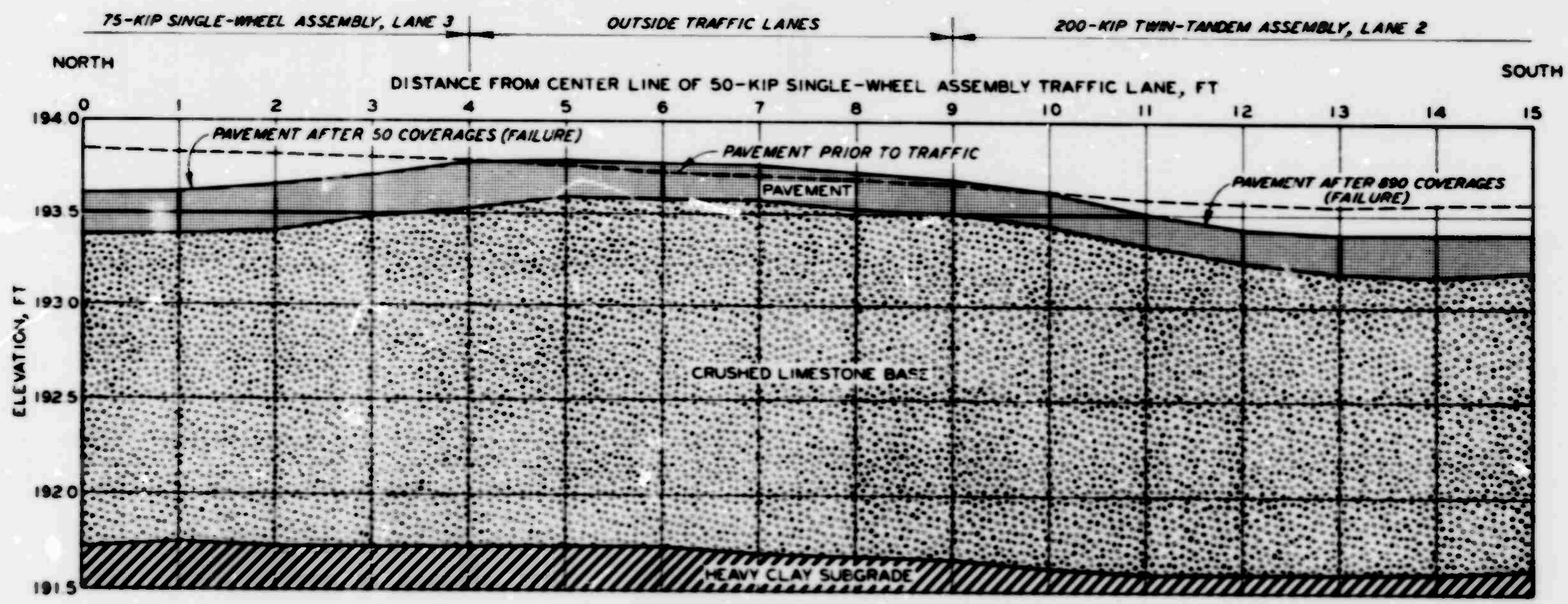


TEST PIT PROFILE
ITEM 2
STA 3+20

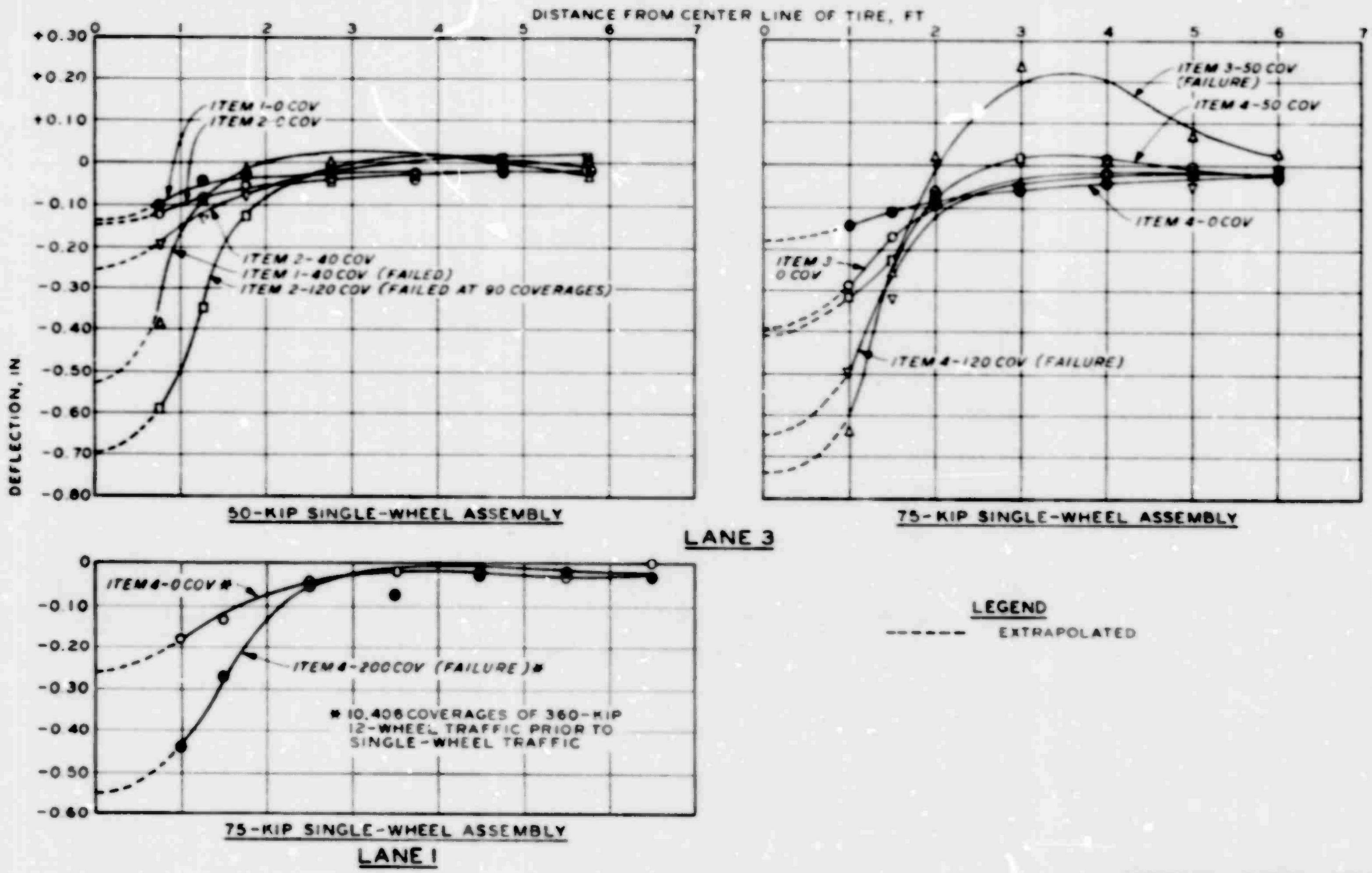


TYPICAL CROSS SECTIONS
LANE 2
200-KIP TWIN-TANDEM ASSEMBLY

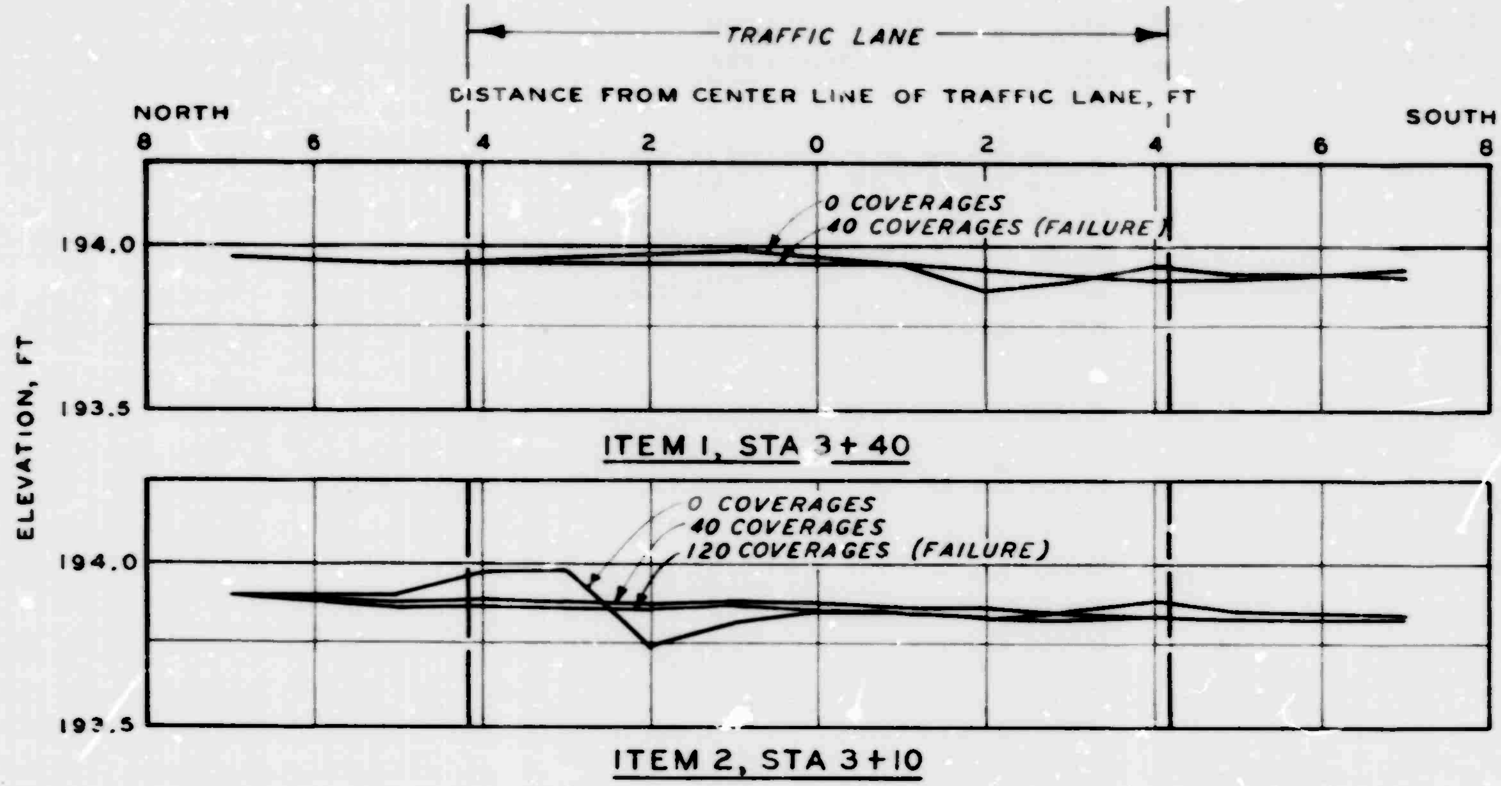
PLATE B12



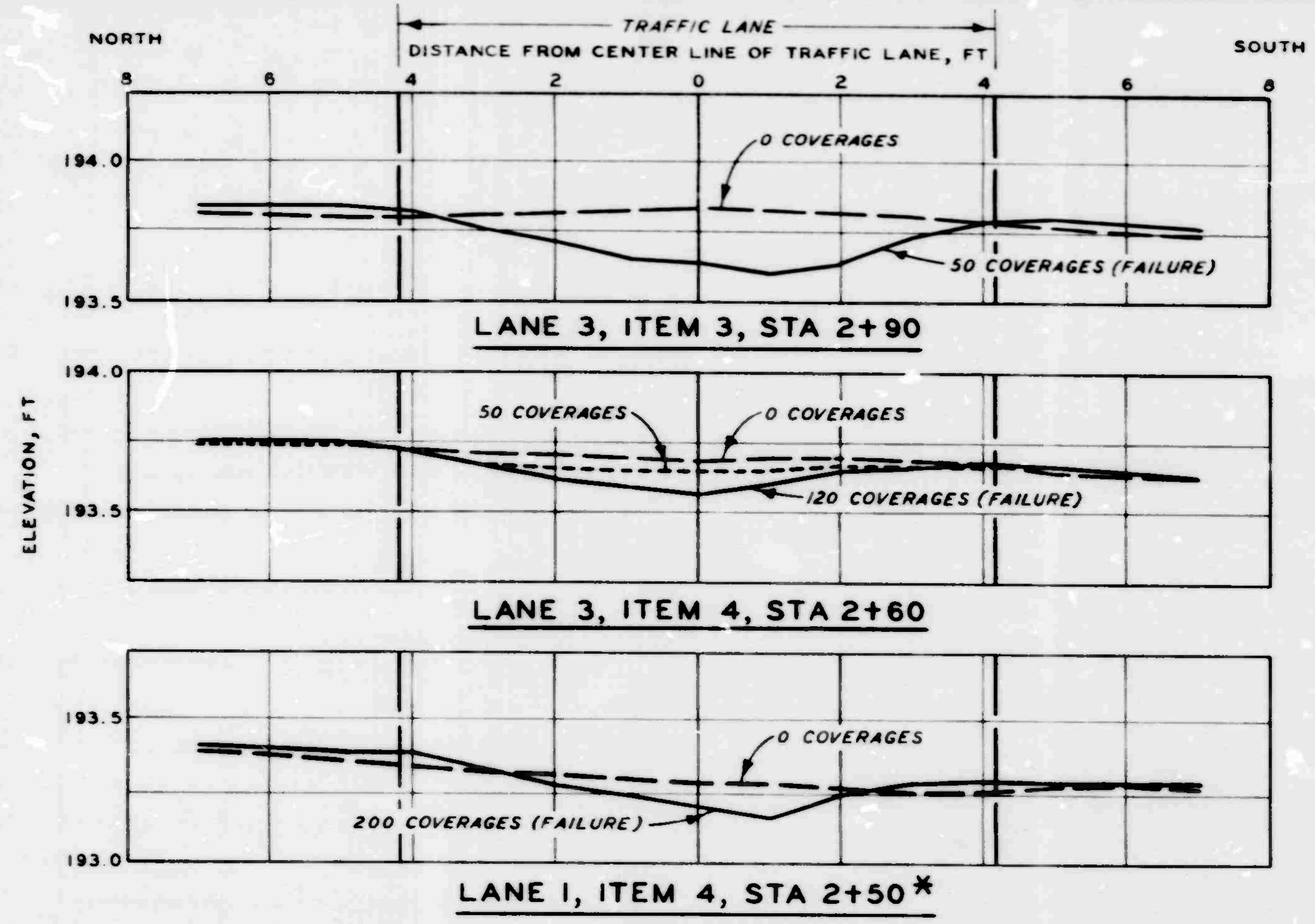
TEST PIT PROFILE
ITEM 3
STA 2+90



TOTAL DEFLECTION TRANSVERSE TO TRAFFIC SINGLE-WHEEL ASSEMBLY



TYPICAL CROSS SECTION
LANE 3
50-KIP SINGLE-WHEEL ASSEMBLY



NOTE: * 10,408 COVERAGES OF 360-KIP 12-WHEEL TRAFFIC WERE APPLIED PRIOR TO THE 75-KIP SINGLE-WHEEL TRAFFIC.

TYPICAL CROSS SECTIONS
75-KIP SINGLE-WHEEL ASSEMBLY