Full-Scale Tests on Rubblized Pavement Test Items at the National Airport Pavement Test Facility

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16. Abstract

Rubblization is defined as the process of fracturing existing concrete pavement in-place into smaller interconnected pieces. Rubblization breaks the slab action and minimizes or prevents the occurrence of reflective cracks in the asphalt concrete overlay. This is the first study conducted on the full-scale accelerated pavement testing of rubblized concrete pavements with hot-mix asphalt overlay under heavy aircraft loading.

Full-scale traffic tests were completed on three rubblized and nonrubblized rigid airport pavements overlaid with 5 inches of hot-mix asphalt at the Federal Aviation Administration National Airport Pavement Test Facility. Initially, the overlaid pavements were trafficked with a four-wheel landing gear (with wander) and a 55,000-lb wheel load. No significant distresses were observed during the first 5000 passes. The wheel load was then increased to 65,000 lb and a six-wheel landing gear was used. Test item MRC (rubblized concrete pavement on conventional base) exhibited complete structural failure. Test item MRG (rubblized concrete on grade) was suffering severe structural deterioration at the end of trafficking but retained sufficient structural capacity to support the applied load. Test item MRS (rubblized concrete over econocrete base) did not exhibit severe structural deterioration at the end of trafficking. Four trenches were excavated perpendicular to the centerline of the test items to conduct posttraffic investigation into the failure mechanism of the pavement structure. The trenching included tests for layer characterization (plate load tests, California Bearing Ratio tests, in situ densities, moisture contents, layer profile measurements, and visual evaluations) and removal of each of the pavement layers to reveal the subgrade interface and subsequent subgrade layers below.

This report summarizes the results from pavement layer characterization tests, pavement structure uniformity from heavy-weight deflectometer tests, pavement performance during the traffic tests, changes in the modulus of the rubblized concrete layer with deterioration in pavement structure backcalculated using BAKFAA. The report also summarizes the results from the posttraffic tests and provides some insight into the failure mechanism of rubblized concrete airport pavements.

17. Key Words

Rubblization, Traffic tests, Posttraffic tests, Overlay, Subgrade, Modulus

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LIST OF SYMBOLS AND ACRONYMS

D0	Peak center deflection
D2	Deflection at 12-inch offset from the center of load plate
D3	Deflection at 24-inch offset from the center of load plate
D4	Deflection at 36-inch offset from the center of load plate
D5	Deflection at 48-inch offset from the center of load plate
D6	Deflection at 60-inch offset from the center of load plate
D7	Deflection at 72-inch offset from the center of load plate
AC	Advisory Circular
AREA	Area of deflection basin normalized with D0
CBR	California Bearing Ratio
CC	Construction cycle
COV	Coefficient of variation
EB	Engineering Brief
FAA	Federal Aviation Administration
HMA	Hot-mix asphalt
HWD	Heavy-weight deflectometer
ISM	Impulse stiffness modulus
NAPTF	National Airport Pavement Test Facility
NE	Northeast
NW	Northwest
PCC	Portland cement concrete
PSPA	Portable Seismic Properties Analyzer
SW	Southwest

EXECUTIVE SUMMARY

Hot-mix asphalt (HMA) overlays placed over existing deteriorated Portland cement concrete pavements reflect the joints and cracks present in the underlying concrete pavement. Once reflected into the HMA overlay, these cracks represent a major maintenance concern and are a known source of foreign object damage. According to the National Asphalt Pavement Association, rubblization is the most effective procedure for addressing reflective cracking in HMA overlays. Rubblization is fast becoming a popular method of concrete pavement rehabilitation. The rubblization process breaks the slab action and minimizes or prevents the occurrence of reflective cracks in the HMA overlay. This is the first study conducted on the full-scale accelerated pavement testing of rubblized concrete pavements with HMA overlay under heavy aircraft loading.

Full-scale traffic tests were completed on three rubblized and nonrubblized rigid airport pavements that were overlaid with 5 inches of HMA at the Federal Aviation Administration (FAA) National Airport Pavement Test Facility. Initially, the overlaid pavements were trafficked with a four-wheel landing gear configuration (with wander) and a 55,000-lb wheel load. Straightedge rut depth measurements and transverse profile measurements were made at regular intervals during the traffic tests. No significant distresses were observed during the 5000 passes. The wheel load was then increased to 65,000 lb and a six-wheel landing gear was used. Test item MRC (rubblized concrete on conventional base and medium strength subgrade) exhibited complete structural failure. Test item MRG (rubblized concrete on medium strength subgrade) suffered severe structural deterioration at the end of trafficking but retained sufficient structural capacity to support the applied load. Test item MRS (rubblized concrete over econocrete base and medium strength subgrade) did not exhibit severe structural deterioration at the end of trafficking despite having accumulated significant levels of rutting and shear flow in the asphalt. Four trenches were opened perpendicular to the centerline of the test items to conduct a posttraffic investigation into the failure mechanism of the pavement structure. The trenching included tests for layer characterization and removal of each pavement layer to reveal the subgrade interface and subsequent subgrade layers below. Tests conducted on the pavement component layers included plate load tests, CBR (California Bearing Ratio) tests, in situ densities, moisture contents, layer profile measurements, and visual evaluations.

The results of the posttraffic tests provided insight into the failure mechanism of rubblized concrete pavements. The results indicate that the assumptions for design in FAA Engineering Brief-66 are overly conservative. For commercial airports serving wide-body aircraft (gross weights >100,000 lb), per FAA Advisory Circular 150/5320-6D, rigid pavements are required to have a stabilized base. MRS is the most representative of pavement structures that are encountered on commercial airports in the U.S. The performance of MRS suggests that rubblized concrete pavements with HMA overlay are a viable option for commercial airports. The presence of a stabilized base underneath the rubblized concrete layer limits the vertical deflection in the layer below the rubblized concrete layer and helps to keep the rubblized pieces tightly interlocked.

INTRODUCTION

Asphalt overlays placed over existing Portland cement concrete (PCC) pavements reflect the joints and cracks present in the underlying concrete pavement. Once reflected into the asphalt overlay, these cracks represent a major maintenance concern and are a known source of foreign object damage. Concrete pavements exhibiting distresses such as cracking, joint deterioration, spalling, and joint faulting can be rehabilitated by constructing a concrete or asphalt overlay. The rubblization process breaks the slab action and minimizes or prevents the occurrence of reflective cracks in the HMA overlay. Rubblization is fast becoming a popular method of concrete pavement rehabilitation. According to the National Asphalt Pavement Association, rubblization is the most effective procedure for addressing reflective cracking in asphalt overlays. Rubblization could be a cost-effective means of converting an existing failed or failing concrete pavement into a superior base, thereby eliminating the expense of removal and replacement. The rubblized concrete layer behaves as a tightly keyed, interlocked, high-density unbound base. A number of airfield projects have used rubblization as a pavement rehabilitation technique. The projects range from heavy-load military airfields to local general aviation airfields. The Federal Aviation Administration (FAA) currently does not have a thickness design standard for hot-mix asphalt (HMA) overlays over rubblized concrete pavements. Engineering Brief (EB)-66 [1] summarizes the guidelines for rubblized PCC base courses. These guidelines are based on industry experience. The brief provides interim guidance, but full-scale tests are needed to develop design standards for using this technology at airports under heavy aircraft loading.

To study the performance of rubblized concrete pavements with HMA overlay under heavy aircraft loading, three rigid airport pavement test items (north of pavement centerline) at the FAA National Airport Pavement Test Facility (NAPTF) were rubblized with a resonant pavement breaker and overlaid with 5 inches of P-401 HMA. Pavements to the south of centerline were nonrubblized. Three test items (MRC, MRG, and MRS) had 12-inch-thick concrete slabs on different support systems (slab on crushed stone base, slab on grade, and slab on stabilized base). The rigid pavements had been trafficked to complete failure, prior to rubblization, using dual-tandem and triple-dual-tandem landing gear configurations at wheel loads of 55,000 lb. All three test items were constructed on medium strength (California Bearing Ratio (CBR)≈7-8) clay subgrades. This is the first study to be conducted on rubblized concrete pavements with HMA overlay under heavy aircraft loading using full-scale, accelerated pavement tests.

Heavy-weight deflectometer (HWD) tests were performed using the FAA Kuab HWD equipment on a 10-foot grid to study the uniformity of the pavement structures. The results showed that the pavement structure within a test item (for all rubblized test items) was fairly uniform. After the completion of uniformity tests, the overlaid pavements were subjected to full-scale accelerated traffic tests to complete structural failure. The traffic tests started with a four-wheel landing gear configuration (with wander) and 55,000-lb wheel load. Straightedge rut depth measurements and transverse profile measurements were made at regular intervals during the traffic tests. No significant distresses were observed for 5000 passes. The wheel load was then increased to 65,000 lb and a six-wheel landing gear was used. HWD tests were routinely performed at three different load levels: 12,000, 24,000, and 36,000 lb. A Portable Seismic

Properties Analyzer (PSPA) was used in conjunction with the HWD to estimate the asphalt concrete modulus. Moduli for the rubblized concrete layer were backcalculated using the FAA BAKFAA software. After the completion of traffic tests, four trenches were excavated perpendicular to the centerline of the test items to conduct posttraffic investigation into the failure mechanism of the pavement structure. The trenching activities included testing for layer characterization and removal of each pavement layer to reveal the subgrade interface and subsequent subgrade layers below. Tests conducted on the pavement component layers included plate load tests, CBRs, in situ densities, moisture contents, layer profile measurements, and visual evaluations.

This report summarizes the results from

- pavement layer characterization tests.
- pavement structure uniformity from HWD tests.
- pavement performance during the traffic tests.
- changes in the modulus of the rubblized concrete layer with deterioration in pavement structure backcalculated using BAKFAA.
- posttraffic tests and provides some insight into the failure mechanism of rubblized concrete airport pavements.

RUBBLIZATION OF PCC PAVEMENTS AT THE NAPTF.

NATIONAL AIRPORT PAVEMENT TEST FACILITY. The NAPTF is located at the FAA William J. Hughes Technical Center, Atlantic City International Airport, New Jersey. The primary purpose of NAPTF is to generate full-scale pavement response and performance data for development and verification of airport pavement design criteria. It is a joint venture between the FAA and the Boeing Company and became operational on April 12, 1999. The test facility consists of a 900-ft-long by 60-ft-wide test pavement area, embedded pavement instrumentation and a dynamic data acquisition system, environmental instrumentation and a static data acquisition system, and a test vehicle for loading the test pavement with up to 12 aircraft tires at wheel loads of up to 75,000 lb. Additional information about the test facility is available at http://www.airporttech.tc.faa.gov.

The NAPTF construction cycle (CC) includes new pavement construction, including instrumentation; traffic tests to failure; posttraffic tests that includes trenching activities and other tests; and pavement removal. Figure 1 shows the construction cycle.

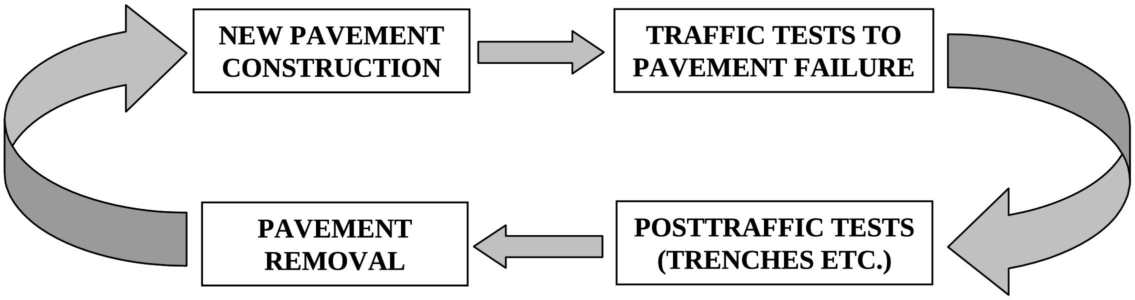


Figure 1. Construction Cycle at the NAPTF

PAVEMENT STRUCTURES. The three original rigid pavement test items to be rubblized were constructed and tested during construction cycle two (CC-2) at the NAPTF. Each test item was 75 feet long by 60 feet wide with 30, 15- by 15-foot by 12-inch-thick concrete slabs. The first test item (MRG—medium strength subgrade, rigid pavement, on grade) was built directly on the subgrade, the second (MRC—medium strength subgrade, rigid pavement, on conventional aggregate base) was built on a 10-inch-thick crushed aggregate subbase on top of the subgrade, and the third (MRS—medium strength subgrade, rigid pavement, on stabilized base) was built on a 6-inch econocrete subbase over a 6-inch crushed aggregate subbase. Each test item was separated into two 30-foot-wide traffic lanes, north and south. Construction was completed in April 2004, and the traffic tests were completed in December 2004. Posttraffic testing included the excavation of four test pits, approximately 5 feet wide by 5 feet long, and extending down into the subgrade. One test pit was opened in the south traffic lane of each test item and one was opened in the north traffic lane of test item MRC. Detailed information on the design and construction characteristics of the pavement structures can be found in reference 2. The structural condition index of all the rigid pavement test items, in both traffic lanes, was less than 20. However, most of the cracks were tight, with none rated worse than low severity. Also, both the transverse and the longitudinal joints were formed and doweled.

In January 2005, all the concrete slabs in the north traffic lane, including those in the transition sections, were rubblized with an RMI RB-500 resonant breaker operating at 44 Hz. In June 2005, the rubblized pavement was lightly wetted, rolled with a vibratory steel drum roller, and overlaid with 5 inches of P-401 HMA (two 2.5-inch-thick lifts). Figures 2 through 5 show, respectively, the vibrating foot of the resonant breaker, the rubblized surface being rolled, the test pavement surface after rubblization, and the test pavement surface after the overlay was applied (with the HWD equipment in position for uniformity tests).

Figure 2. Rubblizing the North Traffic Lane With the Resonant Breaker

Figure 3. Rolling the Rubblized Pavement

Figure 4. Rubblized on the Left (North), Nonrubblized on the Right (South)

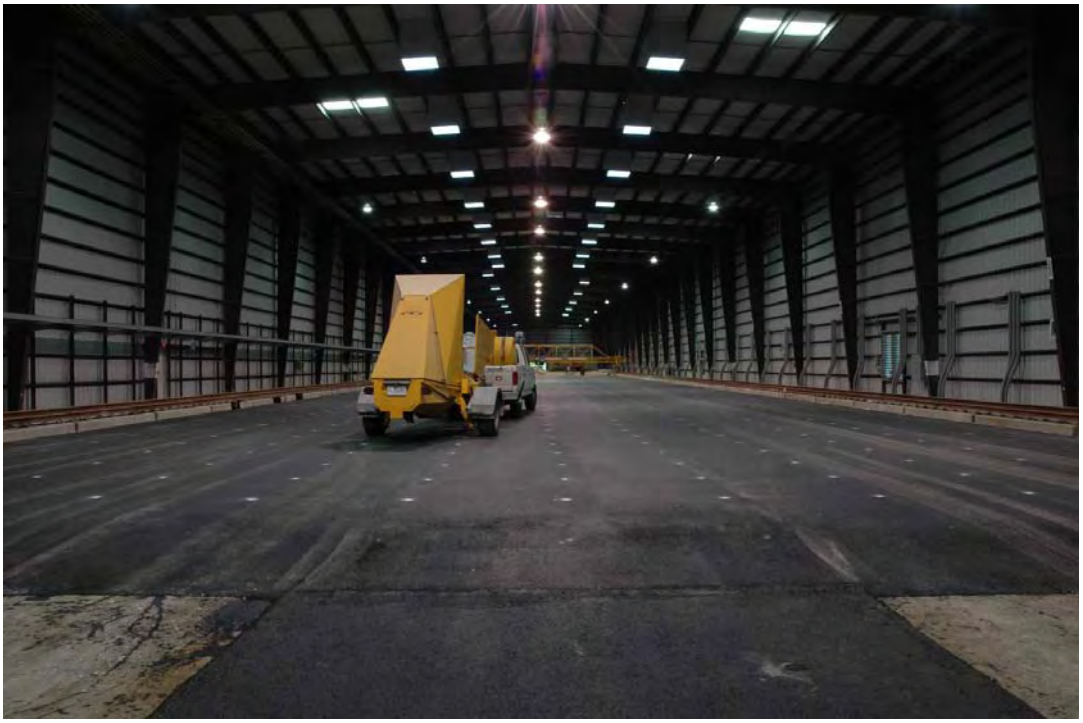


Figure 5. After Asphalt Overlay, With HWD Equipment in Foreground

Figure 6 shows the pavement cross sections after the placement of the HMA overlay.

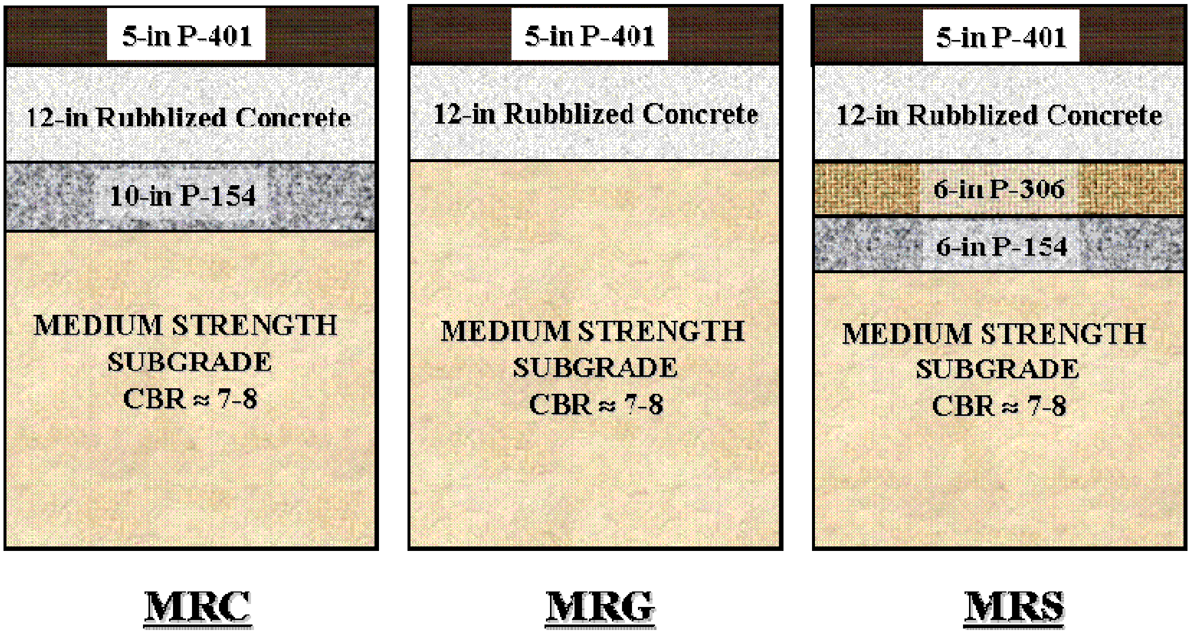


Figure 6. The CC-2 Overlay Pavement Test Items

P-401, P-306, and P-154 are FAA standard specifications for Plant Mix Bituminous Pavement, Econocrete Base Course, and Subbase Course (crushed aggregate screenings were used at NAPTF), respectively.

<u>TEST PITS IN RUBBLIZED TEST ITEMS</u>. After the three test items were rubblized, a 4-foot-long by 4-foot-wide test pit was saw-cut in each test item for visual examination of the rubblized concrete (extent of fractures from rubblization process, particle sizes, etc.). Figure 7 shows fracture patterns and particle sizes in test items MRC, MRG, and MRS, respectively. In general, the top 2 to 3 inches in all the test items were rubblized into dust and stones with a top particle size of 1 inch (figure 7). The particle size in the bottom 9 inches ranged from 4 to 15 inches with larger particle sizes in MRS. The test pits showed that the rubblization process induced cracks/fractures for the entire depth of the slabs and that the cracks were tightly held.

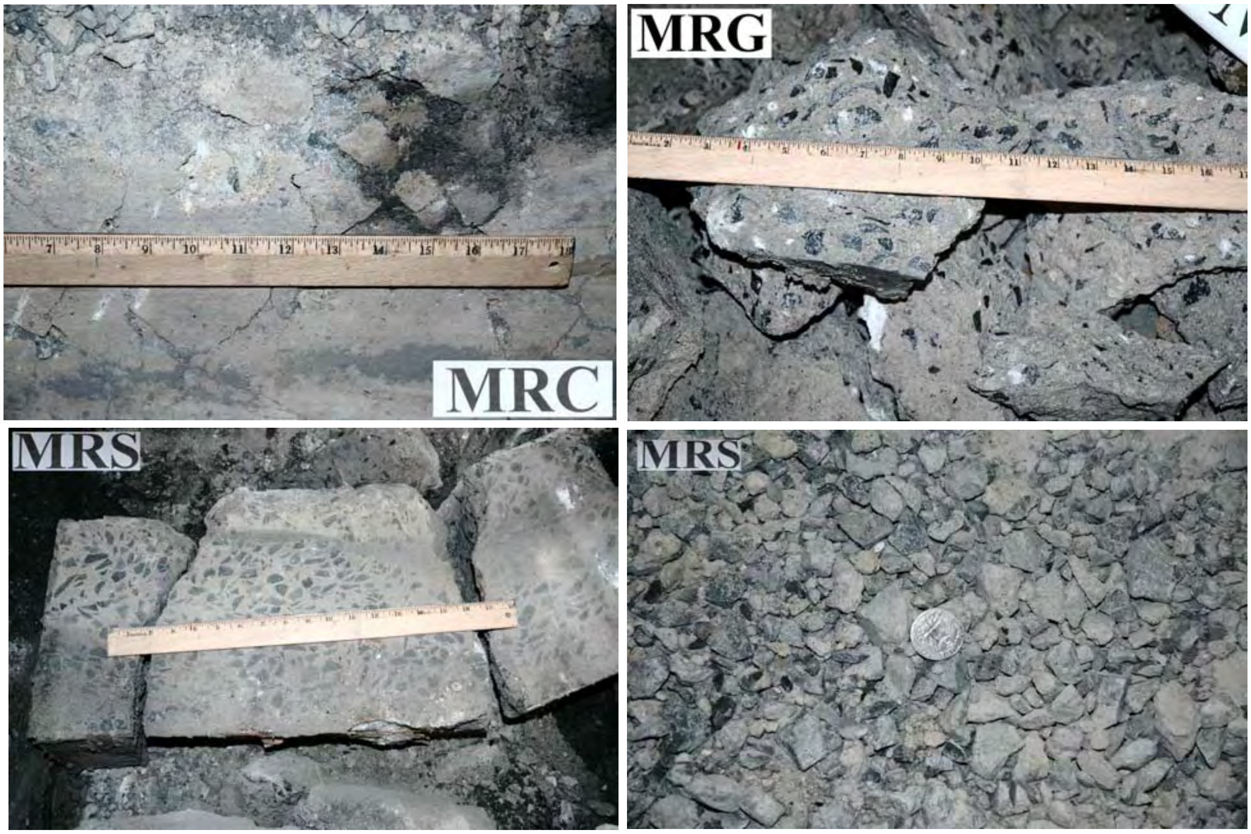


Figure 7. Visual Observations From the Test Pits in Rubblized Test Items

UNIFORMITY OF PAVEMENT STRUCTURES. HWD tests were performed using the FAA Kuab HWD equipment on a 10-foot grid to study the uniformity of the pavement structures, see figure 5. Tests were performed with a 12-inch-diameter plate at three different load levels: 12,000, 24,000, and 36,000 lb. The results showed that the pavement structure within each test item was fairly uniform. For peak center deflection (D0), the coefficient of variation (COV) ranged between 20 and 25 percent. For deflection D7 (at 72-inch offset from the center of load plate, and an indicator of subgrade condition), the COVs were approximately 10 percent. Figure 8 shows that the mean D0s for the rubblized test items were larger than the D0s for the nonrubblized test items. Also, among the rubblized test items, MRC showed the highest deflections, followed by MRG, and then by MRS. This order was counter to expectations because MRC had a crushed aggregate subbase course and would normally be expected to be of higher stiffness than the MRG pavement built directly on the subgrade. Pretraffic measurements of subgrade strength in the test pits showed that water had migrated from the crushed aggregate subbase into the subgrade of MRC and softened approximately the top 3 inches of the subgrade. The subgrade surface in the MRC test pits had strength of approximately 4 CBR, whereas the strength 1 foot below the surface was approximately 8 CBR. The subgrade surface in the MRG and MRS test pits ranged from 7 to 8 CBR, as constructed. The order of failure, discussed in the next section, also followed the order of the HWD deflection magnitudes.

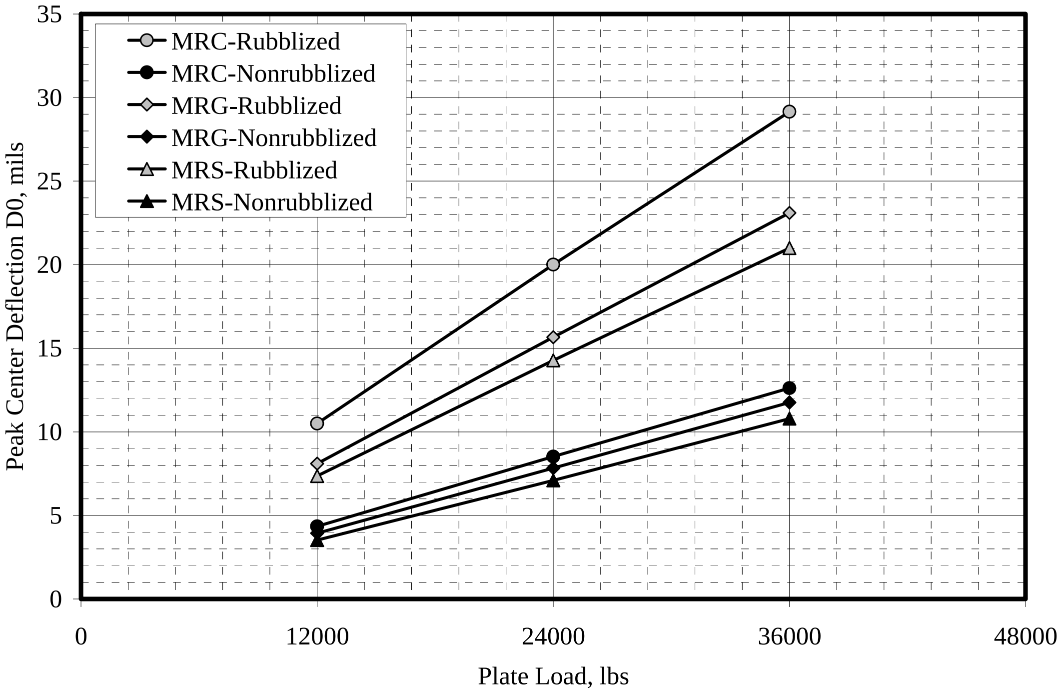


Figure 8. Mean Peak D0s From Uniformity Tests

Figure 9 shows deflection D7 (at 72-inch offset from center of plate) that is indicative of subgrade stiffness. Figure 9 is further indication that the subgrade of MRC was of lower stiffness than the subgrade of MRG and MRS.

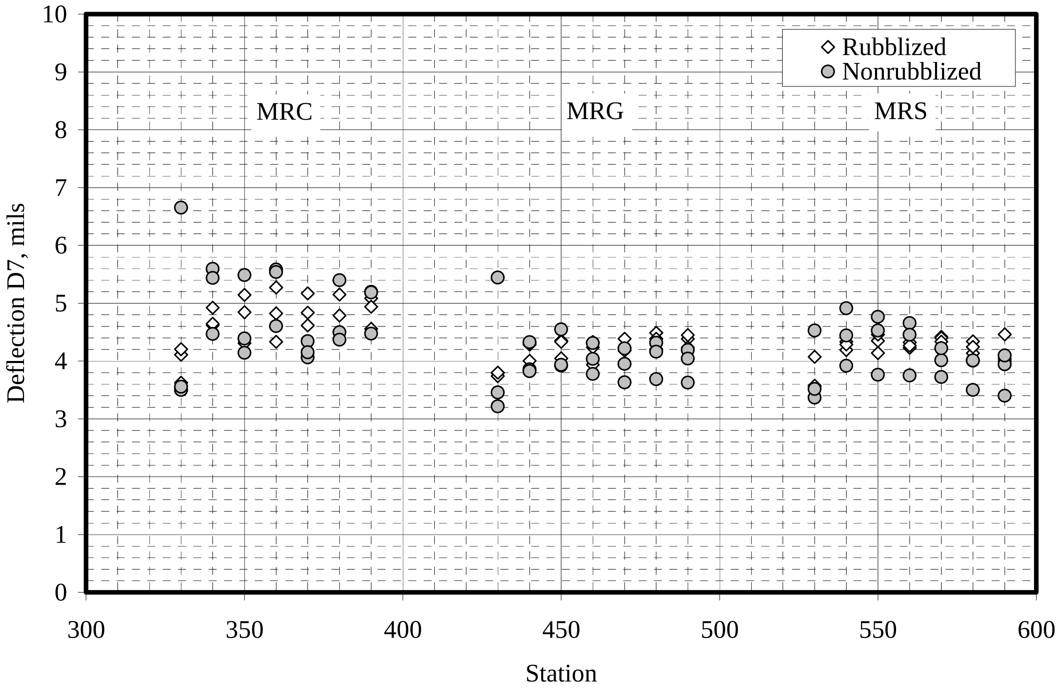


Figure 9. Deflections D7 From Uniformity Tests

AREA is the area of deflection basin normalized with D0 and is a deflection basin shape factor [3]. Figure 10 shows the AREA for rubblized and nonrubblized test items. The magnitude of the AREA term is a fairly good indicator of layer behavior (bound or unbound). Higher AREA values indicate bound material, and lower AREA values indicate unbound material.

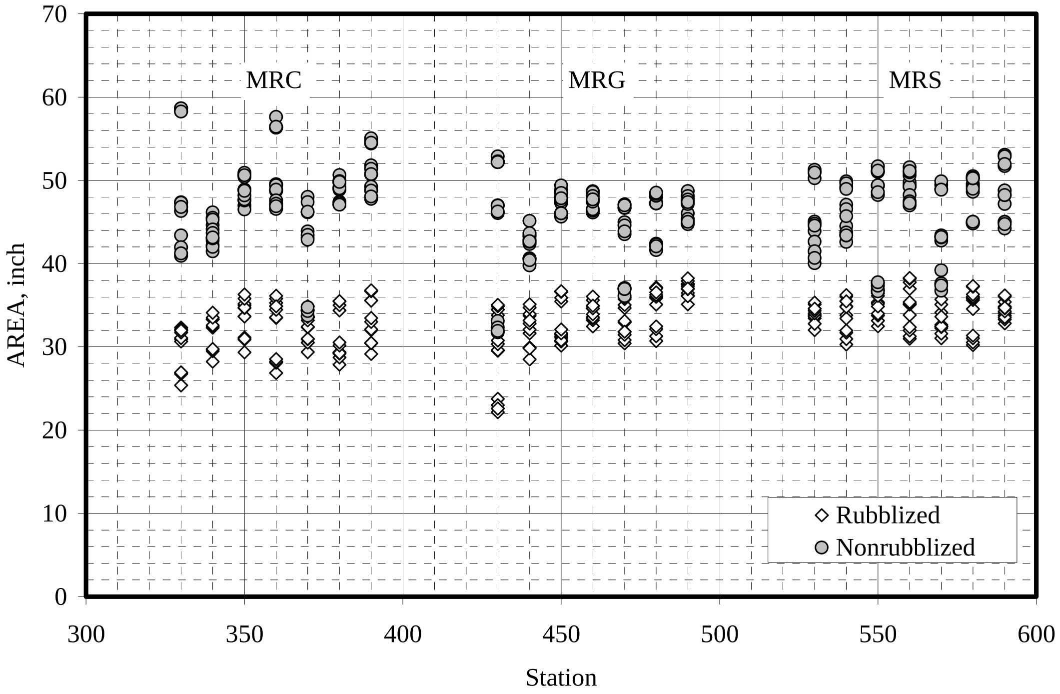


Figure 10. The AREA From Uniformity Tests

The rubblized test items show lower AREA values (mean AREA = 33.2 inches) compared to nonrubblized test items (mean AREA = 46.6 inches).

FULL-SCALE TRAFFIC TESTS OF CC-2 OVERLAY TEST ITEMS.

Rubblizing concrete pavements is a relatively new technique, and full-scale traffic tests of rubblized airport pavements under heavy airplane loading had not been conducted up to now. Design procedures for determining the required thickness of asphalt overlays on rubblized pavements, therefore, have not been developed in the traditional sense. The common assumption is that the rubblized and overlaid pavement behaves like a flexible pavement and that the overlay thickness can be determined by assigning an equivalent thickness or modulus value to the rubblized layer and applying this in a standard flexible pavement design procedure [1]. Since there are no established performance prediction models for rubblized pavements, it was decided to start the tests at an arbitrary loading condition and to adjust the loading according to the observed behavior under traffic.

Only four wheels were available for loading on the nonrubblized traffic lane, so traffic began with a four-wheel, dual-tandem configuration (figure 11) on both traffic lanes. The geometry was the same on both traffic lanes, with dual spacing of 54 inches and tandem spacing of

57 inches. Wheel load was set at 55,000 lb because this was the load applied to the new construction CC-2 test items and, although badly cracked at the end of trafficking, all the test items were capable of structurally supporting the loads applied up to the end of trafficking. Adding 5 inches of asphalt implied that the nonrubblized pavement would be capable of structurally supporting considerably more traffic at the same load. Calculations of the predicted life of the rubblized pavements using the assumptions of flexible pavement response and characteristics indicated that, for the initial traffic loading case, the structure on grade (MRG) might fail fairly quickly, somewhere between a few hundred and a couple thousand repetitions. The structure on stabilized base would probably last for many tens of thousands of repetitions. The standard NAPTF 66 repetitions per cycle wander pattern (figure 12) were used on both traffic lanes.

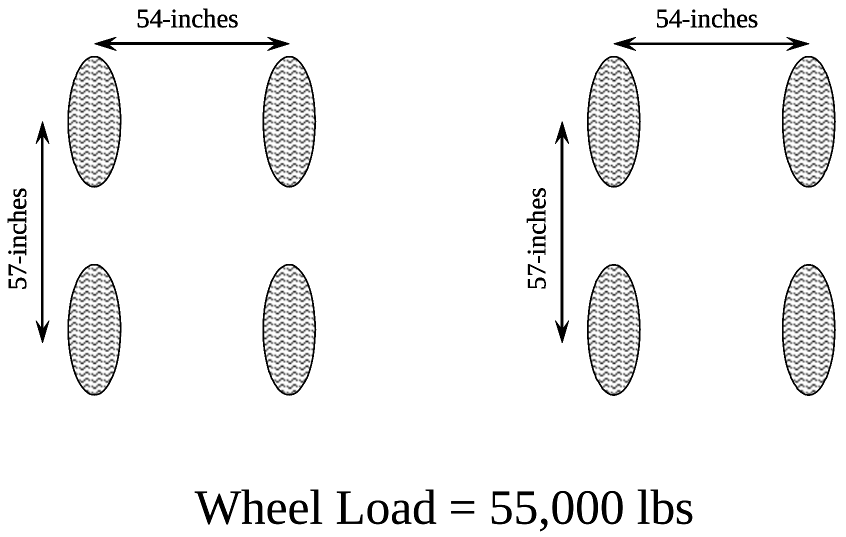
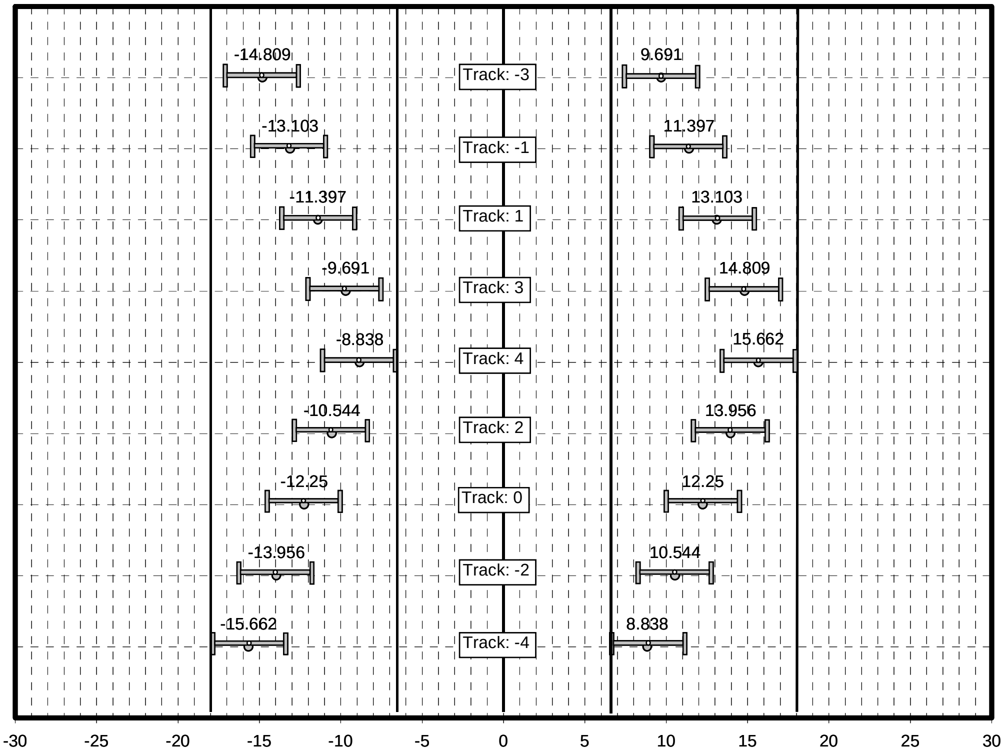


Figure 11. The Four-Wheel Landing Gear Configuration Used on Both North and South Wheel Tracks

Traffic tests were continued until either structural failure (1-inch surface upheaval outside the traffic path) was deemed to have occurred or until it was estimated that failure was unlikely to occur within a reasonable number of passes at the applied load. During the traffic tests, the pavements were monitored through a combination of visual surveys and nondestructive tests, including periodic straightedge rut depth measurements, surface profile measurements, and HWD deflection measurements. Cores were also extracted from the asphalt to monitor asphalt thickness and crack propagation. Data processing and analysis of the surface profile and HWD measurements is time-consuming, and the primary means of monitoring pavement performance as the trafficking progressed was from plots of the straightedge rut depth measurements prepared immediately after the measurements had been taken. A 16-foot-long straightedge was used for rut depth measurements. In each test item, the rut depth measurements and profile measurements were made at two different longitudinal positions located at one-third and two-thirds the distance into the test item. These locations were designated as NW (northwest) and NE (northeast) for the rubblized test items and SW (southwest) and SE (southeast) for the nonrubblized test items (N and S stand for north side and south side of the longitudinal centerline, respectively).



			63 & 64	65 & 66	61 & 62			
		51 & 52	59 & 60	53 & 54	57 & 58	55 & 56		
	43 & 44	45 & 46	41 & 42	47 & 48	39 & 40	49 & 50	37 & 38	
19 & 20	35 & 36	21 & 22	33 & 34	23 & 24	31 & 32	25 & 26	29 & 30	27 &28
1 & 2	17 & 18	3 & 4	15 & 16	5 & 6	13 & 14	7 & 8	11 & 12	9 & 10
-4	-3	-2	-1	0	1	2	3	4

Figure 12. Wander Pattern Used for Traffic Tests

Figures 13 through 15 show the straightedge rut depth measurements for test items MRC, MRG, and MRS, respectively. Traffic tests started on July 7, 2005, and continued until October 6, 2005, following the schedule in table 1. The temperature of the asphalt varied between 66° and 85°F (19° and 29°C) during the test period. The average temperature of the asphalt was approximately 78°F (26°C).

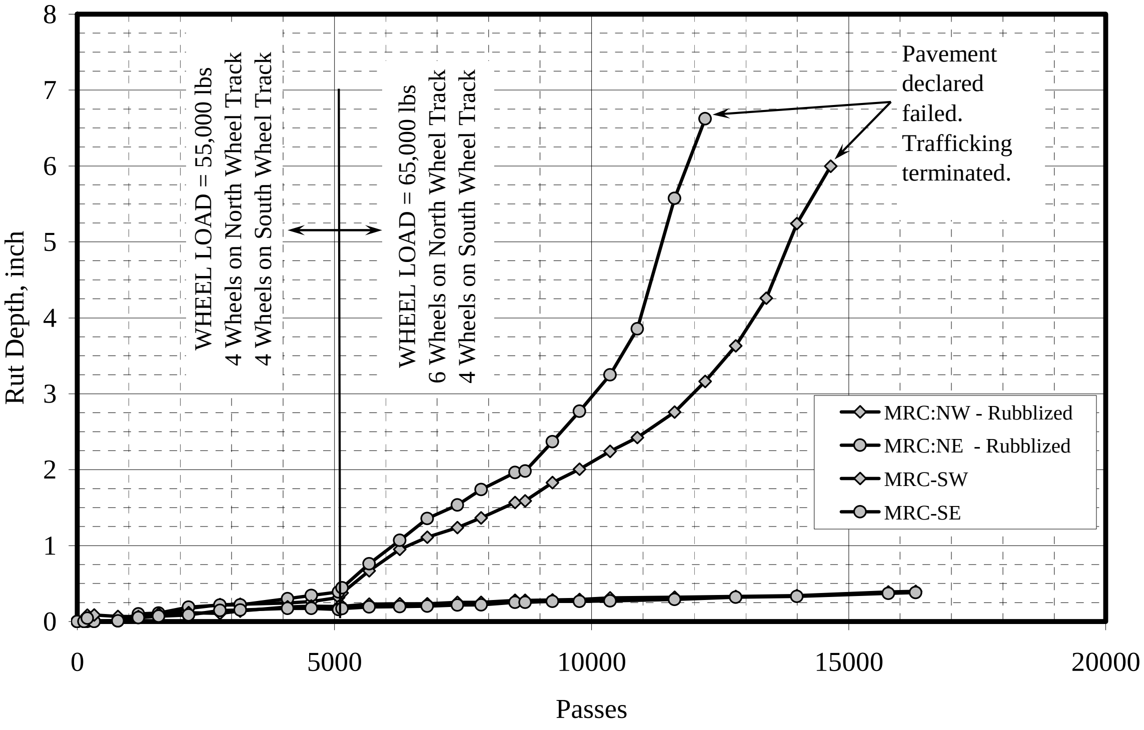


Figure 13. Straightedge Rut Depth Measurements in the Test Item MRC

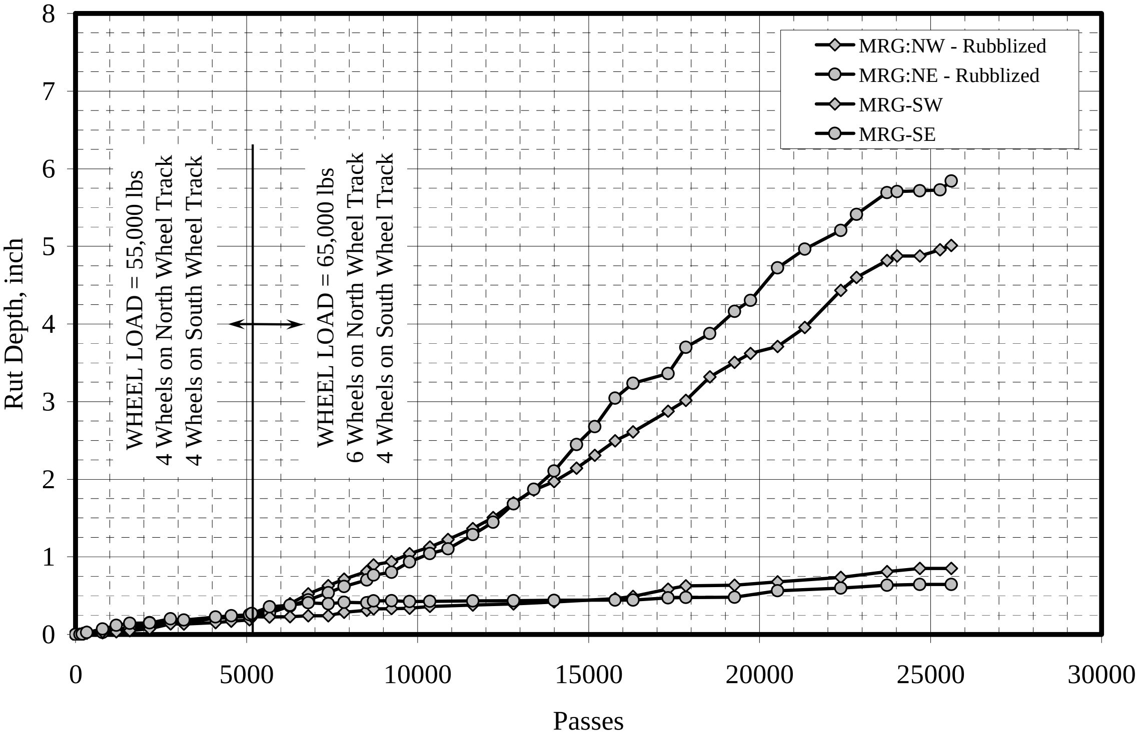


Figure 14. Straightedge Rut Depth Measurements in the Test Item MRG

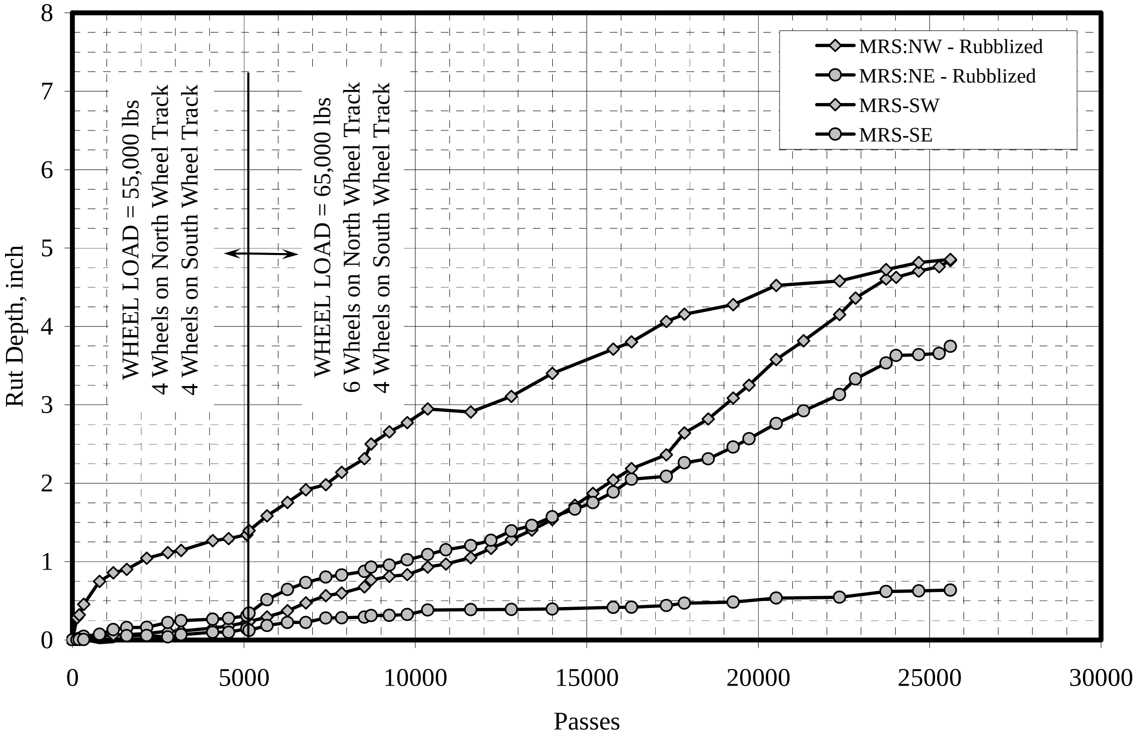


Figure 15. Straightedge Rut Depth Measurements in the Test Item MRS

Table 1. Traffic Schedule for CC-2 Overlay Test Items

Dates (from-to)	Repetitions (from-to)	Test Items Trafficked	Load on North Lane*	Load on South Lane*
07/07/05-07/25/05	1-5,082	MRG-N, MRC-N, MRS-N MRG-S, MRC-S, MRS-S	four-wheel, 55,000 lb	four-wheel, 55,000 lb
07/26/05-08/12/05	5,083-11,814	MRG-N, MRC-N, MRS-N MRG-S, MRC-S, MRS-S	six-wheel, 65,000 lb	four-wheel, 65,000 lb
08/15/05-08/18/05	11,814-14,256	MRG-N, MRC-NW**, MRS-N, MRG-S, MRC-S, MRS-S	six-wheel, 65,000 lb	four-wheel, 65,000 lb
08/19/05-08/24/05	14,257-16,302	MRG-N, MRS-N MRG-S, MRC-S, MRS-S	six-wheel, 65,000 lb	four-wheel, 65,000 lb
09/13/05-10/06/05	16,303-25,608	MRG-N, MRS-N MRG-S, MRS-S	six-wheel, 65,000 lb	four-wheel, 65,000 lb

* Cold, unloaded tire pressures: 220 psi at 55,000 lb and 260 psi at 65,000 lb

** After the localized failure in MRC-NE (northeast portion of the test item), only the northwest portion (MRC-NW) of the test item was trafficked.

Except at the MRS-SW location, all test items showed similar rut depths during the first 5,082 passes (55,000-lb wheel load, four-wheel landing gear configuration). In particular, there was no discernible difference between the performance of the rubblized and nonrubblized pavements. It was also visually observed that the surface deflections of the rubblized pavements under load were negligible, and the response of the rubblized pavements appeared to be very similar to the nonrubblized pavements. Instrumentation was not installed in the pavements to measure surface deflections, so this observation cannot be verified to any degree of accuracy. But the surface deflection of a flexible pavement under load can be visually observed without any magnifying aid. The surface deflection of a rigid pavement cannot be visually observed. Therefore, it was decided that the load should be increased to the largest extent practically allowed by the test vehicle loading system and tires to increase the possibility of inducing significant distress in the rubblized pavements. From 5083 passes to the end of traffic tests, six-wheel, triple-dual-tandem loading at 65,000 lb was applied to the rubblized pavement, and four-wheel, dual-tandem loading at 65,000 lb was applied to the nonrubblized pavements. Both the six- and four-wheel configurations at increased loading had the same dual and tandem spacings of 54 and 57 inches, respectively (figure 16).

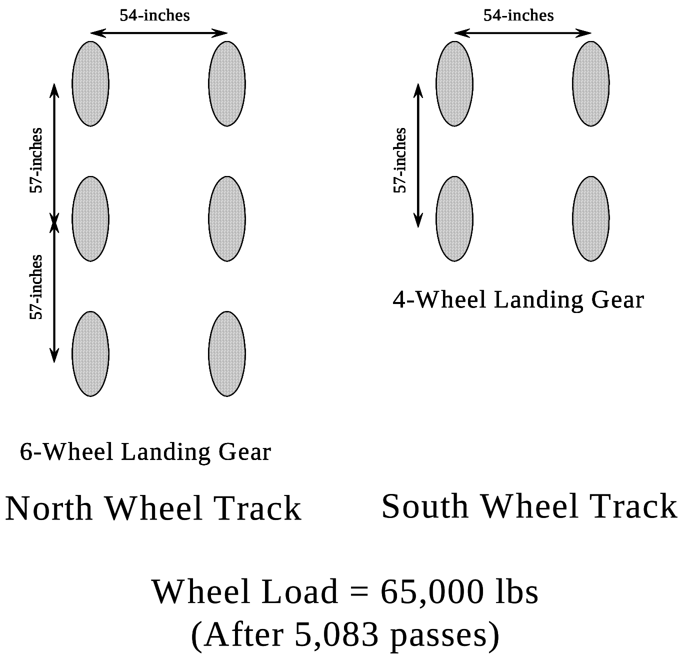


Figure 16. Landing Gear Configurations Used for Traffic Tests After 5083 Passes

At the MRS-SW location (nonrubblized), a test pit (5′ x 4′) was opened in the concrete slab (for subgrade evaluation) prior to placing the HMA overlay. The concrete that was used to refill the test pit was severely broken and a depression formed at this location during the placement and compaction of the HMA overlay. This severely weak area caused significant local accumulation of rutting early in the traffic tests.

After approximately 10,000 passes in MRC, 13,000 passes in MRG, and 15,000 passes in MRS, significant upheaval in the HMA layer at the longitudinal joints just outside the traffic path was observed in the rubblized test items. After this number of passes, the rut depth measurements are exaggerated because the straightedge was resting on top of the upheavals outside the traffic path. More accurate rut depth measurements have been computed from the surface profile measurements (figure 17). Maximum rut depths from the transverse profiles at the end of trafficking were 4 inches on MRC-N, 2.5 inches on MRG-N, and 2 inches on MRS-N.

Significant structural upheaval was also observed outside the wheel track on MRC-N, but neither the straightedge measurement nor the transverse profile measurements could separate the contributions of the underlying structural response and the asphalt upheaval movement. Measurements of the transverse profiles of the structural layer interfaces are presented in the Layer Profile section from trench data. These measurements give a more definitive estimate of the true structural response of the rubblized pavement structures. The NE end of MRC was the first area of the rubblized pavements to show signs of failure (figure 18). This failure was not representative of the structural performance of the test item as a whole because one of the preoverlay test pits (for subgrade evaluation) was located where the pavement failed. A weakened support system resulted because the replaced subbase aggregate material could not be compacted to the same density as in the original construction. A depression in the pavement surface was observed at this location after about 400 load repetitions. The depression migrated longitudinally toward the east until it was about 15 feet long, but the structure continued to support the full traffic load until it appeared to be in danger of suffering complete structural collapse at 11,814 passes. The weakened area did not migrate back into the west half of the test item, and the declared structural life of MRC-NW of 14,256 passes is believed to be a true representation of the structural performance of the test item. Also, MRC-NW did not appear to be in danger of complete structural collapse as had MRC-NE. Trafficking in MRG and MRS was terminated after 25,608 passes. From visual inspection at the end of trafficking, MRG-N appeared to be suffering from structural upheaval outside the wheel track, but MRS-N did not.

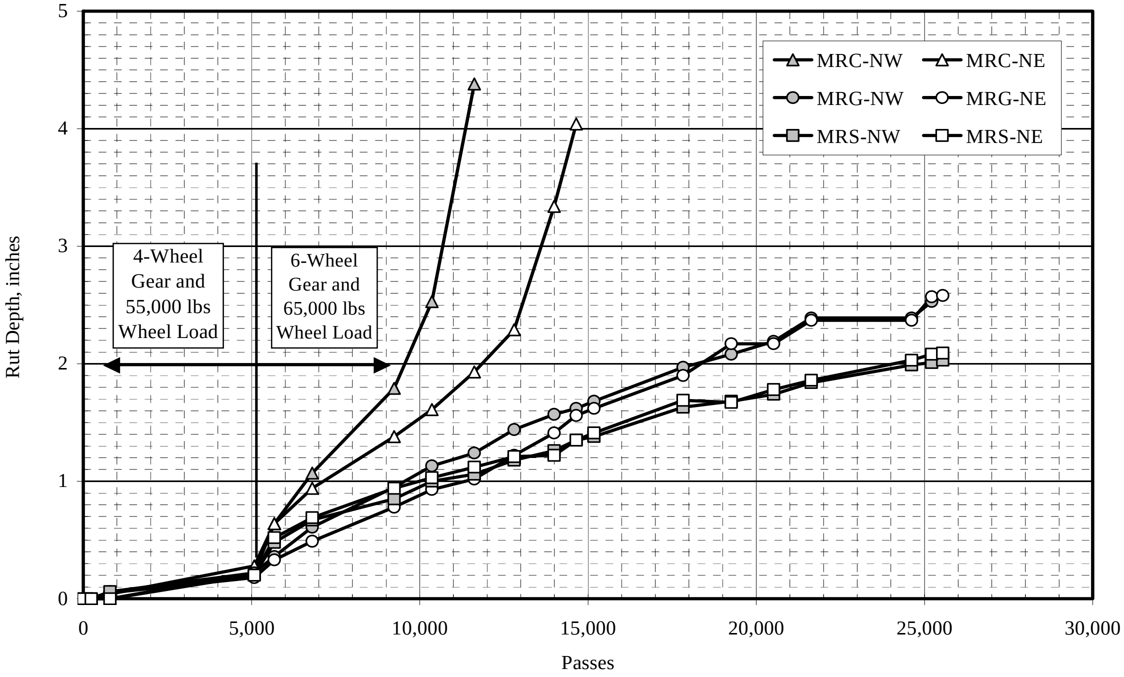


Figure 17. Rut Depths in the Rubblized Concrete Test Items From Transverse Surface Profiles

Figure 18. Pavement Failure in the East End of Rubblized Test Item MRC

POSTTRAFFIC TESTS

TRENCH LOCATIONS AND POSTTRAFFIC TESTS.

Four trenches were dug in the rubblized test items perpendicular to the centerline of test items MRC (two trenches), MRG (one trench), and MRS (one trench) at rut depth measurement location on the test pavements. The locations of the test items and the trenches are summarized in table 2.

Table 2. Posttraffic Trench Locations

Test Item	Trench	Start Station (ft)	End Station (ft)
MRC	-	325	400
MRC	MRC-W	354	364
MRC	MRC-E	374	380
MRG	-	425	500
MRG	MRG	452	458
MRS	-	525	600
MRS	MRS	552	558

The purpose of the trenches was to conduct a posttraffic investigation into the failure mechanism of the pavement structure. The trenching involved removing the P-401 HMA layer, the rubblized concrete layer, P-154 subbase and P-306 econocrete layer (in MRS), and the P-154 subbase layer (in MRC) to reveal the subgrade interface and subsequent subgrade layers below. Tests and measurements were performed on the various layers of the pavement structure. No tests were performed on the HMA layer. After removing the P-401 HMA surface, the rubblized concrete layer was exposed in all four trenches. Plate load tests (American Association of State Highway and Transportation Officials Designation: T 222-81, 2000) were performed inside and outside the traffic path on the surface of the rubblized concrete layer and visual observations were made. Removal of the rubblized concrete layer exposed the P-154 surface in the MRC trenches, the subgrade surface in the MRG trench, and the P-306 econocrete subbase surface in the MRS trench. In the MRC trenches, plate load tests, CBR, and sand cone density measurements were made on the surface of the P-154 layer. In the MRG trench (on the subgrade surface), the tests included CBRs, in situ density measurements (drive cylinder), and plate load tests. Only plate load tests were performed on top of the P-306 econocrete layer in the MRS trench. After removing the P-154 subbase in MRC, CBRs, plate load tests, and density measurements were taken on the subgrade surface. In MRS, P-306 was removed to expose the P-154 subbase surface on which plate load tests, sand cone tests, and CBRs were performed. CBRs, plate load tests, and density measurements were taken on the subgrade surface after removing the P-154 subbase. In all the trenches, CBRs and density measurements were also taken at a depth of 1 foot below the subgrade surface. After completing the tests, the trench walls were cleaned to clearly expose the layer interfaces. Measurements of the pavement layer interface profiles were taken relative to a horizontal string line to quantify the contribution of each component layer to the total pavement rutting and upheaval. CBR tests on P-154 and subgrade were in situ CBR tests.

TEST RESULTS.

The test results from different pavement layers in the four trenches are summarized in table 3. One of the significant observations relative to table 3 was made from the subgrade CBRs in the four trenches. Pretraffic/preoverlay measurements of subgrade strength in the test pits showed that water had migrated from the crushed aggregate subbase into the subgrade of MRC and softened the top 3 or so inches of the subgrade. The surface of the subgrade in the MRC test pits had strength of approximately 4 CBR, whereas the strength 1 foot below the surface was approximately 6 to 8 CBR. The MRG subgrade surface CBR was high (about 11). It is assumed that this was due to water being drawn from the subgrade (since slabs were directly cast over subgrade) by hydration of the concrete during curing. This phenomenon was not observed in MRC (slab over crushed stone base) or MRS (slab over econocrete subbase). The results from the trenches confirmed the observations/measurements from the pretraffic test pits. Also, performing any type of strength tests just on the rubblized material is very difficult (if not impossible) because of the nature of the material. In this project, plate load tests were performed on the top of the rubblized layer. Due to severe rutting in MRC, plate load tests could not be performed inside the traffic path. In test item MRG, the “k” value (modulus of subgrade reaction) from the plate load test inside the traffic path was lower (k = 322 pounds per cubic inch (pci)) than the k value from outside the traffic path test (k = 457 pci). The lower k value inside the traffic path could be the result of incipient failure in MRG. In test item MRS, the k value from the plate load test inside the traffic path was higher (k = 780 pci) than the k value from outside the traffic path test (k = 579 pci).

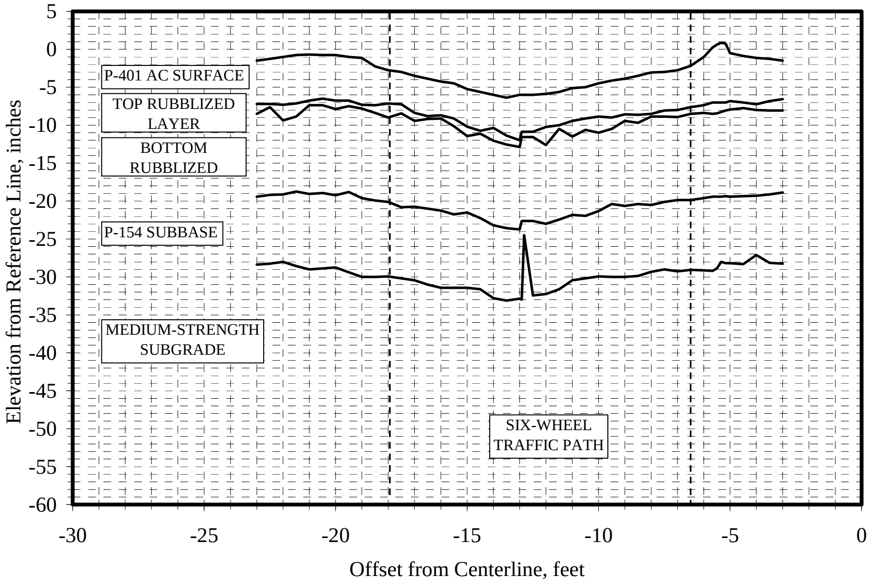
Table 3. Summary of Posttraffic Test Results

Test Item	Trench ID	Layer Type	Test Type	Test Results	
				Inside Traffic Path	Outside Traffic Path
MRC	MRC-W	Rubblized concrete	Plate load test	-	-
		P-154 subbase	Plate load test	144 pci	92 pci
			CBR	35.9	33.7
			In situ dry density	122.4 pcf	122.1 pcf
		Subgrade surface	Plate load test	-	70 pci
			CBR	4.8	4.4
			In situ dry density	89.4 pcf	88.2 pcf
		1 foot below subgrade surface	CBR	6.8	6.4
			In situ dry density	93.1 pcf	93.2 pcf
	MRC-E	Rubblized concrete	Plate load test	-	270 pci
		P-154 subbase	Plate load test	-	87 pci
			CBR	-	-
			In situ dry density	-	-
		Subgrade surface	Plate load test	-	60 pci
			CBR	4.2	3.4
			In situ dry density	89.4 pcf	86.8 pcf
		1 foot below subgrade Surface	CBR	9.4	8.2
			In situ dry density	91.8 pcf	93.5 pcf
MRG	MRG	Rubblized concrete	Plate load test	322 pci	457 pci
		Subgrade surface	Plate load test	106 pci	149 pci
			CBR	11	11.2
			In situ dry density	91.7 pcf	92.9 pcf
		1 foot below subgrade surface	CBR	8.8	8.2
			In situ dry density	92.0 pcf	91.5 pcf
MRS	MRS	Rubblized concrete	Plate load test	780 pci	579 pci
		P-306 econocrete subbase	Plate load test	409 pci	504 pci
		P-154 subbase	Plate load test	270 pci	202 pci
			CBR	-	-
			In situ dry density	-	-
		Subgrade surface	Plate load test	171 pci	101 pci
			CBR	6.9	6
			In situ dry density	91.3 pcf	90.7 pcf
		1 foot below subgrade surface	CBR	10.4	9.3
			In situ dry density	90.0 pcf	89.7 pcf

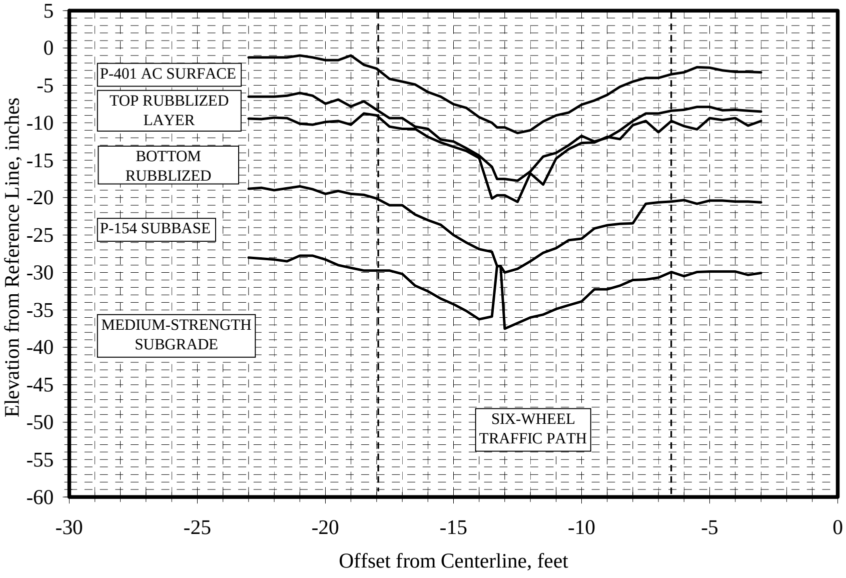
pcf = pounds per cubic foot
pci = pounds per cubic inch

LAYER PROFILES.

When the tests were completed, the trench walls were cleaned to clearly expose the layer interfaces. The pavement layer profile measurements can be used to quantify the contribution of each component layer to the total pavement rutting and upheaval. Measurements of the pavement layer interface profiles were made relative to a horizontal string line. Figure 19 shows the layer profiles in test item MRC.



MRC-W Trench



MRC-E Trench

Figure 19. Pavement Layer Profiles From Trenches in Test Item MRC

The figure shows that the HMA surface and the top rubblized layer (top 3 inches of finely rubblized material) contributed to rutting. Shear failure in the subgrade resulted in significant upheaval outside the traffic path. Subgrade penetration into the subbase was observed. Significant shoving in the HMA layer was also observed.

Figures 20 and 21 show the pavement layer profiles for test items MRG and MRS, respectively. The figures show that most of the rutting was contributed by the top 3 inches of the thin rubblized layer and the HMA overlay. The top 3 inches of rubblized layer are mainly composed of loose dust and stones with a top size of 1 inch. The bottom 9 inches of rubblized layer were 4 to 15 inches of tightly locked concrete pieces. A significant amount of shoving in the HMA layer was observed that resulted in significant upheaval just outside the traffic path. The subgrade in test item MRG (figure 20) showed indications of shear failure, as evidenced by the subgrade upheaval outside the traffic path.

Another observation made during the trenching study was that the rubblization process did not induce any cracks in the underlying P-306 econocrete subbase layer in test item MRS (figure 22).

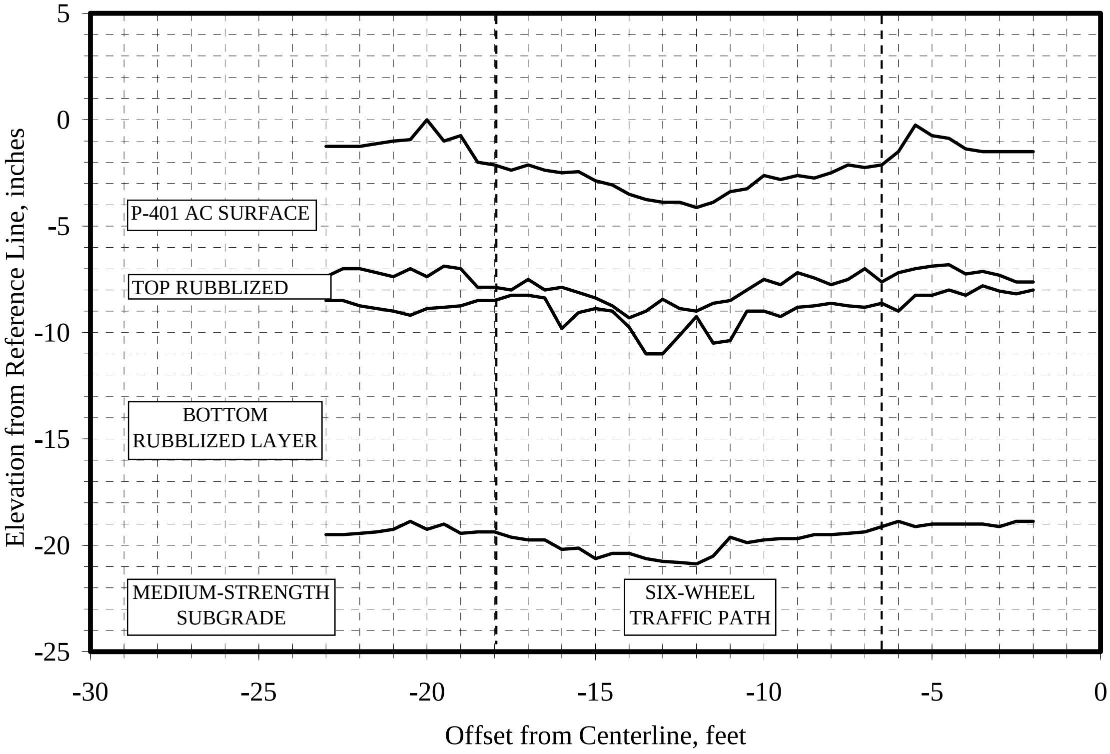


Figure 20. Pavement Layer Profiles From Trench in Test Item MRG

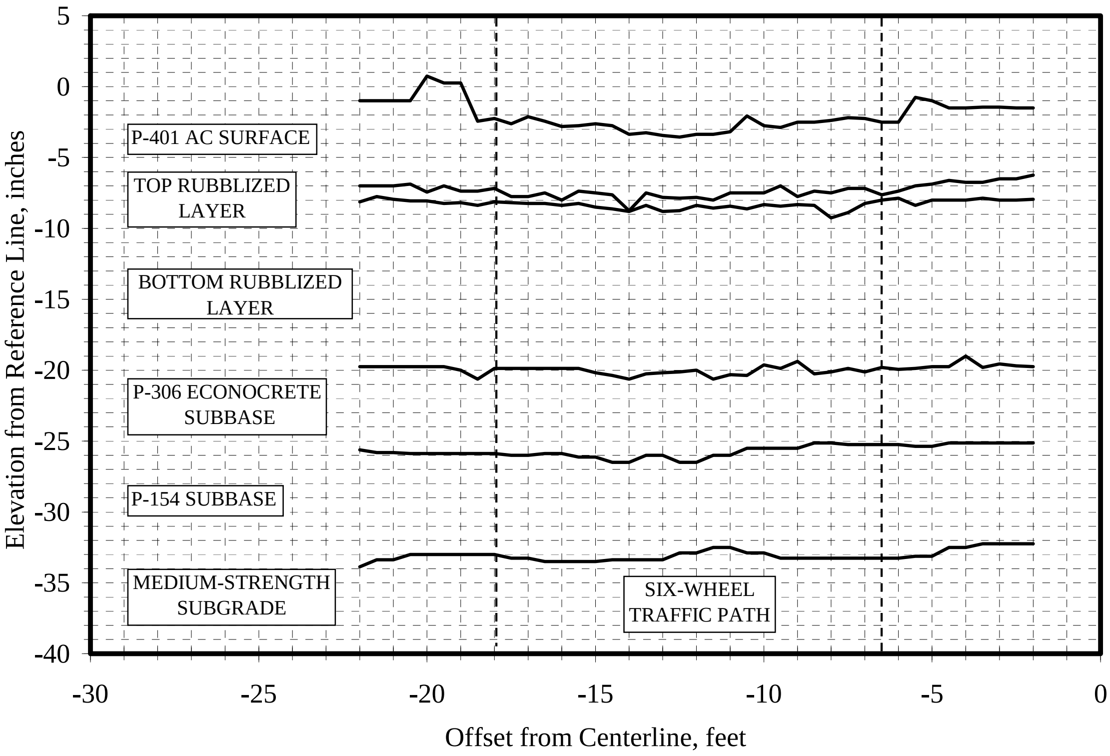
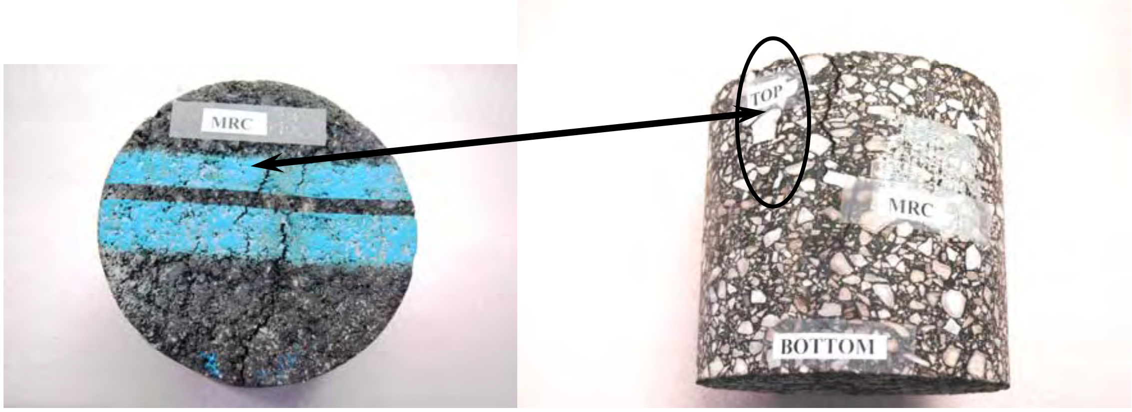


Figure 21. Pavement Layer Profiles From Trench in Test Item MRS

Figure 22. P-306 Econocrete Subbase in Test Item MRS

Cores were also extracted from the asphalt layer to determine the asphalt thickness and crack propagation mode (top-down or bottom-up). Both the trench studies and extracted cores confirmed that the fatigue cracking was top-down with no evidence of bottom-up cracking (figure 23). The fatigue crack was longitudinal and in the direction of the loading. Table 4 shows the number of passes when fatigue cracks were first observed in the three test sections.



(a) (b)

Figure 23. Cores Taken Through Fatigue Cracks—(a) Top View and (b) Side View

Table 4. Fatigue Cracks Observed in Rubblized Concrete Test Items

Test Item	Passes to First Fatigue Crack Observed		Total Passes	Crack Nature
	Passes at 55,000 Wheel Load	Passes at 65,000 Wheel Load		
MRC	5082	594	5,676	Top Down
MRG	5082	9570	14,652	Top Down
MRS	No cracks observed at 26,000 passes			

FAILURE MECHANISM IN RUBBLIZED CONCRETE PAVEMENTS

The NE end of MRC was the first area of the rubblized pavements to show signs of failure (figure 18). This failure was not representative of the test item’s structural performance as a whole because one of the preoverlay test pits (for subgrade evaluation) was located where the pavement failed. A weakened support system resulted because the replaced subbase aggregate material could not be compacted to the same density as in the original construction. A depression in the pavement surface was observed at this location after about 400 load repetitions. The depression migrated longitudinally toward the east until it was about 15 feet long, but the structure continued to support the full traffic load until it appeared to be in danger of suffering complete structural collapse at 11,814 passes. The weakened area did not migrate back into the west half of the test item, and the declared structural life of MRC-NW of 14,256 passes is believed to be a true representation of the structural performance of the test item. Also, MRC-NW did not appear to be in danger of complete structural collapse as had MRC-NE. Trafficking in MRG and MRS was terminated after 25,608 passes. From visual inspection at the end of trafficking, MRG-N appeared to be suffering from structural upheaval outside the wheel track, but MRS-N did not.

Figures 24 through 27 show the trench faces in test item MRC and close-ups of the failure zones. Figures 28 and 29 show the MRG and MRS trenches, respectively.

Figure 24. The MRC-W Trench

Figure 25. Close-Up of Failure Zone (MRC-W)

Figure 26. The MRC-E Trench

Figure 27. Subgrade Intrusion Into Subbase (MRC-E)

Figure 28. The MRG Trench

Figure 29. The MRS Trench

Excluding the top 3 inches of finely rubblized material, the rubblized concrete layer behaved as a tightly interlocked, high-density, unbound base. The strength of the rubblized concrete layer is derived from the tight interlock between the rubblized concrete pieces and the confinement provided by the HMA overlay and the support system underneath (subbase and subgrade, etc.).

This interlock deteriorates under repeated wheel loads. The rate of deterioration is controlled by various factors. Some of the important factors are:

- Magnitude and wander of wheel loads
- Loss of confinement due to fatigue cracks in the HMA overlay layer
- Loss of confinement due to a weak support system (underneath the rubblized concrete layer), allowing high vertical deflections in the pavement structure

In test item MRC, the top 3 to 4 inches of subgrade had reduced strength (CBR 3 to 4) because of moisture migration from the P-154 subbase into the subgrade. This weak layer of subgrade allowed higher vertical deflection in the pavement structure, which resulted in a faster rate of interlock deterioration between the rubblized concrete pieces and ultimate failure of the pavement structure. As shown in figures 20 and 28 (for MRG) and figures 21 and 29 (for MRS), the rubblized layer did not experience severe deterioration since the support system and the HMA overlay provided sufficient confinement and allowed limited vertical movement. This resulted in longer pavement structural life. Also, the rubblization process did not induce cracking in the underlying econocrete layer (as observed during the posttraffic test).

RUBBLIZED CONCRETE MODULUS.

BACKCALCULATION OF RUBBLIZED CONCRETE MODULUS. HWD tests were routinely performed using the FAA KUAB HWD equipment at three different load levels: 12,000, 24,000, and 36,000 lb. A PSPA was used in conjunction with the HWD to estimate the asphalt concrete modulus. Figure 30 shows the PSPA equipment.

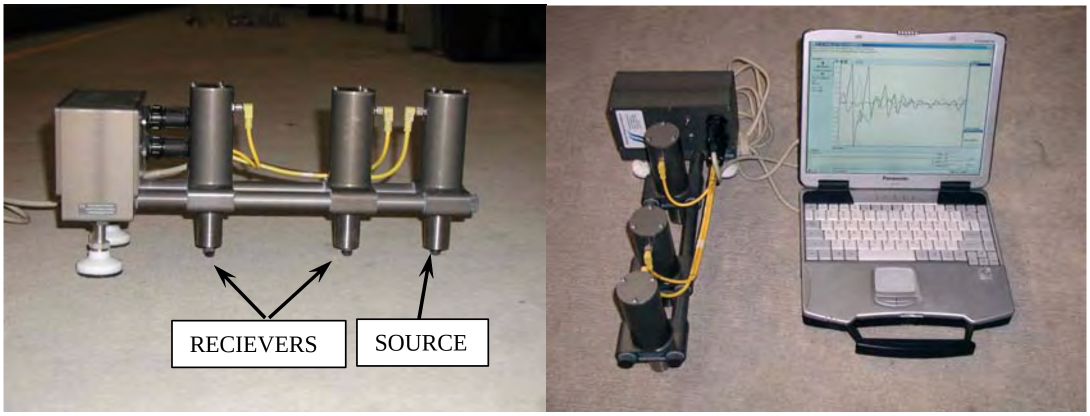


Figure 30. Portable Seismic Pavement Analyzer

PSPA is a portable device and consists of two transducers (receivers) and a source. The device operates from a computer. The operating principle of PSPA is based on generating and detecting stress waves in a layered medium. The data collected by PSPA is processed by spectral analysis to determine the modulus of the layer. A more detailed explanation on theory and equipment can be found in references 4 and 5. Figure 31 shows the P-401 asphalt concrete surface modulus obtained from PSPA tests.

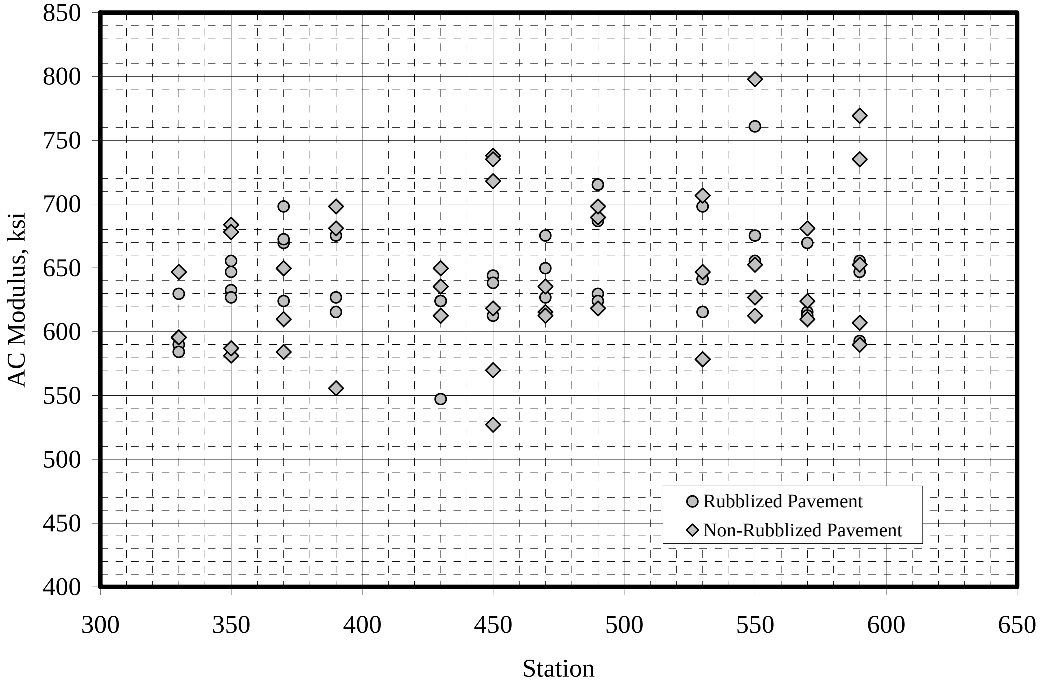


Figure 31. P-401 Asphalt Concrete Surface Modulus From PSPA Tests

Pretraffic HWD tests were performed on a 10-foot grid to study the uniformity of the pavement structures. During the traffic tests, HWD tests were performed at 15-foot (inside trafficked area) and 5-foot (outside trafficked area) offsets north of centerline. Moduli for the rubblized concrete layer were backcalculated using FAA BAKFAA software. The CBR test results from posttraffic tests (trenches) on subgrade (table 3) and the P-401 asphalt concrete layer modulus obtained from PSPA were used as the input properties for subgrade layers in the backcalculation procedure. The elastic modulus was backcalculated only for the rubblized concrete layer. In the backcalculations, a stiff layer (hard bottom) was placed at a 10-foot depth below the pavement surface (this is the depth for which the native subgrade had been replaced with the medium strength subgrade over which the test items were constructed). The native soil was stiff, sandy soil. In addition to the backcalculation of rubblized concrete modulus, the deflection data were used to compute the impulse stiffness modulus (ISM), defined as the force amplitude divided by D0. The deflection basin shape parameter AREA was also computed. More details about backcalculation, ISM, and AREA can be found in Advisory Circular (AC) 150/5370-11A [6]. Figures 32 through 34 show the variation in the backcalculated modulus of the rubblized concrete layer, the ISM, and the deflection basin shape factor AREA inside the trafficked area as the traffic tests progressed.

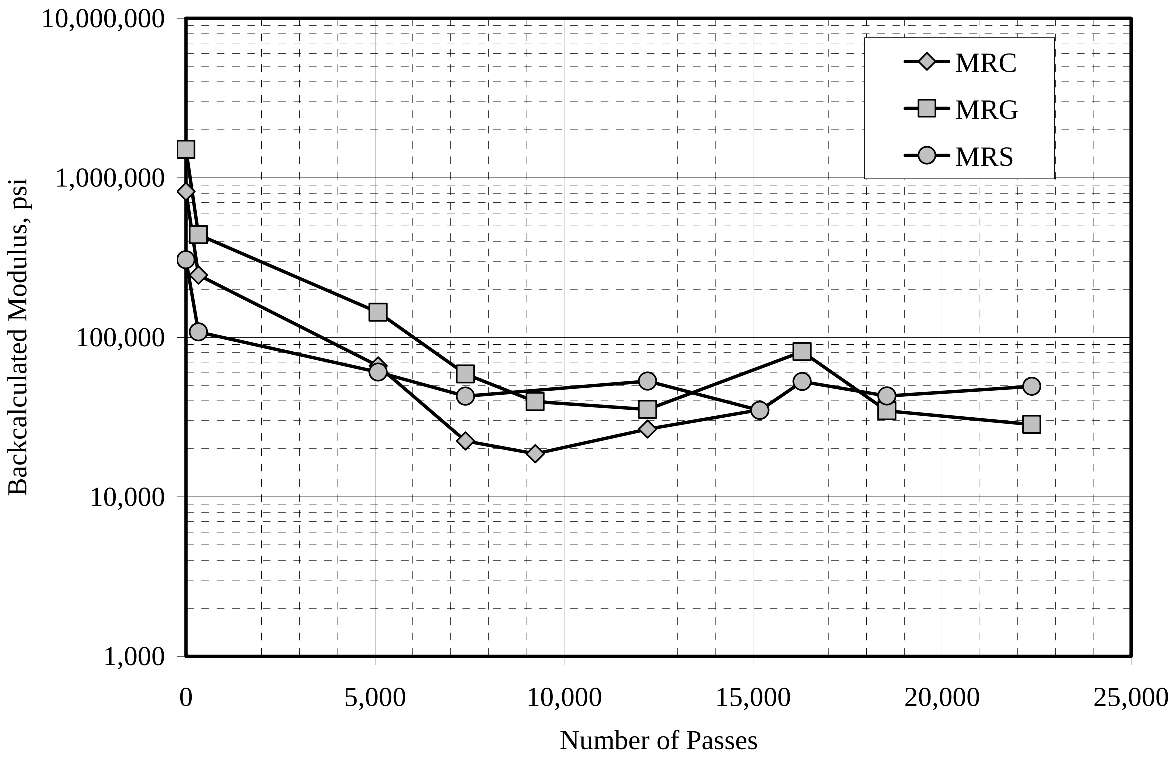


Figure 32. Changes in the Modulus of Rubblized Concrete During the Traffic Tests

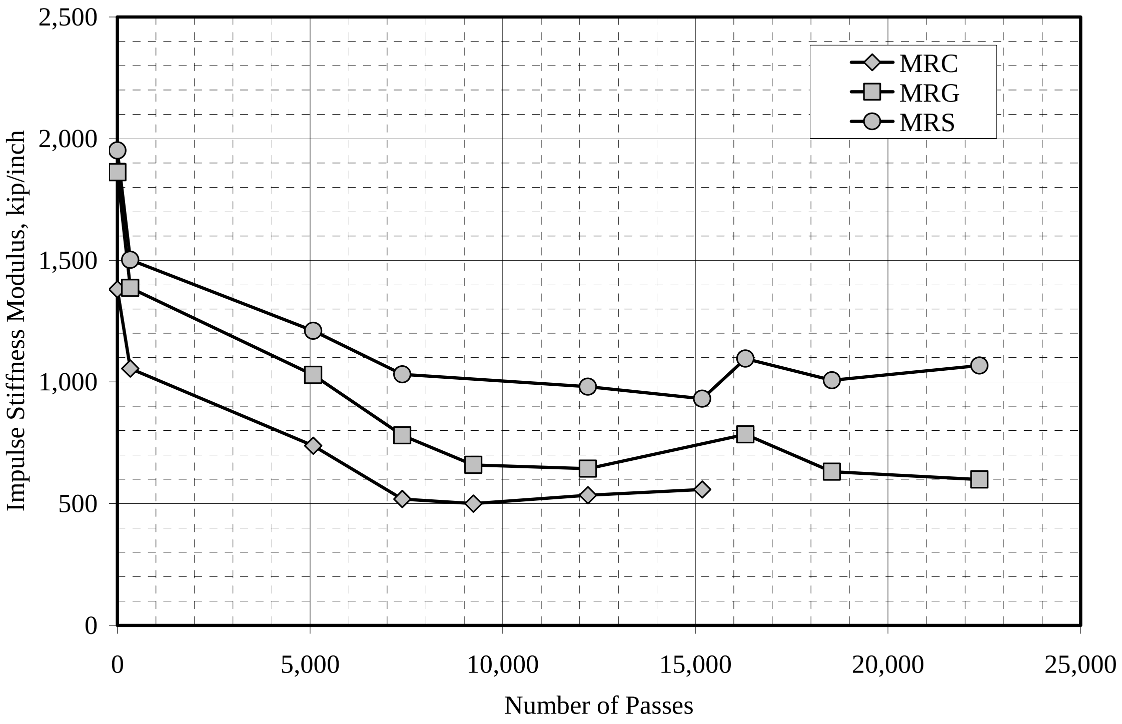


Figure 33. Changes in the Impulse Stiffness Modulus During the Traffic Tests

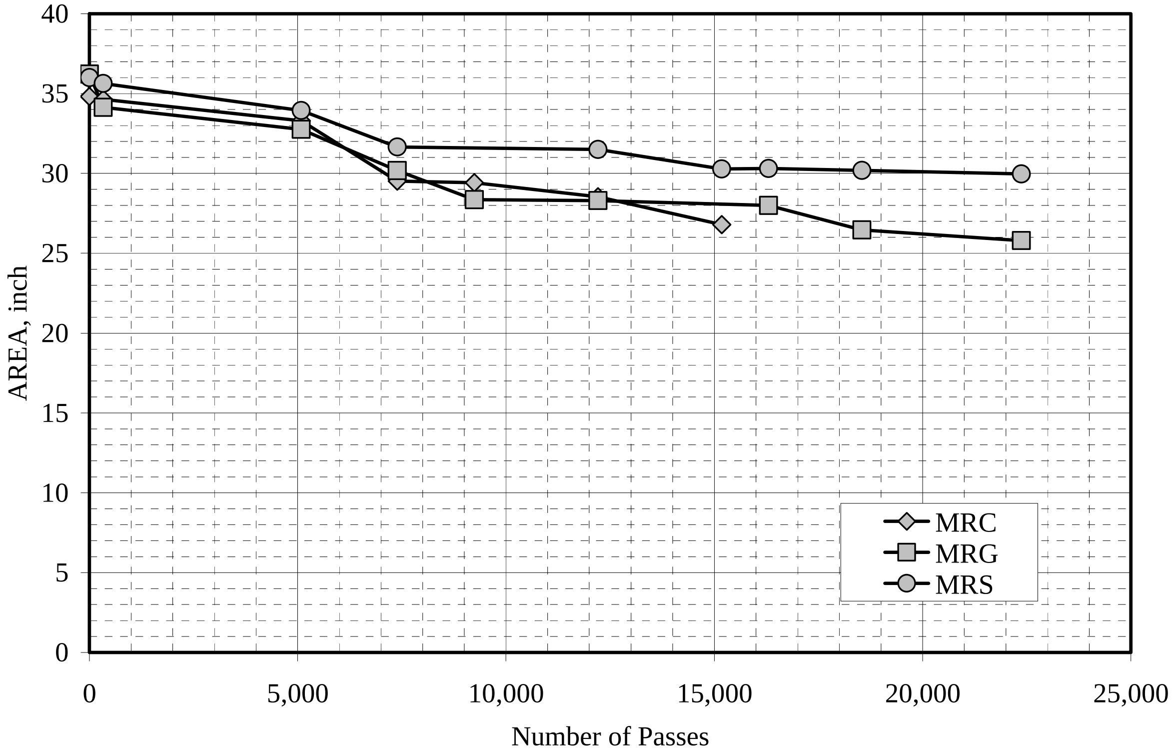


Figure 34. Changes in the Deflection Basin Shape Factor AREA During the Traffic Tests

Figure 32 shows that the rubblized concrete modulus reduces to approximately 30 percent of the initial modulus value after 330 passes for all three test items. The pavement performance, as indicated by rut depth, does not show a decline of this magnitude. In fact, for the first 5,083 passes, the maximum rut depth in all three test items is about 0.25 inch. After 5,083 passes, the modulus drops to approximately 20,000 psi for MRC, 40,000 psi for MRG, and 50,000 psi for MRS. These results indicate that backcalculated modulus may not be a good predictor of pavement performance when applied to a flexible pavement design procedure.

The increased crack width between the rubblized concrete pieces would also contribute to the increased peak surface deflections. Another factor contributing to the vertical deformation is the top 2 to 3 inches of fine material under the HWD load, where increased peak center deflections could result in lower than expected backcalculated modulus values. The HWD tests inside the traffic path were centered over an underlying dowelled longitudinal joint. During the posttraffic trench study, it was observed that the rubblization process did not debond the dowels (figure 35) from the two adjacent slabs, and the size of concrete pieces ranged from 3 to 4 feet in length and width. It is possible that this may have contributed to the higher D0s rather than the deterioration of the rubblized concrete layer. However, none of these factors significantly affected the performance of the rubblized layer, as observed from the rut depth measurements.

Figure 35. Rubblized Concrete Pieces Still Connected With Dowels

Figure 34 shows the changes in the deflection basin shape factor AREA during the traffic tests. To illustrate the procedure used to calculate the AREA shape factor, figure 36 shows a hypothetical deflection basin measured during an HWD test. D0, D2, D3, D4, D5, D6, and D7 are deflections measured at 0-, 12-, 24-, 36-, 48-, 60-, and 72-inch offsets from the center of the load plate.

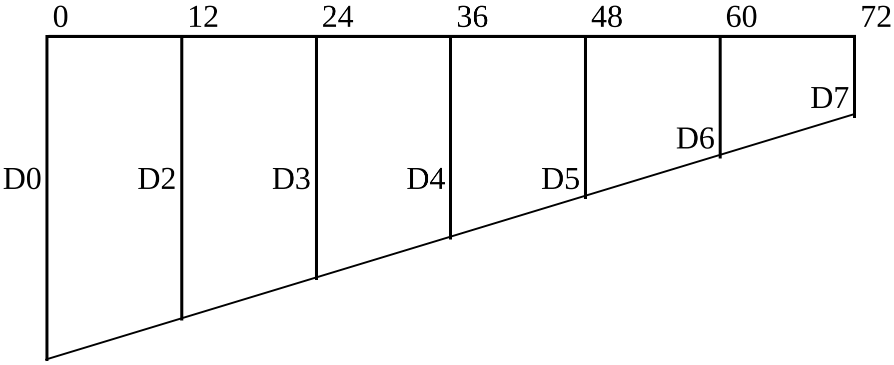


Figure 36. Typical Deflection Basin Under HWD Load

AREA is the area of the deflection basin after all the deflections have been normalized using peak center deflection, D0, and is computed as follows:

$$6*[2*(D2+D3+D4+D5+D6) + (D0+D7)] / D0$$

For the first 5068 passes (at a 55,000-lb wheel load), all three test items showed similar rut depths. The AREA values (figure 34) also suggest similar behavior. The AREA values for test item MRC are somewhat overstated after about 7000 passes because the HWD tests could not be performed in the NE area of the test item due to large rut depths, as that part of the test item deteriorated toward failure.

At the end of trafficking, the AREA values were reduced by approximately 29 percent of the initial value for MRG and reduced by 17 percent for MRS. The rubblized concrete in the MRG and MRS trenches (figures 28 and 29) showed no significant signs of deterioration at the end of trafficking.

<u>PAVEMENT LIFE COMPUTATIONS</u>. According to EB-66,

> “Rubblized pavements modulus have been found to vary from a low of 30 ksi to over 300 ksi depending on the original pavement thickness, base type and condition of base layers. When strength parameters are unknown, it is a fair assumption that most rubblized material will perform equal to or better than FAA standard Item P-209. Unless additional project specific information is available, a one-to-one substitution should be used in the design procedures provided that sufficient subgrade conditions exist to allow proper rubblization.”

Approximately the same range of backcalculated modulus values were measured during trafficking of the three test items, although significantly higher values were measured in MRC and MRG before trafficking. The value of approximately 300,000 psi, measured in MRS before trafficking, was at the top end of the EB-66 range.

Using the EB-66 assumption stated above (of treating the rubblized concrete layer as a P-209 crushed stone base), pavement life was computed using LEDFAA-1.3. The subgrade CBR is the average of CBR values (table 3) at the top of the subgrade and a depth 1 foot below the subgrade surface. The design CBR values were computed in a similar way as the new alpha factor report [7]. The results are summarized in table 5.

Table 5. Predicted Life Computations Using LEDFAA-1.3

Test Item	Pavement Life, Passes Four-Wheel 55,000-lb Wheel Load	Pavement Life, Passes Six-Wheel 65,000-lb Wheel Load	Observed Pavement Life, Passes
MRC	236	29	14,652
MRG	42	10	25,608*
MRS	385,418	6343	25,608**

* Appeared to suffer from structural upheaval; trafficking terminated.
** No signs of failure; trafficking terminated.

The traffic tests for the first 5068 passes were performed at a 55,000-lb wheel load and four-wheel gear. After that, the traffic tests were performed at a 65,000-lb load and six-wheel gear (table 1). Comparing the observed pavement life (figure 13) and predicted pavement life (table 5), the results show that using EB-66 assumptions are very conservative and LEDFAA 1.3 grossly underpredicts pavement life as measured in the full-scale tests reported here.

SUMMARY

Full-scale traffic tests were completed on three rubblized and three nonrubblized rigid airport pavements that were overlaid with 5 inches of HMA. This report describes the performance of rubblized concrete pavements with HMA overlay from full-scale traffic tests under heavy aircraft gear loads. All the rubblized pavements performed equally well at the original pavement test section loads. The load was increased significantly to artificially induce failure by overloading. Of the three rubblized test items, MRC suffered severe structural distress. Test item MRG probably suffered structural deterioration at the end of trafficking but retained sufficient structural capacity to support the applied load. Test item MRS did not suffer severe structural deterioration at the end of trafficking despite having accumulated significant levels of rutting and shear flow in the asphalt layer. The moisture condition that led to the poor performance of MRC was poor drainage. The results from posttraffic tests were useful in providing some insight into the failure mechanism of rubblized concrete pavements.

None of the nonrubblized pavements suffered significant structural deterioration or significant levels of rutting. Nor was any reflective cracking evident at the surface of the nonrubblized pavements, but this was expected because the tests were performed indoors during warm weather. The test results were useful in determining the structural characteristics of rubblized pavements for use in thickness design procedures.

The complex nature of this material makes strength testing on the rubblized material very difficult, if not impossible. HWD test data were used to compute an (effective) modulus of the rubblized concrete layer by backcalculation. The backcalculation of the rubblized concrete modulus yielded values that did not reasonably predict observed life in the traffic tests when substituted into a representative flexible pavement design procedure (LEDFAA 1.3). It was observed that the HWD deflection basins exhibited what appeared to be a typically high D0 relative to the D2 through D7 deflections. Changes in AREA with traffic, computed from the HWD tests and in which D0 is used as a normalizing factor, were more consistent with the observed performance than the backcalculated modulus values.

Also, the results indicate that, for the conditions existing in the test pavements, the assumptions for design in EB-66 are overly conservative.

The pavement design procedure for asphalt concrete overlays over rubblized concrete base should consider limiting vertical deformation/strain in the underlying layers. More full-scale test data are needed to determine the magnitude of limiting deformation/strain to develop a failure criterion.

For commercial airports serving wide-body aircraft (gross weights >100,000 lb), per AC 150/5320-6D [8], rigid pavements are required to have a stabilized base. MRS is the most representative of pavement structures that are encountered on a commercial airport in the U.S. The performance of MRS under a 65,000-lb wheel load suggests that rubblized concrete pavements with HMA overlay are a viable option on commercial airports. The presence of a stabilized base underneath the rubblized concrete layer limits the vertical deflection in the layer below the rubblized concrete layer and helps to keep the rubblized pieces tightly interlocked.

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