Posttraffic Testing Results on Rubblized Concrete Pavements With HMA Overlays at the FAA's National Airport Pavement Test Facility

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Abstract

Full-scale traffic tests were completed on three rubblized rigid airport pavements which had been overlaid with five inches (12.7 cm) of hot mix asphalt at the FAA's National Airport Pavement Test Facility. Initially, the overlaid pavements were trafficked with a 4-wheel landing gear configuration (with wander) and 55,000-lbs (25-tonnes) 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, after which the wheel load was increased to 65,000-lbs (29.5-tonnes) and a 6-wheel landing gear was used for testing. One of the rubblized pavements (MRC – rubblized concrete on conventional base) exhibited complete structural failure. MRG (rubblized concrete on grade) was considered to have suffered severe structural deterioration at the end of trafficking but retained sufficient structural capacity to support the applied load. MRS (rubblized concrete over econocrete base) 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 posttraffic investigation into the failure mechanism of the pavement structure. The trenching included testing for layer characterization and removal of each of the pavement layers 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 paper summarizes the results from posttraffic tests. The performance of MRS under a 65,000-lb (29.5-tonnes) wheel load suggests that rubblized concrete pavements with HMA overlay are a viable option on commercial airports because a stabilized base underneath the rubblized concrete layer limits the vertical deflection in the layer below the rubblized concrete layer and helps in keeping the rubblized pieces tightly interlocked. The results from posttraffic tests were useful in providing some insight into the failure mechanism of rubblized concrete pavements.

INTRODUCTION

Rubblization of deteriorated concrete pavements is fast becoming a popular method of pavement rehabilitation because of its ability to prevent reflective cracking. The rubblized concrete layer behaves as a tightly keyed, interlocked, high-density unbound base. There are a number of airfield projects that have used rubblization as a pavement rehabilitation technique (1). The projects range from heavy load military airfields to local general aviation (GA) 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 (2) summarizes the guidelines for rubblized Portland Cement Concrete base courses. These guidelines are based on industry experience. It provides interim guidance and full-scale testing is still needed to develop design standards for the use of 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 (MRC, MRG, and MRS) at the Federal Aviation Administration's (FAA) National Airport Pavement Test Facility (NAPTF) with 12-inch (305 mm) thick concrete slabs on different support systems (slab on crushed stone base, slab on grade, and slab on stabilized base) were rubblized with a resonant pavement breaker and overlaid with five inches (127 mm) of P-401 HMA. 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 lbs (25-tonnes). All three test items were constructed on medium strength (CBR≈7-8) clay subgrades. The overlaid pavements were subjected to full-scale accelerated traffic loading until complete structural failure was attained. This is the first study ever conducted on the full-scale accelerated pavement testing of rubblized concrete pavements with HMA overlay under heavy aircraft loading.

Four trenches were opened perpendicular to the centerline of the test items to conduct posttraffic investigation into the failure mechanism of the pavement structure. The trenching included testing for layer characterization and removal of each of the pavement layers 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 paper summarizes the results from the posttraffic tests and provides some insight into the failure mechanism of rubblized concrete airport pavements.

NATIONAL AIRPORT PAVEMENT TEST FACILITY (NAPTF)

The National Airport Pavement Test Facility (NAPTF) is an indoor test facility located at the FAA William J. Hughes Technical Center, Atlantic City International Airport, New Jersey. It is used 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 (274.3 m) long by 60 ft (18.3 m) wide test pavement area, embedded pavement instrumentation and a dynamic data acquisition system (20 samples per second), environmental instrumentation and a static data acquisition system (4 samples per hour), and a test vehicle for loading the test pavement with up to twelve aircraft tires at wheel loads of up to 75,000 lbs (34 tonnes). Additional information about the test facility (including layout, instrumentation plans, database etc.) is available elsewhere (http://www.airporttech.tc.faa.gov). A construction cycle at the NAPTF involves test pavement construction including instrumentation, traffic tests to failure, posttraffic testing (includes trenching activities and other tests), and pavement removal. A typical construction cycle (CC) at the NAPTF is shown in figure 1.

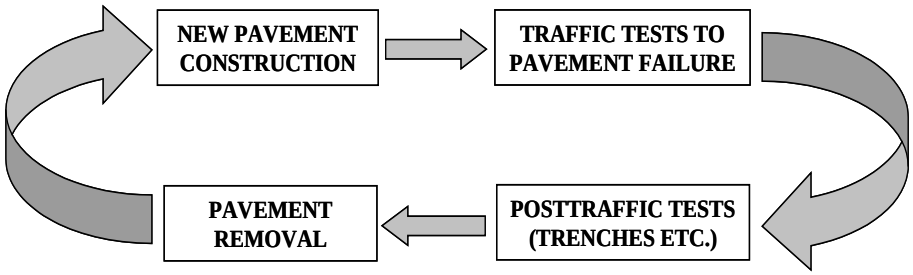


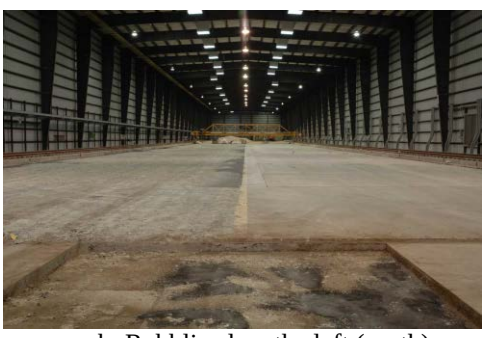
FIGURE 1 Construction cycle at the NAPTF.

PAVEMENT STRUCTURES

Three rigid pavement test items were constructed and tested during construction cycle two (CC2) at the NAPTF. Each test item was 75 feet long by 60 feet wide (22.9 m by 18.3 m) with twenty 15 by 15 foot by 12-inch thick concrete slabs (4.57 m by 4.57 m by 30.5 cm). One of the test items (MRG) was built directly on the subgrade, the second (MRC) was built on a crushed aggregate subbase on top of the subgrade, and the third (MRS) was built on an econocrete subbase (base course composed of aggregate and cement uniformly blended together and mixed with water) over a crushed aggregate lower subbase. Each test item was separated into two 30-foot wide traffic lanes, north and south. Construction was completed in April, 2004. Detailed information on the design and construction characteristics of the pavement structures can be found in (3). Traffic testing was completed in December, 2004. More details about traffic tests and posttraffic tests on CC-2 test items can be found elsewhere (4). The structural condition index (SCI) of all the rigid pavement test items, in both traffic lanes, was less than 20 (shattered slab condition) at the end of trafficking. However, most of the cracks were tight, with none rated worse than low severity. Detailed explanation on SCI computation and slab condition is given in (5).

In January, 2005, all of 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. Then, in June, 2005, the rubblized pavement was lightly wetted, rolled with a vibratory steel drum roller, and overlaid with five inches (305 mm) of P-401 hot mix asphalt. Figures 2a and 2b show, respectively, the vibrating foot of the resonant breaker, and the test pavement surface after rubblization. The pre-traffic test pits showed that the rubblization process induced cracks/fractures for the entire depth of the slabs and that the cracks were tightly held (4). Figure 3 shows the pavement cross sections after the placement of the HMA overlay.

a. Rubblizing the north traffic lane with the resonant breaker.



b. Rubblized on the left (north) as-trafficked on the right (south).

FIGURE 2 Rubblized Concrete Pavement Test Items at the NAPTF.

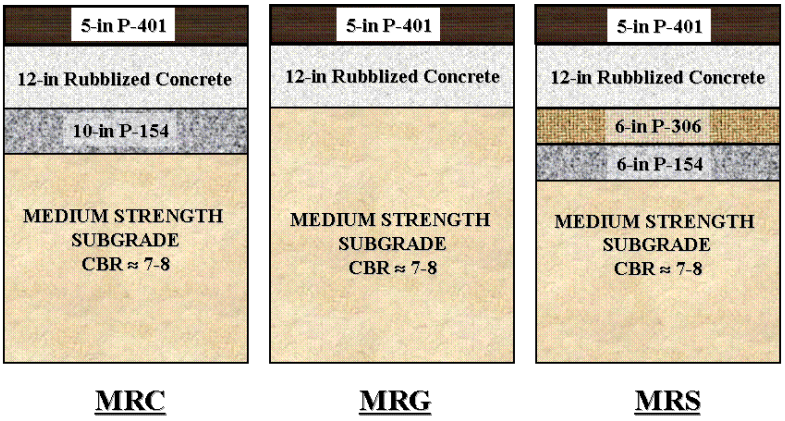


FIGURE 3 CC-2-Overlay Pavement Test Items. (1-inch = 25.4 mm)

P-401, P-306, and P-154 are FAA standard specifications (6) for HMA surface, econocrete subbase, and uncrushed aggregate subbase (crushed aggregate screenings were used at NAPTF) respectively.

TRAFFIC TESTS AND PAVEMENT PERFORMANCE

Since only four wheels were available for loading on the unrubblized traffic lane, the traffic tests were started with a four-wheel dual-tandem configuration on both traffic lanes. The geometry was the same on both traffic lanes, with dual spacing of 54 inches (137.2 cm) and tandem spacing of 57 inches (144.8 cm). Wheel load was set at 55,000 lbs (25 tonnes) because this was the load applied to the new construction CC2 test items and, although badly cracked at the end of trafficking, all of the test items were capable of structurally supporting the loads applied up to the end of trafficking. Adding five 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 (a few hundred or thousands of repetitions) but that the structure on stabilized base would probably last for many tens of thousands of repetitions.

Trafficking started on July 7, 2005, and continued until October 6, 2005, following the schedule in Table 1. (The wheel load was increased after 5,082 repetitions because none of the pavements showed any significant deterioration at that traffic level.) The standard NAPTF 66-repetitions per cycle wander pattern was used on both traffic lanes. The temperature of the asphalt varied between 66 and 85 ºF (19 and 29 ºC) during the period of testing. The average temperature of the asphalt was about 78 ºF (26 ºC).

TABLE 1 Trafficking schedule for CC2 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	4-wheel, 55,000 lbs	4-wheel, 55,000 lbs
07/26/05 08/12/05	5,083 11,814	MRG-N, MRC-N, MRS-N MRG-S, MRC-S, MRS-S	6-wheel, 65,000 lbs	4-wheel, 65,000 lbs
08/15/05 08/18/05	11,814 14,256	MRG-N, MRC-NW**, MRS-N MRG-S, MRC-S, MRS-S	6-wheel, 65,000 lbs	4-wheel, 65,000 lbs
08/19/05 08/24/05	14,257 16,302	MRG-N, MRS-N MRG-S, MRC-S, MRS-S	6-wheel, 65,000 lbs	4-wheel, 65,000 lbs
09/13/05 10/06/05	16,303 25,608	MRG-N, MRS-N MRG-S, MRS-S	6-wheel, 65,000 lbs	4-wheel, 65,000 lbs

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

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

Traffic testing was continued until either structural failure 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 test items were monitored through a combination of visual surveys and non-destructive testing, 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.

A 16-foot (4.88 m) 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 and NE for the rubblized test items (N stands for north side of the longitudinal centerline). Figure 4 shows the straightedge rut depth measurements for all the three rubblized test items (MRC, MRG, and MRS). All of the test items showed similar rut depths during the first 5082 passes (55,000-lbs wheel load, 4-wheel landing gear configuration). It was observed visually 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 that of the nonrubblized pavements. It was therefore 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 trafficking, six-wheel triple-dual-tandem loading at 65,000 lbs (29.5 tonnes) wheel load was applied to the rubblized pavement and four-wheel dual-tandem loading at 65,000 lbs (29.5 tonnes) was applied to the non-rubblized pavements. The six- and four-wheel configurations at increased loading both had the same dual and tandem spacings of 54 and 57 inches (137.2 and 144.8 mm) respectively.

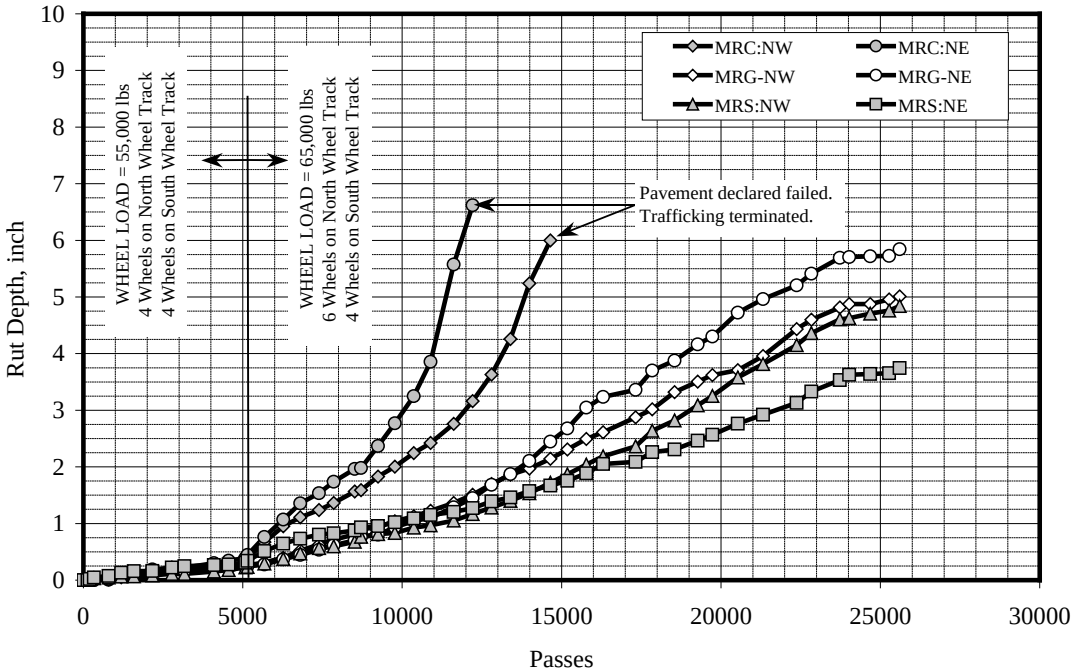


FIGURE 4 Straightedge Rut Depth Measurements in the Rubblized Concrete Test Items. (1-inch = 25.4 mm)

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 these numbers of passes, the rut depth measurements (as shown in Figure 4) 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. Maximum rut depths from the transverse profiles at the end of trafficking were 4 inches (100 mm) on MRC-N, 2.5 inches (64 mm) on MRG-N, and 2 inches (50 mm) on MRS-N. Significant structural upheaval was also observed outside the wheel track in MRC-N, but neither the straightedge measurements nor the transverse profile measurements can separate the contributions of the underlying structural layers and the asphalt upheaval movement. Transverse trenches were therefore opened in the test items so that transverse profiles of the structural layer interfaces could be measured. The NE end of MRC was the first area of the rubblized pavements to show signs of failure (Figure 5).

FIGURE 5 Pavement Failure in the East End of Rubblized Test Item MRC.

MRC-NW did not exhibit 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.

POSTTRAFFIC TESTS

Four trenches were dug in the rubblized test items perpendicular to the centerline of test items MRC (2 trenches), MRG (1 trench), and MRS (1 trench) at the locations of rut depth measurements on the test pavements. The locations of the test items and the trenches were as follows:

TEST ITEM	TRENCH	START STATION (feet)	END STATION (feet)
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 posttraffic investigation into the failure mechanism of the pavement structure. The trenching involved removal of the P-401 HMA layer, the rubblized concrete layer, the P-209 crushed stone base 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 the removal of the P-401 HMA surface, the rubblized concrete layer was exposed in all the four trenches. Plate load tests (AASHTO 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 removal of the P-154 subbase in MRC, CBRs, plate load tests and density measurements were made 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 made on the subgrade surface after removal of the P-154 subbase. In all the trenches, CBRs and density measurements were also made at a depth of 1-foot (305 mm) below the subgrade surface. After the completion of testing, the trench walls were cleaned to clearly expose the layer interfaces. Measurements of the pavement layer interface profiles were made 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 4 trenches are summarized in Table 2. One of the significant observations relative to Table 2 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 three or so inches of the subgrade. The surface of the subgrade in the MRC test pits had a strength of approximately 4 CBR whereas the strength one foot (30 cm) 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 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 from the plate load test inside the traffic path was lower (k = 322-pci) than the ‘k’ value

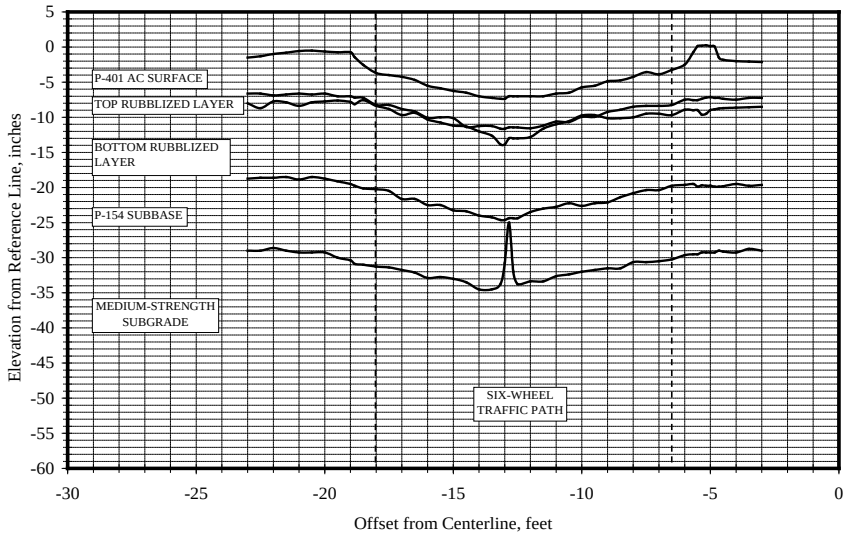
from outside the traffic path test (k = 457-pci). The lower ‘k’ value inside the traffic path could be the result of incipient failure in MRG. In test item MRS, the ‘k’ value from the plate load test inside the traffic path was higher (k = 780-pci) than the ‘k’ value from outside the traffic path test (k = 579-pci).

TABLE 2 Summary of Test Results from Trenching Study

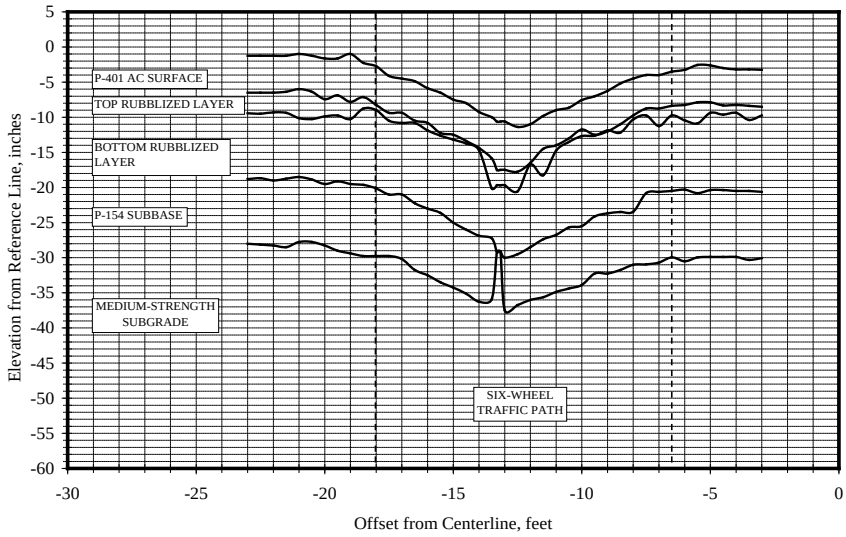
Test Item	Trench ID	Layer Type	Test Type	Test Results	
				Inside Traffic Path	Outside Traffic Path
MRC	MRC-W	Rubblized Concrete	Plate Load Test	-	-
		P-154 Subbase	Plate Load Test	144 pci	92 pci
			CBR	35.9	33.7
			In-Situ Dry Density	122.4 pcf	122.1 pcf
		Subgrade Surface	Plate Load Test	-	70 pci
			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

LAYER PROFILES

After the completion of testing, 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 6 shows the layer profiles in test item MRC.



a. MRC-W Trench



b. MRC-E Trench

FIGURE 6 Pavement Layer Profiles from Trenches in Test Item MRC. (1-inch = 25.4 mm)

The figure shows that the HMA surface and the top rubblized layer (top 3-inches (76 mm) 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 7 and 8 show the pavement layer profiles for test items MRG and MRS respectively. Figures 7 and 8 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 (76 mm) of rubblized layer are mainly composed of loose dust and stones with a top size of 1 inch (25 mm). The bottom 9-inches (229 mm) of rubblized layer were 4 inches (10.16 cm) to 15 inches (38.1 cm) concrete pieces tightly locked.

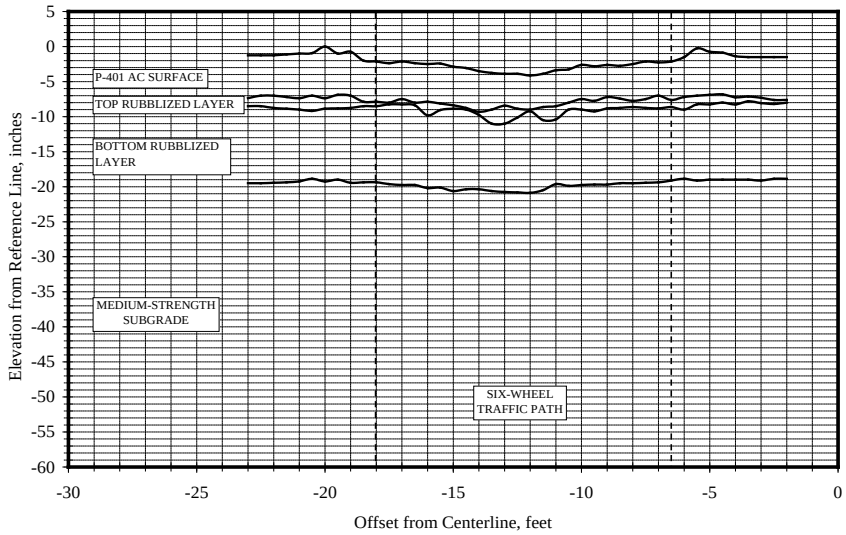


FIGURE 7 Pavement Layer Profiles from Trench in Test Item MRG. (1-inch = 25.4 mm)

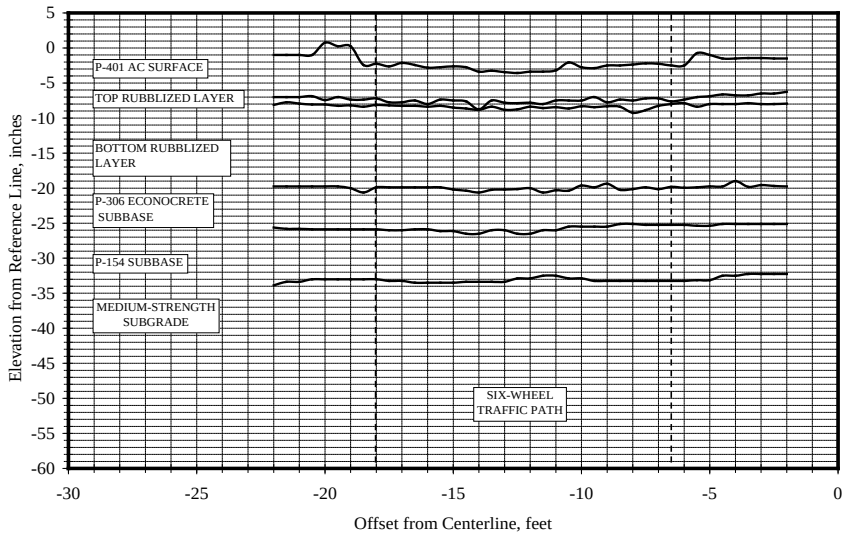


FIGURE 8 Pavement Layer Profiles from Trench in Test Item MRS. (1-inch = 25.4 mm)

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 7) showed indications of shear failure as evidenced by the subgrade upheaval outside the traffic path.

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 5). This failure was not representative of the structural performance of the test item as a whole because one of the pre-overlay 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 towards the east until it was about 15 feet (4.6 m) 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 9 shows the photographs of trench faces in test item MRC and close ups of the failure zones. Figures 10 and 11 show photographs of MRG and MRS trenches respectively.

MRC-W Trench

MRC-E Trench

Close-up of Failure Zone (MRC-W)

Subgrade Intrusion into Subbase (MRC-E)

FIGURE 9 Photographs from MRC Trenches.

FIGURE 10 Photograph of MRG Trench.

FIGURE 11 Photograph of MRS Trench.

Excluding the top 3-inches (76 mm) of finely rubblized material, the rubblized concrete layer behaves 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 will deteriorate under repeated wheel loads. The rate of deterioration is controlled by various factors. Some of the important ones are as follows:

- magnitude and wander of wheel loads;
- loss of confinement due to fatigue cracks in the HMA overlay layer;
- loss of confinement due to 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 (76 to 102 mm) 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 for higher vertical deflection in the pavement structure which resulted in a faster rate of deterioration of interlock between the rubblized concrete pieces and ultimate failure of the pavement structure. As can be observed from Figures 7 and 10 (for MRG) and Figures 8 and 11 (for MRS), the rubblized layer did not experience severe deterioration since the support system and the HMA overlay provided sufficient confinement and allowed for 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 trenching study).

SUMMARY / CONCLUSIONS

Full-scale traffic tests were completed on three rubblized and three nonrubblized rigid airport pavements which had been overlaid with five inches (12.7 cm) of hot mix asphalt. This paper presented the performance of rubblized concrete pavements with HMA overlay from full-scale traffic tests under heavy aircraft gear loads. Of the three rubblized test items, MRC was observed to suffer severe structural distress. Test item MRG was probably suffering severe structural deterioration at the end of trafficking but retained sufficient structural capacity to support the applied load. The third rubblized pavement (MRS) did not appear to be suffering severe structural deterioration at the end of trafficking despite having accumulated significant levels of rutting and shear flow in the asphalt layer.

The results from posttraffic tests were useful in providing some insight into the failure mechanism of rubblized concrete pavements. Also, the results indicate that, for the conditions existing in the test pavements, the assumptions for design in EB66 are overly conservative.

None of the non-rubblized pavements suffered significant structural deterioration or significant levels of rutting. There was no evidence of reflection cracking at the surface of the non-rubblized pavements, but this is to be expected because the tests were performed indoors during warm weather.

For commercial airports serving wide body aircraft (gross weights > 100,000 lbs), as per the FAA AC 150/5320-6D, 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 in keeping the rubblized pieces tightly interlocked.

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