STUDY OF FAILURE MECHANISMS IN RUBBLIZED CONCRETE PAVEMENTS WITH HOT MIX ASPHALT (HMA) OVERLAYS

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ABSTRACT: Full-scale traffic tests were completed on three rubblized rigid airport pavements 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 (with wander) and 55,000-lbs (25-tonnes) wheel load. No significant distresses were observed during the 5,000 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. Test item MRC (rubblized concrete pavement on conventional base) exhibited complete structural failure. 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. MRS (rubblized concrete over econocrete base) did not exhibit severe structural deterioration at the end of trafficking. Four trenches were opened perpendicular to the centerline of the test items to conduct post-traffic investigation into the failure mechanism of the pavement structure. The trenching included testing for layer characterization (plate load tests, CBR tests, in situ densities, moisture contents, layer profile measurements, and visual evaluations) and removal of each pavement layer to reveal the subgrade interface and subsequent subgrade layers below. This paper summarizes the results from the post-traffic tests. The performance of test item MRS suggests that rubblized concrete pavements with HMA overlay are a viable option on commercial airports. The results from post-traffic tests were useful in providing insight into the failure mechanism of rubblized concrete pavements.

1 INRODUCTION

Rubblization of deteriorated concrete pavements is becoming a popular method of pavement rehabilitation because of its ability to prevent reflective cracking, and it could be a cost-effective means of converting an existing failed or failing 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 (1). The projects range from heavy load military airfields to local general aviation (GA) airfields. Engineering Brief (EB) 66 (2) summarizes the guidelines for rubblized Portland Cement Concrete (PCC) base courses. These guidelines are based on industry experience. It provides interim guidance and full-scale testing is still needed to develop design standards. 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 FAAs 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 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 post-traffic investigation into the failure mechanism of the pavement structure. The trenching included testing for layer characterization (plate load tests, CBR tests, in situ densities, moisture contents, layer profile measurements, and visual evaluations) and removal of each pavement layer to reveal the subgrade interface and subsequent subgrade layers below.

2 OBJECTIVES

The objectives of this research were to:

- Study the performance of rubblized concrete pavements with HMA overlay under heavy aircraft loading.
- Study the post-traffic trench results and failure mechanism of rubblized concrete airport pavements.
- Provide guidance for the characterization of rubblized concrete layer in the pavement thickness design procedure.

This paper summarizes the results from the post-traffic tests and provides some insight into the failure mechanism of rubblized concrete airport pavements.

3 NATIONAL AIRPORT PAVEMENT TEST FACILITY

The 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, 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 lbs (34 tonnes). Additional information about the test facility is available elsewhere (http://www.airporttech.tc.faa.gov). A construction cycle (CC) at the NAPTF involves test pavement construction, including instrumentation, traffic tests to failure, post-traffic testing (includes trenching activities and other tests), and pavement removal.

4 PAVEMENT STRUCTURES

Three rigid pavement test items were constructed and tested during 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). The slabs were doweled in both the transverse and longitudinal directions. Test item MRG was built directly on the subgrade, MRC was built on a crushed aggregate subbase on top of the subgrade, and MRS was built on an econocrete subbase 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 post-traffic 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. A detailed explanation on SCI computation and slab condition is given in (5).

In January 2005, all the concrete slabs in the north traffic lane 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 five inches (127 mm) of P-401 HMA. Figures 1a and 1b show the vibrating foot of the resonant breaker and the test pavement surface after rubblization.

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

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

Figure 1. Rubblized concrete pavement test items at NAPTF

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 2 shows the pavement cross sections after the placement of the HMA overlay.

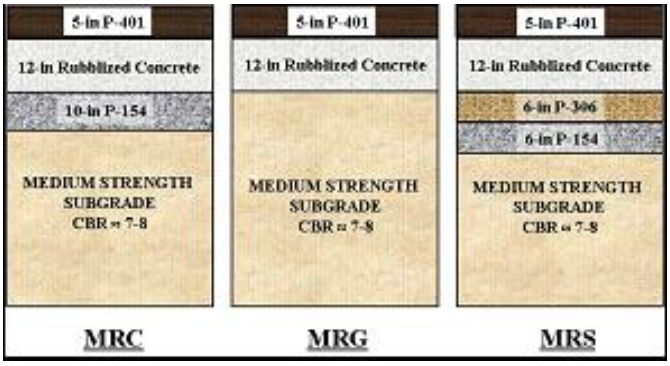


Figure 2. CC2 overlay pavement test items (1 inch = 25.4 mm).

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.

5 TRAFFIC TESTS AND PAVEMENT PERFORMANCE

The traffic tests were started with a four-wheel, dual-tandem configuration (dual spacing of 54 inches (137.2 cm) and tandem spacing of 57 inches (144.8 cm)) on both traffic lanes. 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 the test items were capable of structurally supporting the loads applied. Heavy-weight deflectometer (HWD) tests were performed using the FAAs HWD equipment on a 10-foot grid to study the uniformity of the pavement structures. The results showed that the pavement structure within each test item was fairly uniform (coefficient of variation less than 25 percent). MRC showed the highest deflections, followed by MRG and then MRS. This order was counter to expectations because MRC had a crushed aggregate subbase course and would normally be expected to have a higher stiffness than the MRG pavement built directly on the subgrade. Pretraffic measurements of subgrade strength in test pits excavated for material characterization 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 CBR at the surface of the subgrade in the MRC test pits was 4, whereas at 1 foot below the surface it was approximately 8. The surface of the subgrade in the MRG and MRS test pits ranged from 7 to 8 CBR, as constructed. The order of failure of the rubblized test items also followed the order of the HWD deflection magnitudes. A detailed discussion on the HWD test results is presented elsewhere (7).

Trafficking started on July 7, 2005, and continued until October 6, 2005, following the schedule in Table 1. A fixed wander pattern was applied to the traffic during the tests. The wander pattern consisted of 66 repetitions, 33 traveling east and 33 traveling west. The transverse position of the gears was changed only at the start of the eastward repetitions. The wander pattern was designed to simulate a normal distribution with standard deviation of 30.5 in (775 mm) (equivalent to a taxiway distribution for design). The distribution of the transverse wheel positions is not random, but consists of nine equally spaced wheel paths at intervals of 10.25 in (260 mm). Additional details about the wander pattern can be found elsewhere (8). The temperature of the asphalt varied between 66º and 85 º F (19 º and 29 º C) during the test period. The average temperature of the asphalt was about 78 º F (26 º C).

Traffic testing was continued until either structural failure occurred, or it was estimated that failure was unlikely to occur within a reasonable number of passes at the applied load. The failure criterion was the presence of at least 1 inch (25.4 mm) of surface upheaval adjacent to the traffic lane. This is the same as the criterion used by the U.S. Army Corps of Engineers in previous full-scale tests of flexible airport pavements and is indicative of shear failure in the subgrade. During the traffic tests, the test items were monitored through a combination of visual surveys and nondestructive testing, including periodic straightedge rut depth measurements, surface profile measurements, and HWD deflection measurements. A 16-foot (4.88-m) long straightedge and laser profiler were used for rut depth measurements. In each test item, the rut depth 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 3 shows the rut depth measurements during traffic tests (as computed from transverse surface profile measurements). All the test items showed similar rut depths during the first 5,082 passes (55,000-lb wheel load, 4-wheel landing gear). Due to a minimal amount of distress (as shown in Figure 3), applied wheel loads were increased from 55,000 lbs (4-wheel dual-tandem landing gear) to 65,000 lbs (6-wheel triple-tandem landing gear). The 6- and 4-wheel configurations at increased loading both had the same dual and tandem spacings of 54 and 57 inches (137.2 and 144.8 cm), respectively.

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 (25 tonnes)	4-wheel, 55,000 lbs (25 tonnes)
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 (29.5 tonnes)	4-wheel, 65,000 lbs (29.5 tonnes)
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 (29.5 tonnes)	4-wheel, 65,000 lbs (29.5 tonnes)
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 (29.5 tonnes)	4-wheel, 65,000 lbs (29.5 tonnes)
09/13/05 10/06/05	16,303 25,608	MRG-N, MRS-N MRG-S, MRS-S	6-wheel, 65,000 lbs (29.5 tonnes)	4-wheel, 65,000 lbs (29.5 tonnes)

* Cold, unloaded tire pressures: 220 psi (1.52 MPa) at 55,000 lbs (25 tonnes) and 260 psi (1.79 MPa)at 65,000 lbs (29.5 tonnes).
** After the localized failure in MRC-NE (northeast portion of the test item), only northwest portion (MRC-NW) of the test item was trafficked.

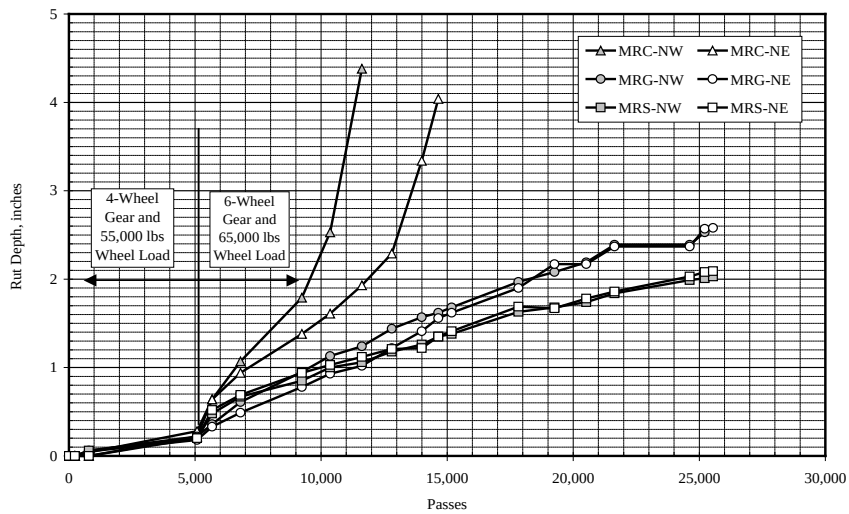


FIGURE 3 Rut depth measurements in rubblized concrete test items (1 inch = 25.4 mm).

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 passes, the rut depth measurements (from straightedge) were 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 and are shown in Figure 3. 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 4).

FIGURE 4 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 tracks but MRS-N did not.

6 POST-TRAFFIC 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 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 post-traffic 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. After removing the P-401 HMA surface, the rubblized concrete layer was exposed in all 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 taken on the surface of the P-154 layer. In the MRG trench (on the subgrade surface) the tests included CBRs, insitu density measurements (drive cylinder), and plate load. Only the plate load tests were performed on top of the P-306 econocrete layer in the MRS trench. After removing MRC P-154 subbase, CBRs and plate load tests were performed, 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, sand cone, and CBR tests were performed. CBR and plate load tests were performed, and density measurements were taken on the subgrade surface after removal of the P-154 subbase. In all the trenches, CBRs and density measurements were also taken at a depth of 1-foot (305 mm) below the subgrade surface. After completing the test, 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 (ASTM D 4429).

7 TEST RESULTS

The test results from different pavement layers in the four 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. Pre-traffic/pre-overlay 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 CBR at the subgrade surface in the MRC test pits was approximately 4, whereas the CBR at 1 foot (30 cm) below the subgrade surface was approximately 6 to 8. The MRG subgrade surface CBR was high (about 11). It was assumed that this was due to water being drawn from the subgrade (since the 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 pre-traffic 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 (87.26 kPa/mm)) than the k value outside the traffic path test (k = 457 pci (123.85 kPa/mm)). 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 (211.38 kPa/mm)) than the k value outside the traffic path test (k = 579 pci (156.91 kPa/mm)).

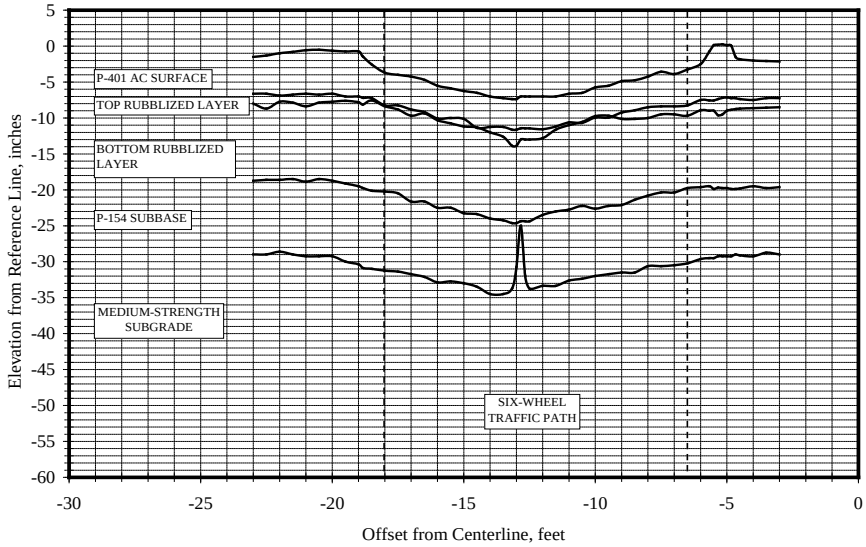
Table 2. Summary of Test Results from Trenching Study

Test Item	Trench ID	Layer Type	Test Type	Test Results: Inside Traffic Path	Test Results: 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 (MC)	5 (33.5%)	4 (34.5%)
			In-Situ Dry Density	89.4 pcf	88.2 pcf
		1-foot Below Subgrade Surface	CBR (MC)	7 (31.3%)	6 (31.6%)
			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 (MC)	4 (36.1%)	3 (38.5%)
			In-Situ Dry Density	89.4 pcf	86.8 pcf
		1-foot Below Subgrade Surface	CBR (MC)	9 (30%)	8 (30.7%)
			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 (MC)	11 (30.6%)	11 (30.5%)
			In-Situ Dry Density	91.7 pcf	92.9 pcf
		1-foot Below Subgrade Surface	CBR (MC)	8 (31.5%)	8 (31.5%)
			In-Situ Dry Density	92.0 pcf	91.5 pcf
MRS	MRS	Rubblized Concrete	Plate Load Test	780 pci	579 pci
		P-306 Econocrete Sub-base	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 (MC)	7 (32%)	6 (33.4%)
			In-Situ Dry Density	91.3 pcf	90.7 pcf
		1-foot Below Subgrade Surface	CBR (MC)	10 (30.4%)	9 (30.2%)
			In-Situ Dry Density	90.0 pcf	89.7

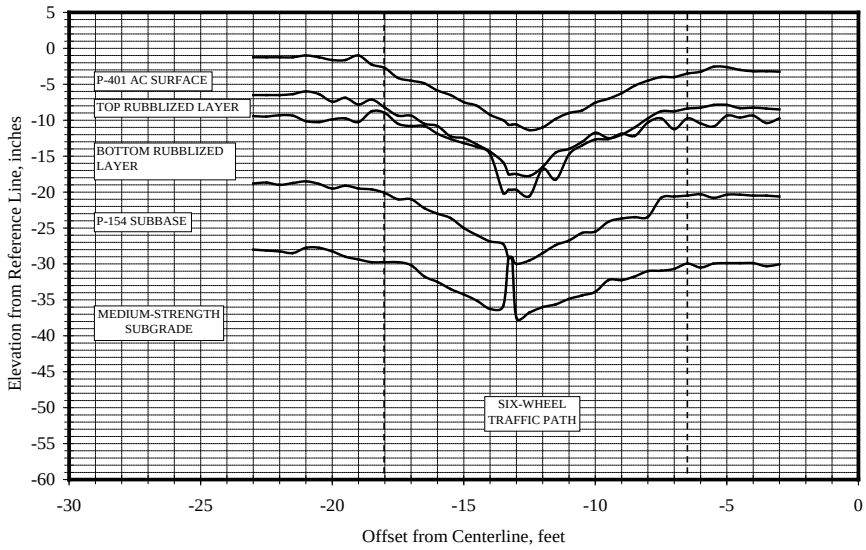
MC – Moisture Content
As Constructed Moisture Content for subgrade – 30.7%.
1 pcf = 16.018 kg/m^3
1 pci = 0.271 kPa/mm

8 LAYER PROFILES

After completing the tests, 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 5 shows the layer profiles in test item MRC.



a. MRC-W trench



a. MRC-E trench

FIGURE 5 Pavement layer profiles from trenches in test item MRC (1 inch = 25.4 mm).

The figure shows that the subgrade 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 6 and 7 show the pavement layer profiles for test items MRG and MRS respectively. It is observed 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 the 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 the rubblized layer were composed of 4 (10.16) to 15-inches (38.1-cm) concrete pieces tightly locked.

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

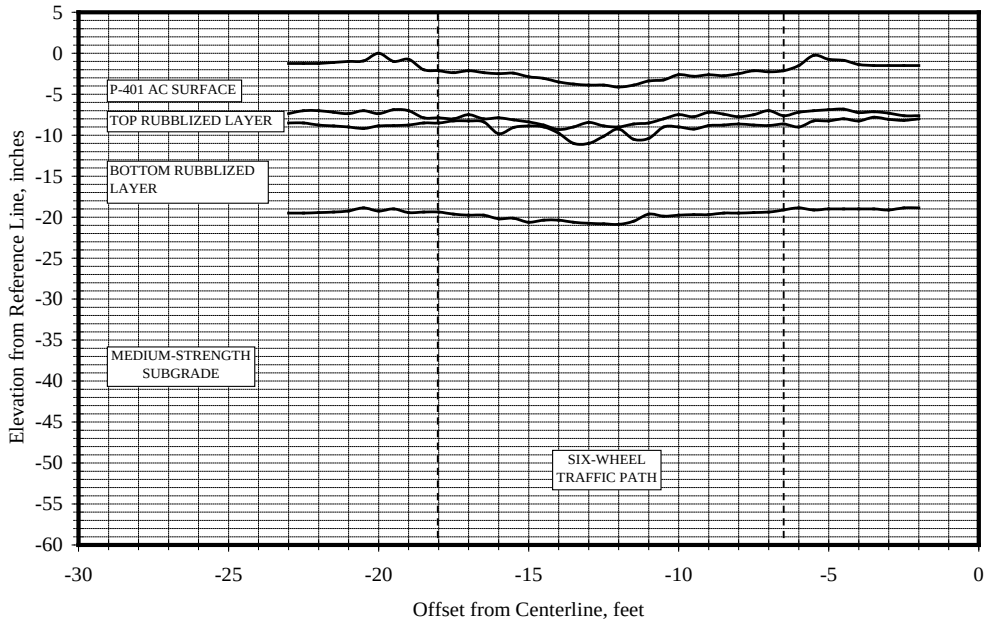


FIGURE 6 Pavement layer profiles from trenches in test item MRG (1 inch = 25.4 mm).

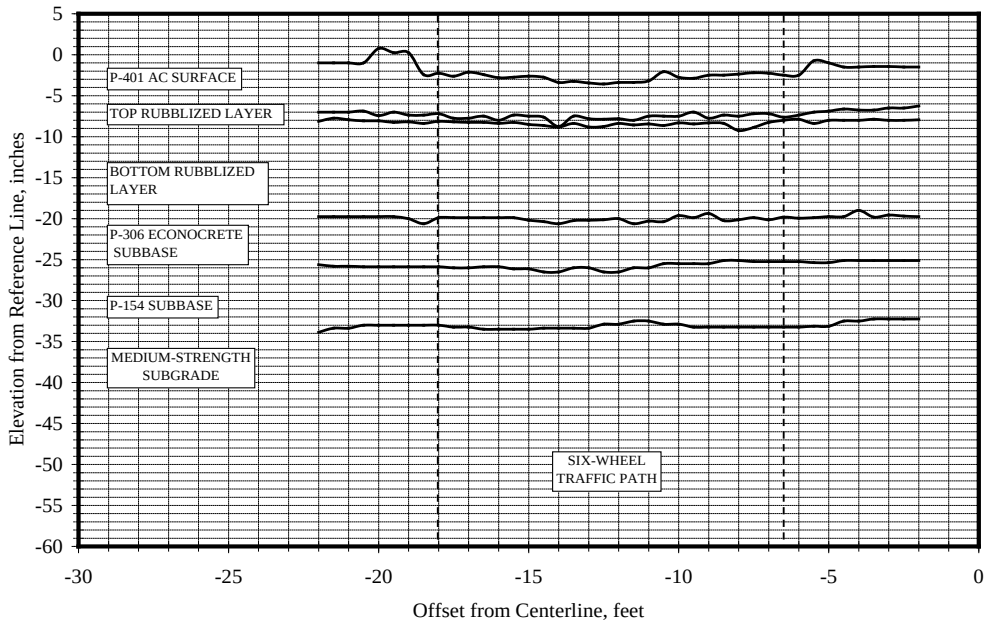


FIGURE 7 Pavement layer profiles from trenches in test item MRS (1 inch = 25.4 mm).

9 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 3, 4). 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 8 shows the photographs of trench faces in test item MRC and close-ups of the failure zones. Figures 9 and 10 show photographs of the 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 8. Photographs from MRC Trenches.

Figure 9. Photographs from MRG Trench.

Figure 10. Photograph from MRS Trench.

Excluding the top 3 inches (76 mm) 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). 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.
- any moisture/water migration from base/subbase into the subgrade.

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 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. Figures 6 and 9 (for MRG) and Figures 7 and 10 (for MRS) show 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 trenching study).

HWD tests were performed to study the variation in the backcalculated modulus of the rubblized concrete layer, impulse stiffness modulus, and the deflection basin shape factor AREA as the traffic tests progressed. The results indicate that backcalculated modulus of rubblized concrete may not be a good predictor of pavement performance (see reference 7 for a detailed discussion on this topic).

10 SUMMARY/CONCLUSIONS

Full-scale traffic tests were completed on three rubblized rigid airport pavements that were overlaid with 5 inches (12.7 cm) of HMA. This paper presents 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 under the same load as the original test items were tested. 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 was probably suffering severe 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 post-traffic 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 ("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.") are overly conservative.

For commercial airports serving wide-body aircraft (gross weights >100,000 lb), as per AC 150/5320-6E, 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 at 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.

A thickness design procedure for HMA overlays over rubblized concrete pavement is now incorporated in FAARFIELD (FAA airport pavement thickness design procedure, AC 150/5320-6E). The recommended modulus values for the rubblized concrete layer range from 100 ksi (689.5 MPa) to 400 ksi (2758 MPa). Some engineering judgement is required for the selection of an appropriate modulus value. The following ranges are suggested for selecting a design modulus value of rubblized PCC on airfields:

- For slabs 6 to 8 inches thick:
  Moduli from 100 to 135 ksi
  (689.5 to 930.8 MPa)
- For slabs 8 to 14 inches thick:
  Moduli from 135 to 235 ksi
  (930.8 to 1620 MPa)
- For slabs >14 inches thick:
  Moduli from 235 to 400 ksi
  (1620 to 2758 MPa)

The selected value is influenced by considerations such as level of conservatism in the design, exact slab thickness within the above ranges, pre-rubblized PCC modulus, anticipated particle size, steel debonding conditions, and relevant historical data. For further insight into selecting a design modulus of rubblized PCC, see reference AAPTP 04-01 (9).

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