CHARACTERIZATION OF RUBBLIZED CONCRETE AIRPORT PAVEMENTS AT THE NAPTF USING NON-DESTRUCTIVE TESTING METHODS

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ABSTRACT

At the Federal Aviation Administration's (FAA) National Airport Pavement Test Facility (NAPTF), three rigid airport pavements (MRC, MRG, and MRS) with 12-inch thick concrete slabs on different support systems (slab on crushed stone base, slab on grade, and slab on stabilized base) were trafficked to complete failure using dual tandem (B-747) and triple dual tandem (B-777) landing gear configurations. All three test items were constructed on CBR 7 subgrade (DuPont clay). Test item MRC consisted of 12-inch concrete slabs over 10-inches of crushed stone subbase, MRG consisted of 12-inch concrete slabs over subgrade, and MRS consisted of 12-inch concrete slabs over a 6-inch econocrete subbase. The north sides of the test items were rubblized with a resonant pavement breaker. After rubblization, the rubblized concrete was rolled and paved with a 5-inch thick HMA (hot mix asphalt) overlay. The overlaid pavements were subjected to full-scale accelerated traffic tests under the 4-wheel landing gear configuration (with wander) and 55,000-lbs wheel load. No significant distresses were observed for 5000 passes after which the wheel load was increased to 65,000-lbs and 6-wheel landing gear was used for testing. Heavy-weight deflectometer (HWD) tests were routinely performed using FAA's KUAB HWD equipment three different load levels – 12,000, 24,000, and 36,000 lbs. Portable Seismic Pavement Analyzer (PSPA) was used in conjunction with the HWD to estimate the asphalt concrete modulus. Moduli for the rubblized concrete layer were backcalculated using FAA's BAKFAA software. This paper summarizes the pavement structure uniformity within a given test item from HWD tests, and changes in the modulus of rubblized concrete layer with deterioration in pavement structure backcalculated using BAKFAA. Pavement performance during the traffic tests is also described.

INTRODUCTION

Rubblization of deteriorated concrete pavements is fast becoming a popular method of pavement rehabilitation. 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. Engineering Brief (EB) 66 [2] summarizes the guidelines for rubblized Portland Cement Concrete base courses. These guidelines are based on industry experience and provide interim guidance. 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 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 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. 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.

Heavy-weight deflectometer (HWD) tests were routinely performed using FAA's KUAB HWD equipment at three different load levels – 12,000, 24,000, and 36,000 lbs. 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's BAKFAA software. This paper summarizes the pavement structure uniformity within a given test item from HWD tests, and changes in the modulus of the rubblized concrete layer with deterioration in pavement structure backcalculated using BAKFAA. Pavement performance during the traffic tests is also described.

PAVEMENT STRUCTURES

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.

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 with twenty 15 by 15 foot by 12-inch thick concrete slabs. 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

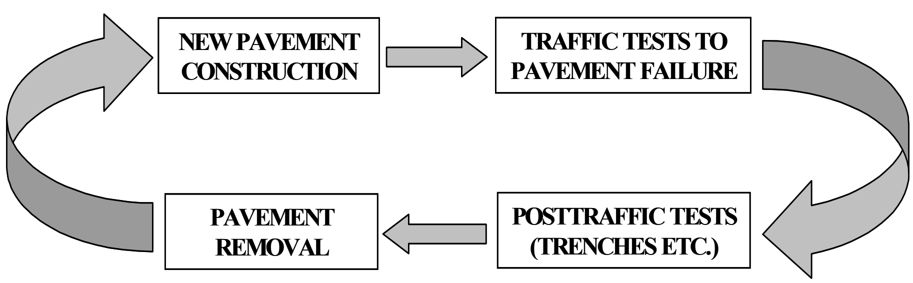

Figure 1. Construction Cycle at the NAPTF.

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 of P-401 hot mix asphalt. Figure 2 shows the pavement cross sections after the placement of the HMA overlay. 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.). Details along with photographs are provided in [4].

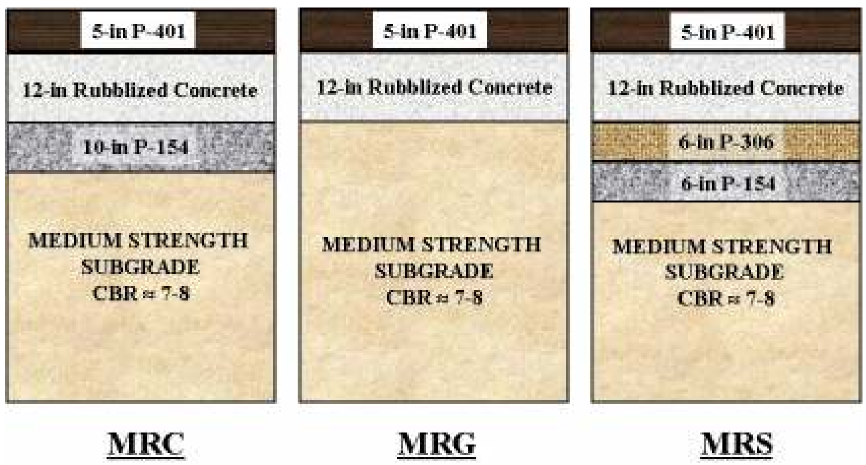

Figure 2. CC2-Overlay Pavement Test Sections.

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.

UNIFORMITY OF PAVEMENT STRUCTURES

Heavy-weight deflectometer (HWD) tests were performed using the FAA's HWD equipment on a 10-foot grid to study the uniformity of the pavement structures. Tests were performed with a 12-inch diameter plate at three different load levels – 12,000, 24,000, and 36,000 lbs. 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 to 25 percent. For deflection D7 (at 72-inch offset from the center of the load plate, and an indicator of subgrade condition), the COVs were around 10 percent. Figure 3 shows that the mean peak center deflections (D0s) for the rubblized test items were larger than the D0 deflections for the unrubblized 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 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 surface of the subgrade in the MRC test pits had a strength of approximately 4 CBR whereas the strength one foot below the surface was approximately 8 CBR. The surface of the subgrade in the MRG and MRS test pits was in the range 7 to 8 CBR, as constructed. The order of failure of the rubblized test items also followed the order of the HWD deflection magnitudes.

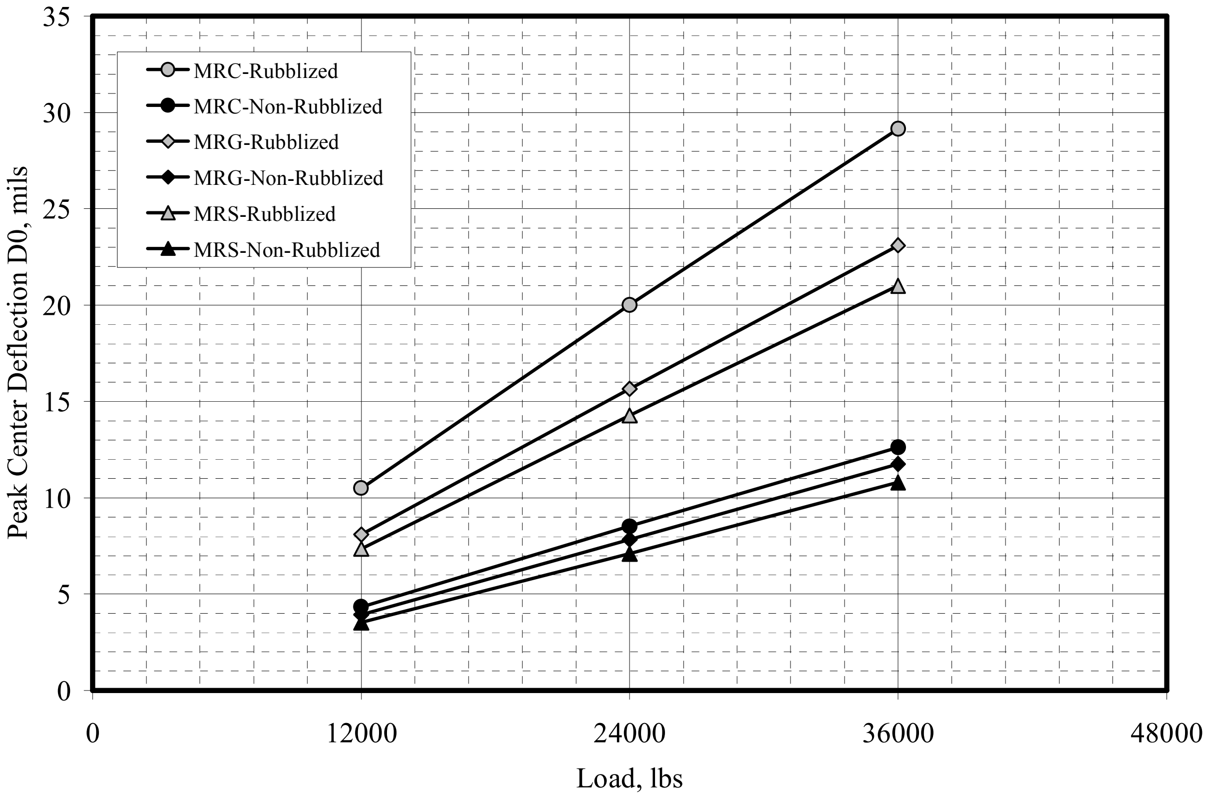

Figure 3. Mean Peak Center Deflections D0 from Uniformity Tests.

Figure 4 shows deflections D7 (at 72-inch offset from center of plate) that are indicative of subgrade stiffness. Figure 4 is further indication that the subgrade of MRC was of lower stiffness than the subgrade of MRG and MRS.

Figure 5 shows the AREA for rubblized and non-rubblized test items. AREA is the area of the deflection basin normalized with respect to D0 and is a deflection basin shape factor [7]. 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.

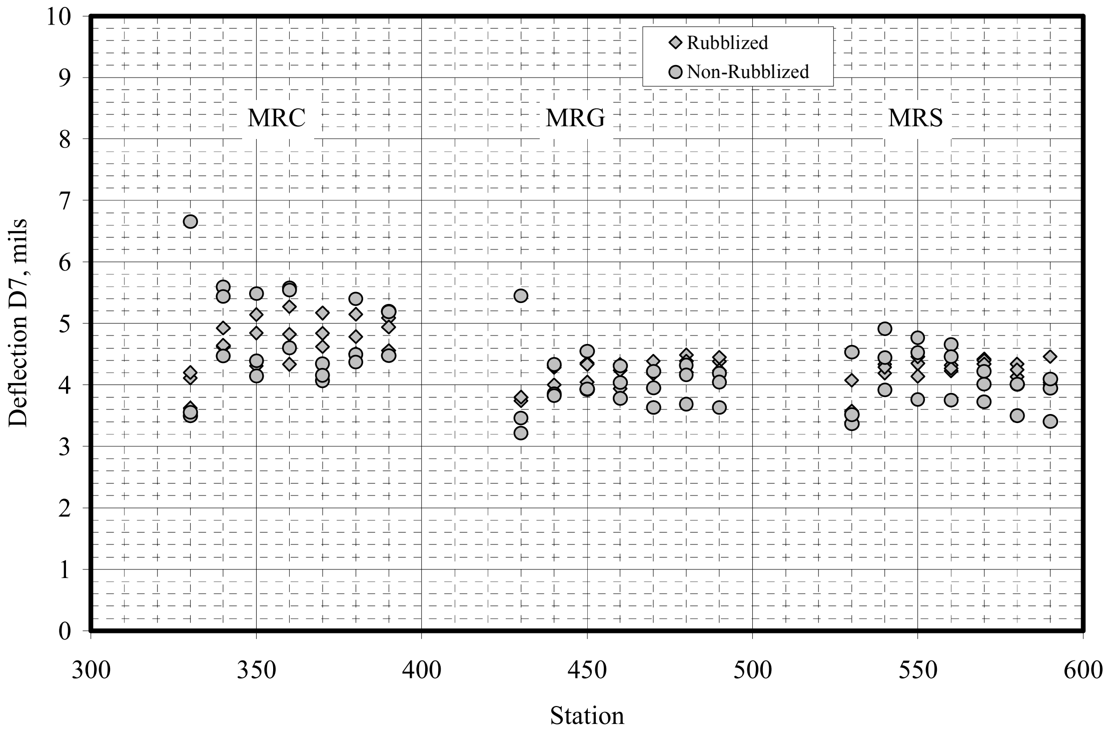

Figure 4. Deflections D7 from Uniformity Tests.

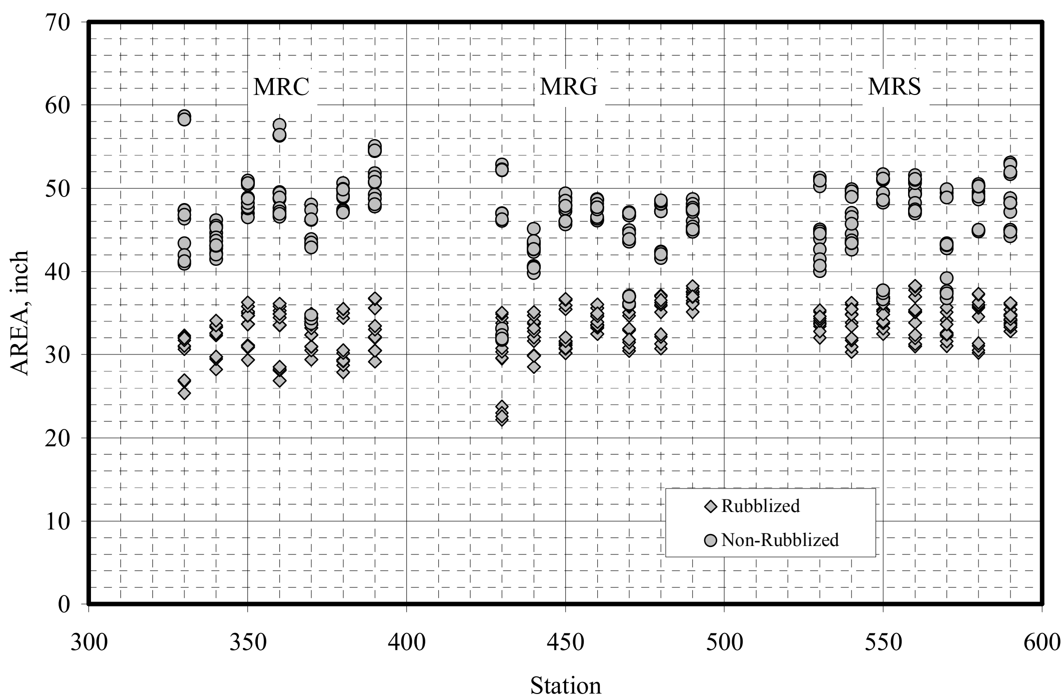

Figure 5. AREA from Uniformity Tests.

TRAFFIC TESTING AND PAVEMENT PERFORMANCE

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 and tandem spacing of 57 inches. Wheel load was set at 55,000 lbs 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

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.

Figure 6 shows the rut depth measurements during traffic tests (as computed from transverse surface profile measurements). The NE end of MRC was the first area of the rubblized pavements to show signs of failure and exhibited complete structural failure in an area of pavement reconstructed over a pretraffic material characterization test pit. 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.

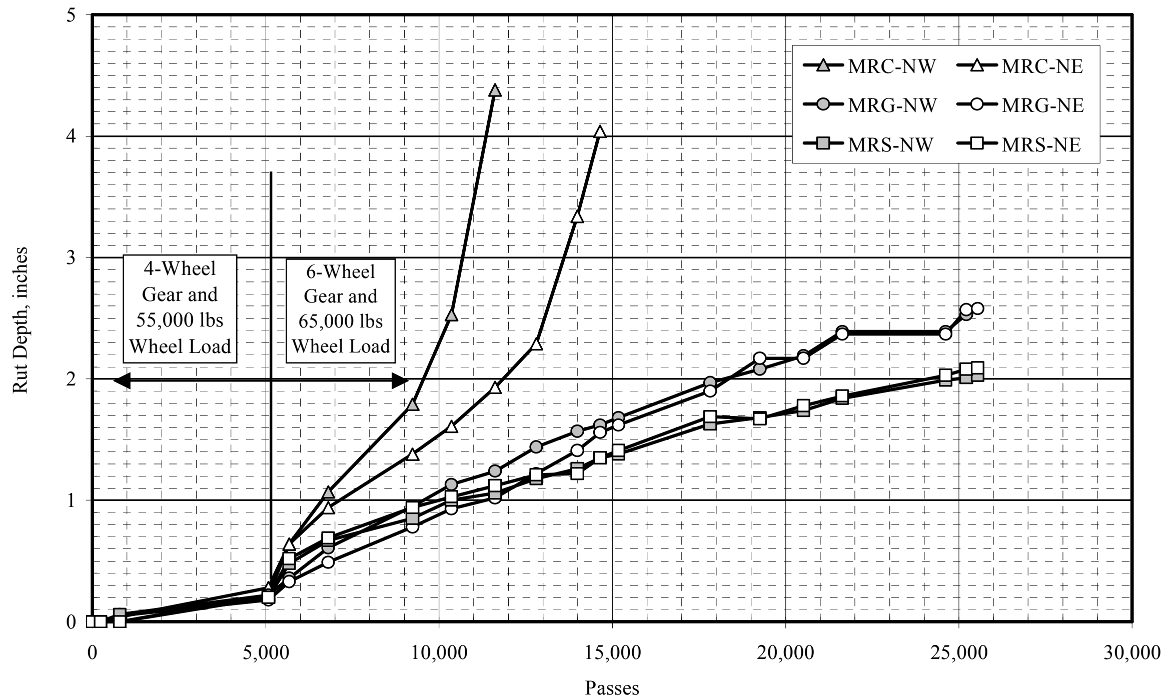

Figure 6. Rut Depths in the Rubblized Concrete Test Items from Transverse Surface Profiles.

BACKCALCULATION OF RUBBLIZED CONCRETE MODULUS

Heavy-weight deflectometer (HWD) tests were routinely performed using FAA's KUAB HWD equipment at three different load levels – 12,000, 24,000, and 36,000 lbs. A Portable Seismic Properties Analyzer (PSPA) was used in conjunction with the HWD to estimate the asphalt concrete modulus. 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-feet (inside trafficked area) and 5-feet (outside trafficked area) offsets north of centerline. Moduli for the rubblized concrete layer were backcalculated using FAA's BAKFAA software. The CBR test results from posttraffic testing (trenches) on subgrade (table 2) 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 10-feet 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

Table 2.
Results from Posttraffic 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	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

divided by peak center deflection D_0. The deflection basin shape parameter AREA was also computed. More details about backcalculation, ISM, and AREA can be found in the Advisory Circular 150/5370-11A [8].

Figures 7, 8, and 9 show the variation in the backcalculated modulus of the rubblized concrete layer, ISM, and the deflection basin shape factor AREA inside the trafficked area as the traffic testing progressed.

Figure 7 shows that the rubblized concrete modulus reduces to approximately 30 percent of the initial modulus value after 330 passes for all of the three test items. The pavement performance as indicated by rut depth does not show a decline of this magnitude. In fact, for the first 5000 passes, the maximum rut depth in all of the three test items is about 0.25-inches. After 5000 passes, the modulus drops down 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 (see further discussion below).

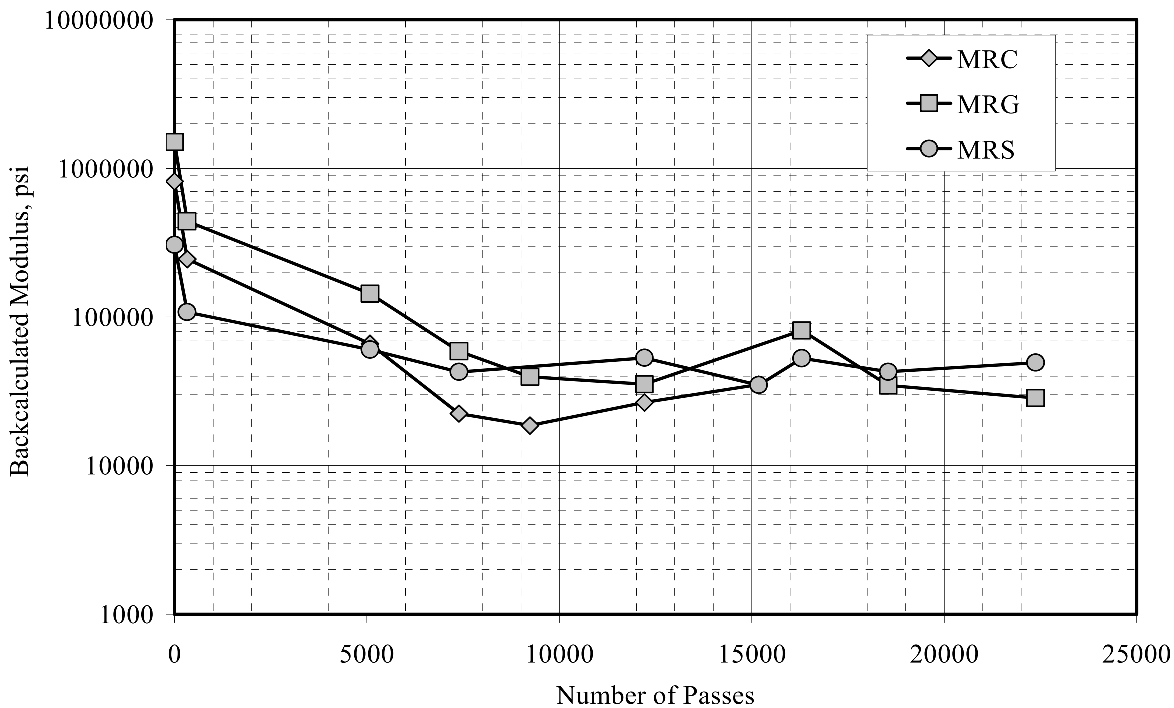

Figure 7. Changes in the Modulus of Rubblized Concrete During the Traffic Tests.

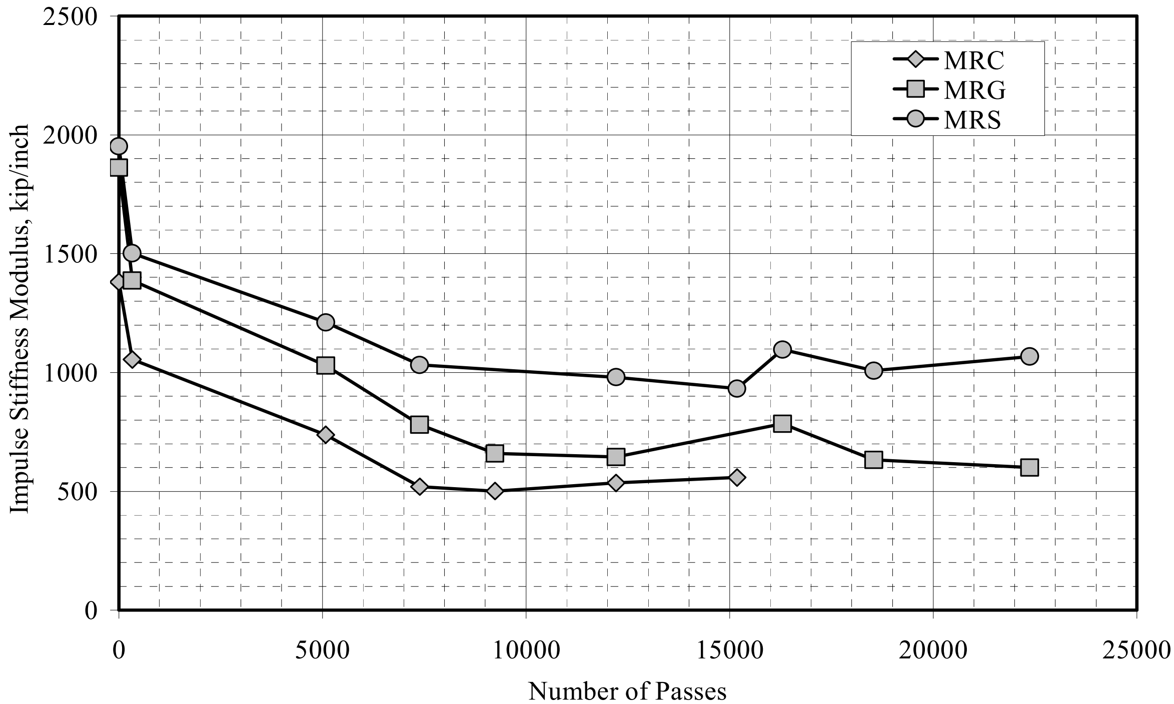

Figure 8. Changes in the Impulse Stiffness Modulus (ISM) During the Traffic Tests.

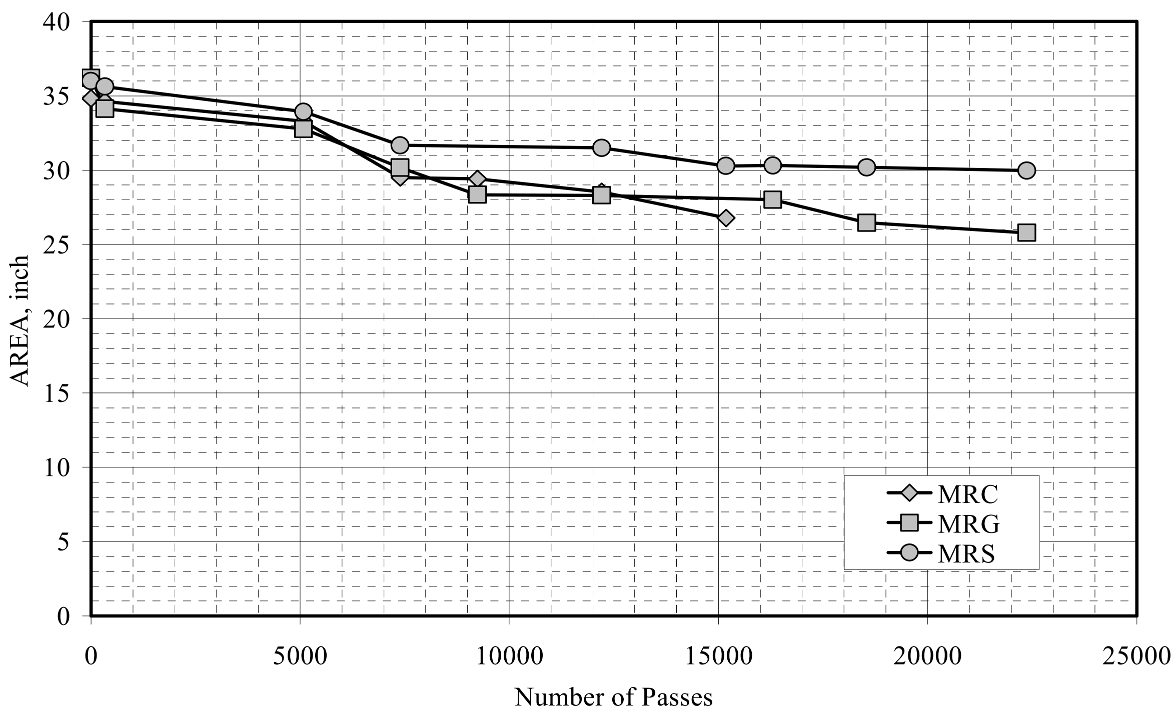

Figure 9. Changes in the Deflection Basin Shape Factor AREA During the Traffic Tests.

The results from posttraffic testing in the trenches show that, in test item MRC, the failure in the subgrade resulted in failure of the pavement structure, including the full depth of the rubblized layer. In test item MRC, the top 3 to 4 inches of the 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. Profile measurements showed that the top of the rubblized layer (3 inches of finely rubblized material) contributed to the rutting. 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 was 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.). As can be observed from figure 10 (for MRG) and figure 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.

The test pits opened in the rubblized concrete layers prior to the placement of the HMA overlay showed that, in general, the top 2 to 3 inches in all of the test items was reduced to dust and stones with a top particle size of 1 inch. The particle size in the bottom 9 inches ranged from 4 inches to 15 inches with larger particle sizes in MRS. The test pits showed that the rubblization process induced angled cracks, or, more accurately at this stage, fractures, through the entire depth of the slabs and that the cracks were tightly held. This interlock deteriorated under repeated wheel loads, as shown in the posttraffic trenches. The rate of deterioration is controlled by various factors. Some of the important ones are the magnitude and wander of wheel loads, loss of confinement due to fatigue cracks in the HMA overlay layer, and loss of confinement due to weak support system (underneath the rubblized concrete layer) allowing high vertical deflections in the pavement structure. 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 backcalculated modulus values lower than expected. The HWD tests inside the traffic path were centered over an underlying dowelled longitudinal joint. During the posttraffic trenching study, it was observed that the rubblization process did not debond the dowels from the two adjacent slabs and the size of concrete pieces ranged from 3-feet to 4-feet in length and width. It is quite possible that this may have contributed to the higher peak center deflections rather than the deterioration of the rubblized concrete layer. However, all these factors did not significantly affect the performance of the rubblized layer as was observed from the rut depth measurements.

Figure 9 shows the changes in deflection basin shape factor AREA during the traffic tests. In order to illustrate the procedure used to calculate the AREA shape factor, figure 12 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 10. Photograph of MRG Trench.

Figure 11. Photograph of MRS Trench.

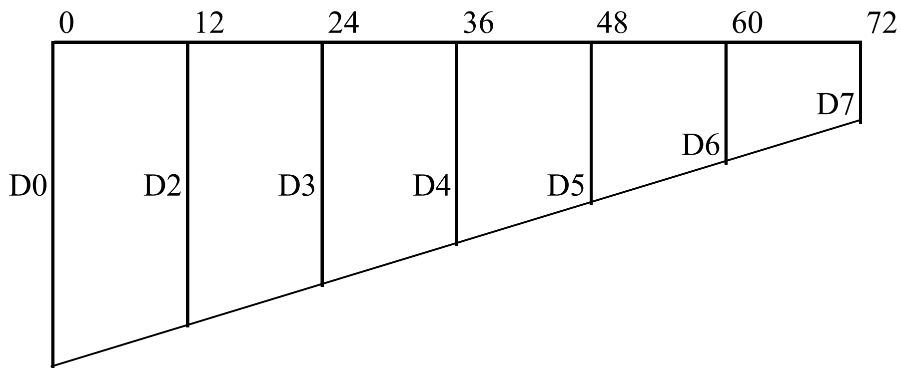

Figure 12. Typical Deflection Basin Under HWD Load.

AREA is the area of the deflection basin after all of 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 55,000 lbs wheel load), all of the three test items showed similar rut depths. The AREA values (figure 9) 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 north east area of the test item due to large rut depths as that part of the test item deteriorated towards 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 10 and 11) showed no significant signs of deterioration at the end of trafficking.

PAVEMENT LIFE COMPUTATIONS

According to EB-66 [2], "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 2) at the top of the subgrade and a depth 1-foot below the subgrade surface. The design CBR values were computed in similar way as in the new alpha factor report [9]. The results are summarized in table 3.

Table 3.
Predicted Life Computations Using LEDFAA-1.3.

Test Item	Pavement Life, passes 4-Wheel 55,000 lb Wheel Load	Pavement Life, passes 6-Wheel 65,000 lb Wheel Load	Observed Pavement Life, passes
MRC	236	29	14652
MRG	42	10	25608*
MRS	385418	6343	25608**

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

The traffic tests for the first 5068 passes were performed at 55,000 lbs wheel load and 4-wheel gear. After that, the traffic tests were performed at 65,000 lbs and 6-wheel gear (Table 1). Comparing the observed pavement life (figure 6) and predicted pavement life (table 3), 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/CONCLUSIONS

The results from non-destructive tests and full-scale traffic tests on three rubblized concrete pavements which had been overlaid with five inches of HMA are presented. The main objective of this paper was characterization of rubblized concrete for design. Performing any type of strength tests on the rubblized material is very difficult (if not impossible) because of the nature of the material. HWD test data were utilized to compute an (effective) modulus of the rubblized concrete layer by backcalculation. The backcalculation of the rubblized concrete modulus yielded values which 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 atypically high peak center deflections 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, are 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 EB66 are overly conservative.

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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