TECHNICAL MEMORANDUM

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Date: 09/29/2021	**Study No: 47-1XXIA003, 0000024741**
To: Pavement Analysis and Design Branch	**Tech Memo No:** NTP 22-01-1
From: Research Team	**Revision No:**
Subject: **Pavement Evaluation of Roller Compacted Concrete Pavement in Leakey**	

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Introduction

This technical memorandum is not intended to provide in-depth information on RCC pavement; rather, its objective is to present the analysis results on the condition of the RCC pavement on US 83 and RM 377 in Leakey in the San Angelo District. Before presenting the analysis results of the field testing conducted, general information on RCC pavement is presented first.

Roller-compacted concrete (RCC) pavement is regarded to be cost saving, faster construction and traffic opening, and having a sound structural capacity. The first use of RCC pavement built that was reported in US was in a test section at the Waterways Experiment Station in Vicksburg, Mississippi, in 1975, and its use exceeded 13 million square yards by 2011 (Pittman and Anderton, 2012). Because of the characteristics of its mixture, RCC pavement is constructed without dowel bars, tie bars or reinforcing steel for load transfer; rather any load transfer at cracks and joints is to be achieved by aggregate interlock and slab support system. How effective this aggregate interlock could be for load transfer and how long it will last under heavy traffic are still unknown, since the information available on these is quite limited, except that RCC has been used more widely in parking areas and port pavement where traffic moves at a relatively slow speed. Harrington et al (2010) state that:

> As a result, there is a significant amount of concrete that can be placed continuously within a shorter period of time. With these benefits, the RCC pavements are excellent alternative for parking and storage areas, highway shoulders, streets and highways.

RCC is a hydraulic cement concrete such that its properties are just like the conventional concrete, since it is a mixture of the cement, aggregate, water and admixtures. FHWA (2016) reported that RCC should have a "negative" slump where the presence of slump in the mixtures is excessive. Hence, it should be more rigid than a zero-slump concrete such that it is stiff to resist segregation during vibration. At the same time, it should be adequately wet to facilitate mixing and enable sufficient distribution of cement paste.

Due to the mixture's consistency, it is usually placed using an asphalt-type paver with screed and is, subsequently, followed by roller compaction with a short time window that is dependent on the cement hydration. This poses a challenge in the construction procedures since RCC pavement needs to achieve high densities (e.g., 98% of modified Proctor maximum dry density). LaHucik and Roesler (2017) reported that cores extracted from RCC pavements, whose densities were about 4% less than those of the laboratory samples, were 45% lower in compressive strengths. The laboratory samples require 50-70 gyrations in order to achieve a 98% compaction of the modified Proctor maximum dry density. In addition, previous studies reported that the mechanical properties and durability of the RCC are affected by density (Pittman, 1989; Shihata, 2000; Delatte and Storey, 2005; Harrington et al, 2010). Other reports also state that the mechanical properties of RCC such as elastic modulus and flexural strength will be compromised if the required density is not achieved (LaHucik and Roesler, 2010). In addition, there are reports that show the density of the RCC material decreases with depth from the surface of the RCC slab (Nanni and Johari, 1989; Nanni et al, 1996; Pitman, 1989). To summarize, RCC mechanical properties are significantly affected by density, and achieving proper density during RCC pavement construction is critical for satisfactory performance of RCC pavement. In addition, density could vary through the RCC slab depths, which might compromise RCC pavement performance unless it is properly addressed during the construction.

Because the RCC mechanical properties are somewhat different from those of Portland cement concrete (PCC), the behavior and structural responses of RCC pavement (RCCP) are expected to be different from those of normal PCC pavement. Pittman and McCullough (1997) reported that, during the start of the large-scale construction of RCCP in the United States in the early 1980s, natural cracking was allowed to develop instead of providing saw-cut joints since jointing was not necessary for industrial applications where this pavement was used at that time. However, as the extent of the RCCP application has progressed, the drive towards the installation of the sawcut joints has intensified. It was reported that the naturally occurring cracks had a spacing between 30 to 100 feet depending on slab thickness and other conditions, and the crack widths went up to 0.5 in. Larger natural crack interval was attributed to the low amount of water in the mixture, which yields lower drying shrinkage of the RCC. Sok et al (2018) reported that, in South Korea, the crack spacing was even larger, between 70 to 121 feet. Accordingly, in the absence of load transfer devices (e.g. dowel bars), larger crack spacings and wider crack openings will not engage the aggregate interlock and, thus, reduces the load transfer efficiency at the cracking location, which questions the reasonableness of the practice of not utilizing dowels with the assumption of a high level of aggregate interlock. Pittman (1996), based on field evaluations, concluded that the joint load transfer efficiency (LTE) was between 22-89% which varied with joint types or crack spacing. LTE values were significantly reduced with larger crack spacing, greater crack widths and temperature decrease. He also reported that slab thickness, maximum coarse aggregate size, pavement age and slab support condition did not have significant effects on LTE of the RCCP. Larger crack widths and resulting low LTEs are expected to cause faulting in RCCP. However, Piggott (1999) presented a result of an RCCP performance, showing closely spaced natural cracks and no signs of faulting. Since there are a number of factors related to faulting development, such as LTE at joints or cracks, traffic, and slab support condition, no signs of faulting in one RCCP project should not be interpreted as RCCP's resistance to faulting. As will be discussed later, the major distress in RCCP evaluated in this study was faulting at transverse contraction joints, and resulting noise. As for the saw-cut depth in RCCP, the ACI Guide (2015) suggests that the sawcuts shall not exceed ¼ of its depth for RCCP with a joint spacing of 15 feet for slab thicknesses of up to 8 inches. Currently in TxDOT, the saw-cut depth for transverse contraction joints is 1/3 of the slab depth. This difference in saw-cut depths may not be due to the differences in RCCP and PCC pavement; rather, the range of required saw-cut depths in PCC pavement varies from ¼ to 1/3 of the slab depths and it appears that ACI adopted the smaller saw-cut depth.

RCC pavement was constructed on US 83 and RM 337 in Leakey in the San Angelo District, Texas in 2016. This is the only RCCP built in main lanes in highways managed by TxDOT. As of this techmemo writing, the pavement is 5 years old. To evaluate the overall and structural condition of this pavement, visual condition survey as well as deflection testing with FWD was conducted on September 13, 2021. This technical memorandum presents the findings of the field evaluations conducted on September 13, 2021 as well as previously.

Test Section Overview

The RCC pavements evaluated are on US 83 and RM 337 in downtown Leakey. The general information on this RCCP is shown in the table below.

Attribute	Information	Notes
District & Highway	SJT, US 83	RM 337 (0792-02-031)
CSJ	0036-05-033	
County	Real County	
Reference Marker		US 83, RM 337 L=2.055-mi
Project Limits	US 83: From RM 336 to FM 1120 RM 337: From 0.5 Miles West of US 83 to US 83	
GPS Coordinates		
Date Work Begin	October 26, 2016 (Concrete Pouring)	
Date Completed	-	
Pavement Type	Roller Compacted Concrete (RCC)	
Slab thickness	US 83: 8.0-in., RM 337: 7.5-in.	
Shoulder Type	RCC	
Base Type		
Subgrade Type	8-in. CTS (4% Cement)	
Drainage Type	Curb and Gutter	
Coarse Aggregate Type		
Con. Pavement Details	-	

The US 83 section is from RM 336 to FM 1120, with a total length of 2.055 miles. The RM 337 section is located west of US 83, with a total length of 0.5 miles. RCC slab thickness is 8-in and 7.5-in for US 83 and RM 337, respectively. According to the plan set, there was no base placed; instead, RCC was placed directly on subgrade treated with 4 % cement up to 8-in. However, the research team was informed by a TxDOT personnel that there was a base layer built with recycled materials. Since no coring was conducted during the field evaluations, the verification was made on the existence of a base layer. The width of the RCCP is 42-ft at both US 83 and RM 337, with one 12-ft wide lane in each direction and 8-ft and 10-ft outside shoulders. The concrete placement started in October of 2016. Saw-cuts were made at 15-ft spacing at transverse contraction joints, and longitudinal construction and warping joints were installed at the centerline and joint between main lane and shoulder, respectively.

Figure 1 shows the title sheet of the plan set and Figures 2 and 3 present typical sections of the RCCP for US 83 and RM 337, respectively.

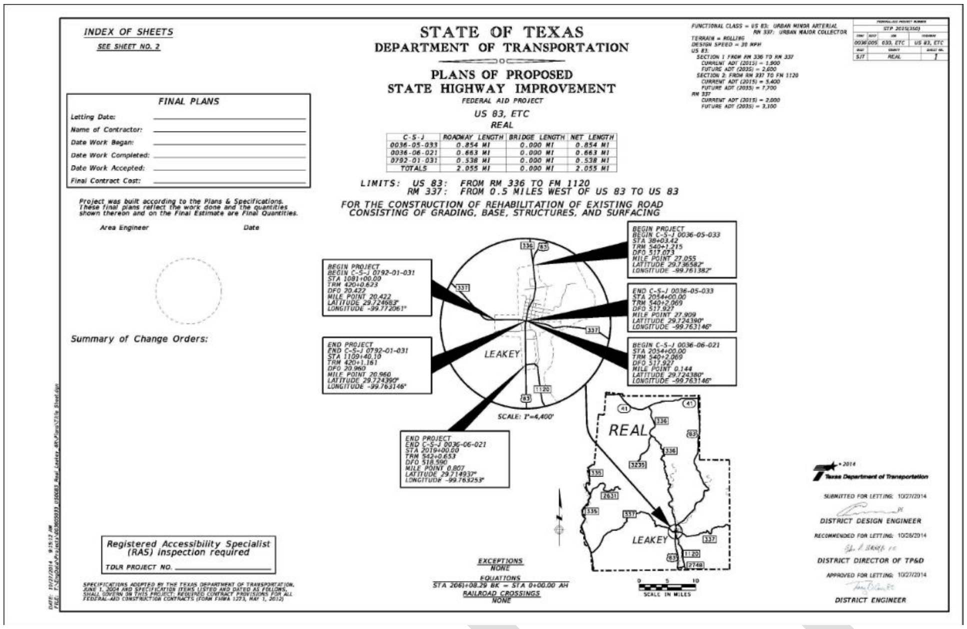


Figure 1 Title sheet

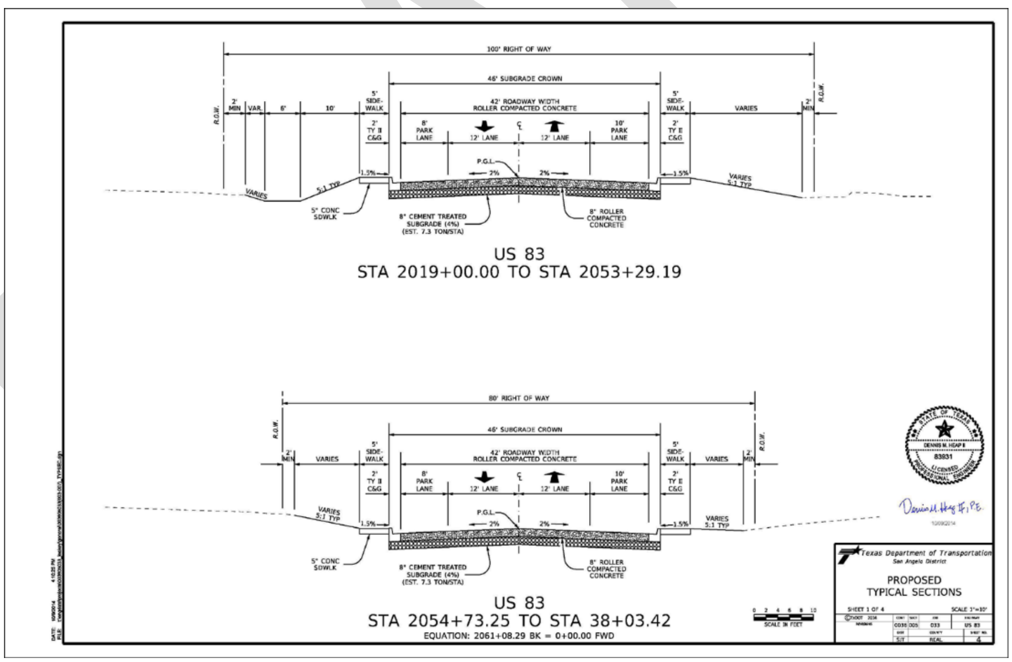


Figure 2 Typical section of US 83

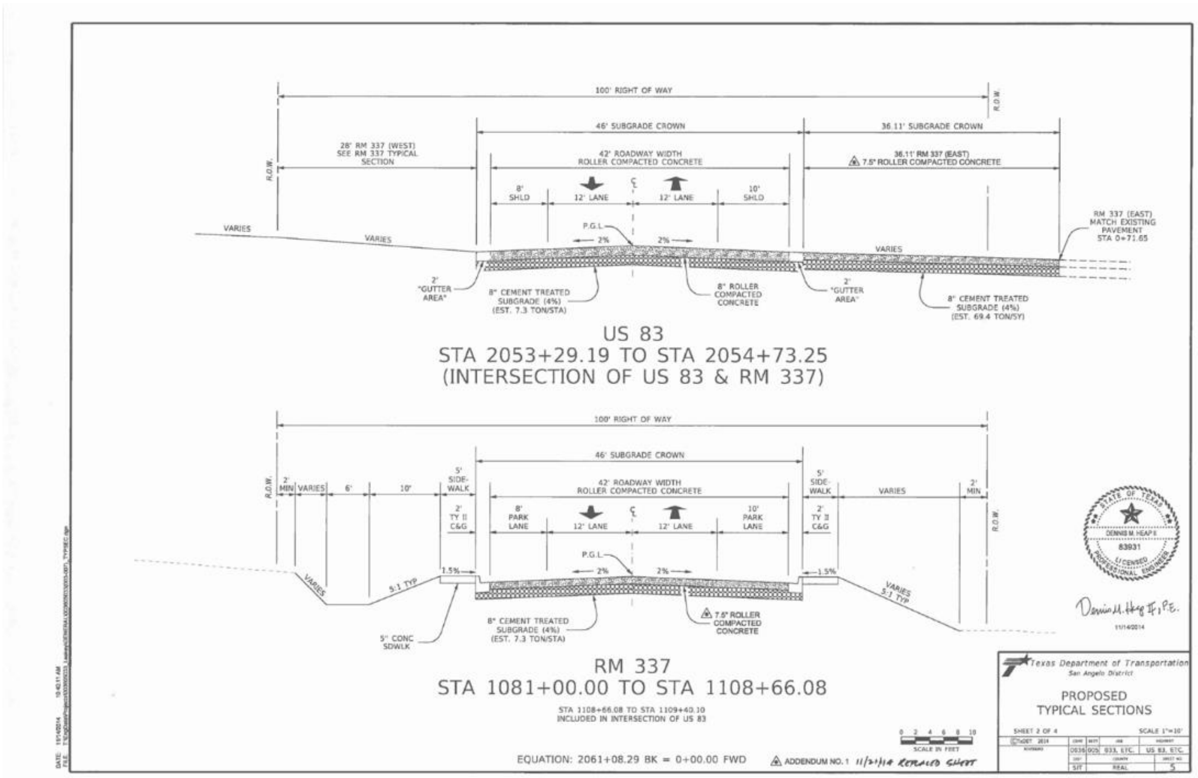


Figure 3 Typical section of RM 337

Overall Condition of RCCP

Figures 4 presents typical condition of the RCCP on US 83. Visual condition survey indicated few mid-slab and longitudinal cracking, as well as faulting at a number of transverse contraction joints in both US 83 and RM 337. Faulting was widespread throughout the projects, and noise from traffic driving over faulted joints was quite noticeable.

Major Distresses

Figure 5 shows a typical mid-slab transverse crack, while Figure 6 illustrates a typical longitudinal crack. Identifying the causes of mid-slab and longitudinal cracking was beyond the scope of this investigation; however, it appears that well-known mechanisms for both cracking were also at play over here, which will be discussed later.

Cracking in CPCD, either transverse or longitudinal, is considered as distress. However, CPCDs with cracking have performed satisfactorily, as long as the slab support is maintained. On the other hand, nationally, faulting has been a major issue in CPCD.

Figure 4 Present condition of RCC pavement on US 83

Figure 5 Typical mid-slab transverse crack on RCC pavement on US 83

Figure 6 Longitudinal crack on RCC pavement on US 83

In this RCCP project, cracking and faulting were two major distresses observed. Transverse cracking observed in this project is different from normal mid-slab cracking observed in CPCD. As discussed earlier, transverse joint spacing is 15-ft, and it is believed that stresses in RCCP due to temperature and moisture variations were well below tensile strength of RCC due to short joint spacing and low drying shrinkage. However, transverse cracks were observed that appeared to have developed from wheel loading applications. These cracks were not in the middle of the two adjacent transverse joints; rather, they occurred close to the joint right after trucks pass over it, as shown in Figure 7. Two transverse cracks are observed; one in the northbound, and the other in the southbound (the picture was taken facing east). The crack in the northbound lane – the lane on the other side of the centerline – is closer to the joint on the right, while the crack in the southbound lane is closer to the joint on the left. It appears that impact loading from truck traffic driving over a faulted joint caused excessive wheel loading stresses a transverse crack developed.

In CPCD, faulting is the most serious distress, since faulting impacts ride quality negatively as well as causes noise issues, while cracking does not necessarily affect ride quality as long as slab support is adequate. In general, if the average faulting is 2.5 mm (0.1-in) or greater, it causes noise and degrades ride quality. If the faulting is greater than 4.0 mm (0.15-in), the condition is considered poor and diamond grinding or other measures need to be performed. Accurate measurement of faulting requires specific tools, such as Georgia fault meter. The research team does not possess the device and faulting was not measured. However, faulting was not small in some areas, as shown in Figure 8. Even though the speed limit in this highway is low – 30 mph or 45 mph – noise was quite noticeable, which should not be acceptable to both traveling public and TxDOT.

Figure 7 Cracking in RCC pavement

Figure 8 Faulting at the joint in RCC pavement

Evaluation of Pavement Structural Condition

Since the completion of the RCCP construction, it appears that there have been several evaluations made on the condition of this RCCP section – one by TTI and the others by MNT. TTI research team evaluated RCC density from cores taken at US 83 section by CT scan. Figure 9 illustrates its findings, showing large variations in RCC density through the slab depth, with a good density near the top of the slab and quite large voids or low density near the bottom of the slab. The density starts to decrease from about 2-in from the top, with the voids greater than 20 % near the bottom of the slab. As discussed earlier, 4 % more void than in the laboratory prepared specimens in RCC resulted in 45 % reduction in compressive strength. In Figure 9, it is observed that void at 2.5-in depth is about 6 %. Even though this data is from only one core, with the assumption of this core representing the whole project, it could be concluded that the compressive strength of the RCC in this pavement section below 2.5-in from the surface could be significantly low – maybe less than a half of the strength of the top quarter of the slab. If fatigue cracking is considered as a failure criterion of RCCP in this project, as has been the case for CPCD, the life expectancy of this RCCP could be severely reduced. What may be working for this RCCP is low traffic for both US 83 and RM 337. The annual average daily traffic (AADT) in 2020 was 4,749 on US 83, with 86 % passenger vehicles. For RM 337, AADT in 2020 was 1,900 with 94 % passenger vehicles. On the other hand, it is somewhat disappointing that faulting occurred under this low traffic, which implies that the aggregate interlock at transverse joints was not able to effectively transfer loading from one slab to the next.

County: Real
Project:
Resp. Office: Junction AO

Highway: US 83
Limits: In Leakey, TX

Report Date: October 12, 2017
Field Coring Personnel: TxDOT

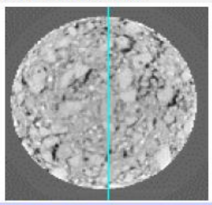
CT Scan Roller Compacted Concrete			
Dia. of Core:	6"	Core Location	**NB lane**
Comments:			

Air Void Determination - CT SCAN

Comments	Depth (in) From	Depth (in) To	Avg. Air Voids (%)
Top portion	0	2	1.0
AV increasing	2	3 1/2	5.7
AV increasing	3 1/2	7	11.9
Bottom	7	8	21.0

Specification requires 98% Density

CT Scan Composite Image



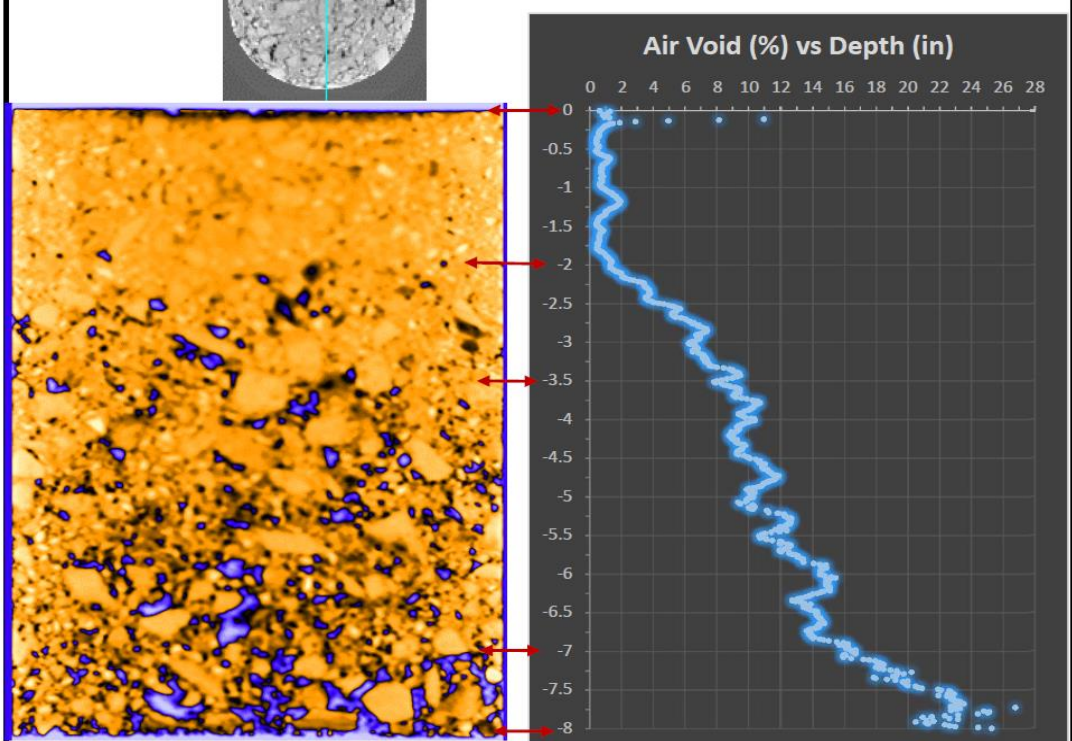


Figure 9-a First page of the CT scan report of the RCC core sample

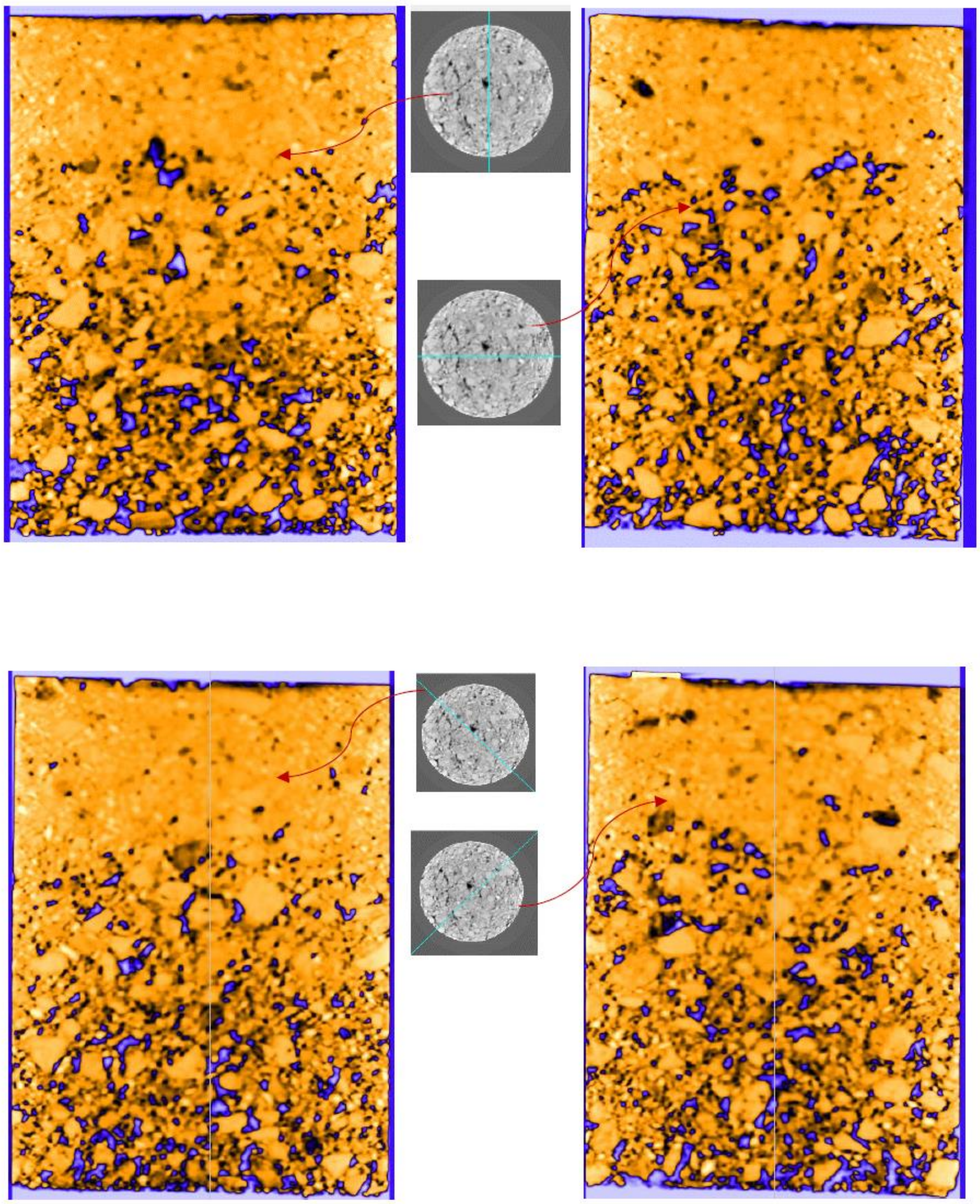
County: Real
Project:
Resp. Office: Junction AO

Highway: US 83
Limits: In Leakey, TX

Report Date: October 12, 2017
Field Coring Personnel: TxDOT

CT Scan Composite Image cut through center of Stack at different angles

Note: Black and white images are the top view of image stack



Texas A&M Transportation Institute

TTI – Materials and Pavements

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Figure 9-b Second page of the CT scan report of the RCC core sample

Slab support condition

The slab thickness on US 83 is 8-in, while that on RM 337 is 7.5-in. The average deflections at mid-slabs were 3.8 and 3.9 mils for SB and NB on US 83, respectively, and 3.7 mils for EB on RM 337. The average deflection for 8-in CRCP, collected from TxDOT Rigid Pavement Database project FWD, is about 4-mils. Accordingly, it could be stated that the slab support condition in the RCC pavement in Leakey is, compared with state-wide average, not necessarily deficient. To evaluate slab support condition, deflections measured at mid-slabs were analyzed using AREA method.

Figures 10-12 show that deflections at sawcut joints are higher than those at mid-slab. It is shown that the most deflections at the mid-slab across all 3 sections are 5 mils or less, while higher deflections were recorded at the joints/cracks. Intuitively, the larger deflections at cracks and joints than at mid-slabs indicate a low level of load transfer, which will be discussed in the next section of this memorandum. In addition, the deflections at mid-slab are small, as discussed above, indicating that the slab support under RCC slabs is satisfactory.

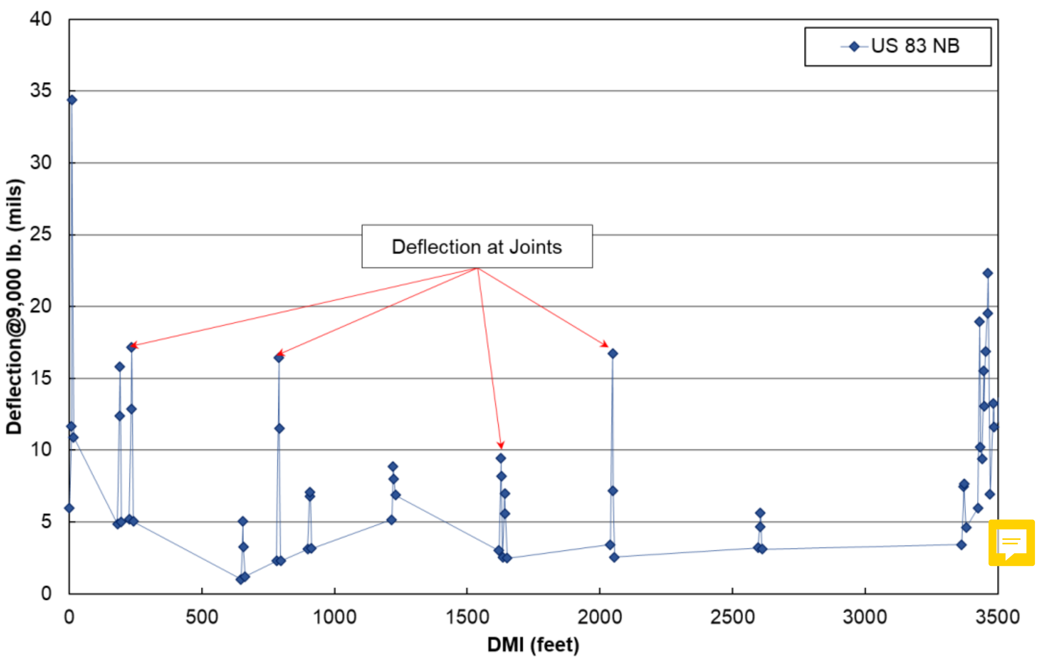

Figure 10 FWD deflections at US 83 northbound

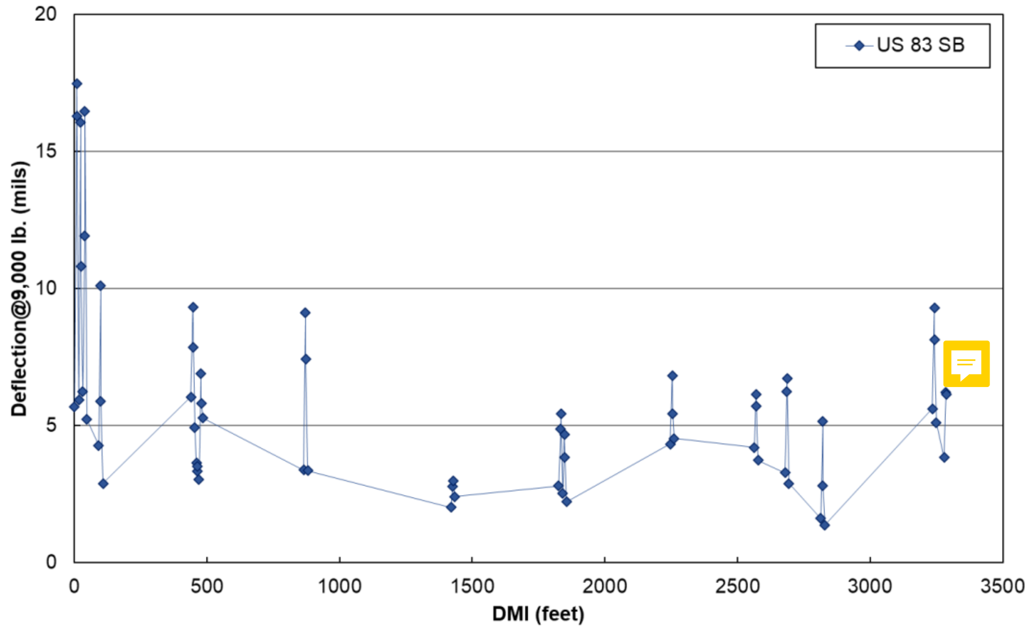

Figure 11 FWD deflections at US 83 southbound

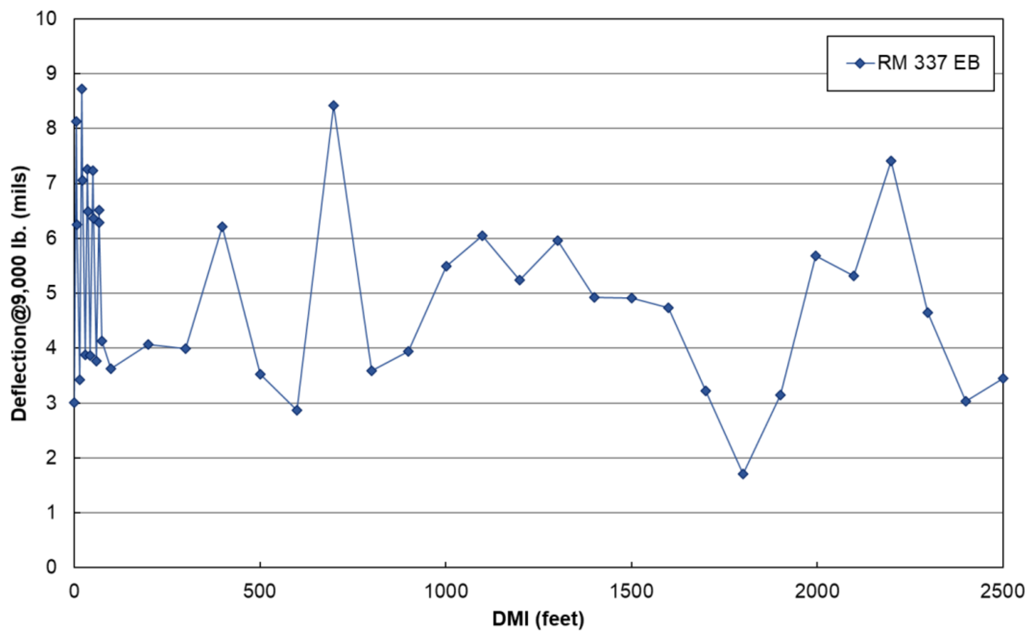

Figure 12 FWD deflections at RM 337 eastbound

Modulus of subgrade reaction (k-values) was estimated with the deflection data at mid-slabs using AREA method. Figure 13 shows static k-values obtained. It shows a wide variability in k-values in the areas evaluated, which may be a result of the absence of cement- or ~~asphalt-stabilized base~~. The average k-values are 242, 240 and 248 psi/in for US 83 Northbound, US 83 Southbound and RM 337 Eastbound, respectively. A few points with values greater than 600 psi/in were excluded in developing Figure 13. ~~Noise evaluations indicate that~~

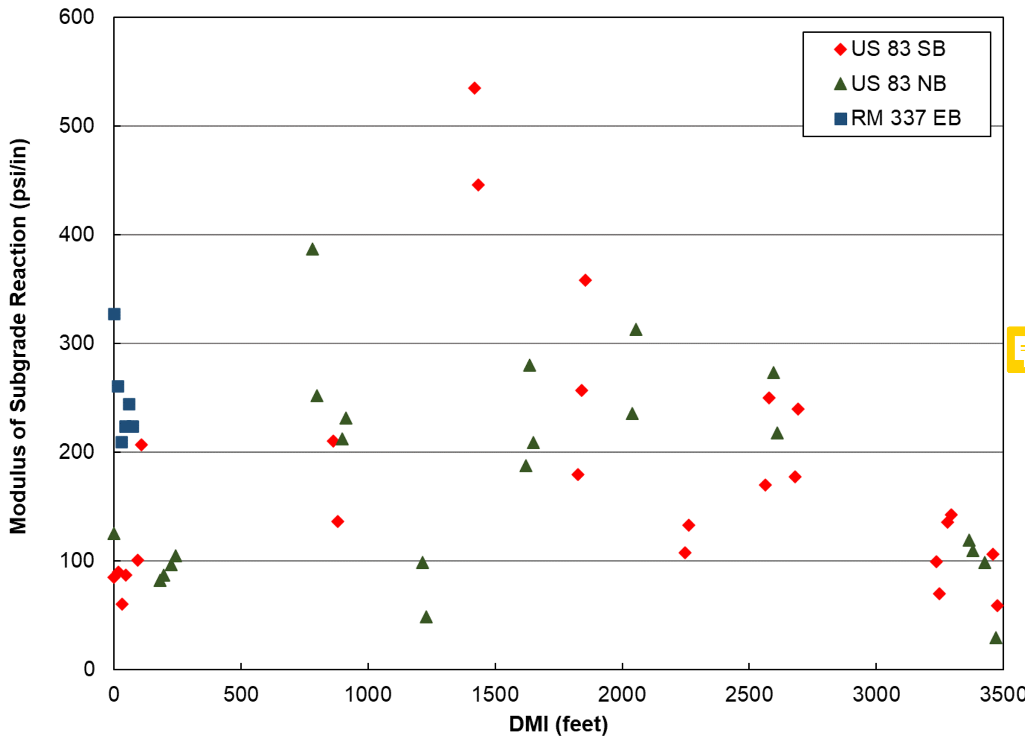

Figure 13 Modulus of subgrade reaction in the RCC pavement

Load transfer efficiency at the joints and cracks

FWD testing was conducted on both north- and south-bound lanes on US 83 for 3500-ft from the south end of the project. For RM 337, FWD testing was conducted eastbound direction only from the west end of the project to US 83, a total of approximately 2500-ft. To evaluate load transfer efficiency (LTE) at transverse contraction joints and cracks, a total of 17, 19 and 5 locations were selected on US 83 northbound, southbound and RM 337, respectively, and FWD testing conducted. Tables 1 through 3 show the locations of the joints and cracks selected for LTE evaluations. Here, the locations with Joint x were are those where FWD testing was conducted previously by MNT, while the locations with Sx are where FWD testing was conducted in this investigation. For LTE evaluation at each joint, FWD drops were made at 4 locations – mid-slab, upstream and downstream of the joint, and mid-slab. Figure 14 shows FWD testing at downstream of the joint.

Table 1 Joint and Crack Locations at US 83 Northbound Section

Location ID		DMI [ft]	GPS Coordinates	
			DMS	Decimal
Joint 1	Upstream	8	29° 42' 53.23500", -99° 45' 47.71764"	29.7147875, -99.7632549
	Downstream	10	29° 42' 53.24580", -99° 45' 47.71692"	29.7147905, -99.7632547
Joint 2	Upstream	189	29° 42' 55.01700", -99° 45' 47.74932"	29.7152825, -99.7632637
	Downstream	190	29° 42' 55.02960", -99° 45' 47.74860"	29.7152860, -99.7632635
Joint 3	Upstream	234	29° 42' 55.46304", -99° 45' 47.74788"	29.7154064, -99.7632633
	Downstream	235	29° 42' 55.47780", -99° 45' 47.74716"	29.7154105, -99.7632631
Joint 4	Upstream	655	29° 42' 59.62284", -99° 45' 47.81448"	29.7165619, -99.7632818
	Downstream	656	29° 42' 59.63400", -99° 45' 47.81160"	29.7165650, -99.7632810
Joint 5	Upstream	790	29° 43' 00.95088", -99° 45' 47.82960"	29.7169308, -99.7632860
	Downstream	791	29° 43' 00.96240", -99° 45' 47.83140"	29.7169340, -99.7632865
Joint 6	Upstream	906	29° 43' 02.09640", -99° 45' 47.85264"	29.7172490, -99.7632924
	Downstream	907	29° 43' 02.11080", -99° 45' 47.85264"	29.7172530, -99.7632924
Joint 7	Upstream	1221	29° 43' 05.21904", -99° 45' 47.86740"	29.7181164, -99.7632965
	Downstream	1223	29° 43' 05.23200", -99° 45' 47.86632"	29.7181200, -99.7632962
S1	Upstream	1627	29° 43' 09.23196", -99° 45' 47.92896"	29.7192311, -99.7633136
	Downstream	1628	29° 43' 09.24204", -99° 45' 47.92968"	29.7192339, -99.7633138
S2	Upstream	1642	29° 43' 09.37848", -99° 45' 47.93256"	29.7192718, -99.7633146
	Downstream	1643	29° 43' 09.39216", -99° 45' 47.93184"	29.7192756, -99.7633144
Joint 8	Upstream	2048	29° 43' 13.38888", -99° 45' 47.97432"	29.7203858, -99.7633262
	Downstream	2049	29° 43' 13.40112", -99° 45' 47.97576"	29.7203892, -99.7633266
Joint 9	Upstream	2604	29° 43' 18.87852", -99° 45' 48.02976"	29.7219107, -99.7633416
	Downstream	2605	29° 43' 18.89436", -99° 45' 48.03156"	29.7219151, -99.7633421

Joint 10	Upstream	3372	29° 43' 26.44536", -99° 45' 47.51568"	29.7240126, -99.7631988
	Downstream	3373	29° 43' 26.45796", -99° 45' 47.51496"	29.7240161, -99.7631986
S3	Upstream	3432	29° 43' 27.03108", -99° 45' 47.40516"	29.7241753, -99.7631681
	Downstream	3434	29° 43' 27.04296", -99° 45' 47.40300"	29.7241786, -99.7631675
S4	Upstream	3447	29° 43' 27.17904", -99° 45' 47.37996"	29.7242164, -99.7631611
	Downstream	3448	29° 43' 27.19056", -99° 45' 47.38536"	29.7242196, -99.7631626
S5	Upstream	3462	29° 43' 27.32304", -99° 45' 47.35764"	29.7242564, -99.7631549
	Downstream	3463	29° 43' 27.33456", -99° 45' 47.35404"	29.7242596, -99.7631539
Crack 1	Upstream	3484	29° 43' 27.54300", -99° 45' 47.31804"	29.7243175, -99.7631439
	Downstream	3486	29° 43' 27.55344", -99° 45' 47.31552"	29.7243204, -99.7631432
Crack 2	Upstream	3502	29° 43' 27.71292", -99° 45' 47.29284"	29.7243647, -99.7631369
	Downstream	3503	29° 43' 27.72516", -99° 45' 47.29104"	29.7243681, -99.7631364

Table 2 Joint and Crack Locations at US 83 Southbound Section

Location ID		DMI [ft]	GPS Coordinates	
			DMS	Decimal
S5	Upstream	9	29° 43' 27.38244", -99° 45' 47.48364"	29.7242729, -99.7631899
	Downstream	10	29° 43' 27.36624", -99° 45' 47.48184"	29.7242684, -99.7631894
S4	Upstream	24	29° 43' 27.23448", -99° 45' 47.50596"	29.7242318, -99.7631961
	Downstream	25	29° 43' 27.22188", -99° 45' 47.50992"	29.7242283, -99.7631972
S3	Upstream	39	29° 43' 27.08760", -99° 45' 47.53548"	29.7241910, -99.7632043
	Downstream	40	29° 43' 27.07464", -99° 45' 47.53836"	29.7241874, -99.7632051
Joint 10	Upstream	99	29° 43' 26.50728", -99° 45' 47.67444"	29.7240298, -99.7632429
	Downstream	100	29° 43' 26.49108", -99° 45' 47.67660"	29.7240253, -99.7632435
Culvert 1	Upstream	447	29° 43' 23.10024", -99° 45' 48.16872"	29.7230834, -99.7633802
	Downstream	449	29° 43' 23.08728", -99° 45' 48.16548"	29.7230798, -99.7633793

Culvert 2	Upstream	463	29° 43' 22.94004", -99° 45' 48.16656"	29.7230389, -99.7633796
	Downstream	464	29° 43' 22.92744", -99° 45' 48.16764"	29.7230354, -99.7633799
Culvert 3	Upstream	478	29° 43' 22.79280", -99° 45' 48.17484"	29.7229980, -99.7633819
	Downstream	479	29° 43' 22.78092", -99° 45' 48.17880"	29.7229947, -99.7633830
Joint 9	Upstream	871	29° 43' 18.91488", -99° 45' 48.19140"	29.7219208, -99.7633865
	Downstream	873	29° 43' 18.90048", -99° 45' 48.19212"	29.7219168, -99.7633867
Joint 8	Upstream	1427	29° 43' 13.42308", -99° 45' 48.14820"	29.7203953, -99.7633745
	Downstream	1428	29° 43' 13.41192", -99° 45' 48.14676"	29.7203922, -99.7633741
S2	Upstream	1833	29° 43' 09.41448", -99° 45' 48.07764"	29.7192818, -99.7633549
	Downstream	1834	29° 43' 09.40296", -99° 45' 48.07800"	29.7192786, -99.7633550
S1	Upstream	1848	29° 43' 09.26652", -99° 45' 48.07404"	29.7192407, -99.7633539
	Downstream	1849	29° 43' 09.25428", -99° 45' 48.07224"	29.7192373, -99.7633534
Joint 7	Upstream	2254	29° 43' 05.25036", -99° 45' 48.02796"	29.7181251, -99.7633411
	Downstream	2255	29° 43' 05.24028", -99° 45' 48.02688"	29.7181223, -99.7633408
Joint 6	Upstream	2570	29° 43' 02.13816", -99° 45' 47.98512"	29.7172606, -99.7633292
	Downstream	2571	29° 43' 02.12700", -99° 45' 47.98764"	29.7172575, -99.7633299
Joint 5	Upstream	2686	29° 43' 00.99624", -99° 45' 47.97252"	29.7169434, -99.7633257
	Downstream	2687	29° 43' 00.98328", -99° 45' 47.96892"	29.7169398, -99.7633247
Joint 4	Upstream	2821	29° 42' 59.65236", -99° 45' 47.95848"	29.7165701, -99.7633218
	Downstream	2822	29° 42' 59.63940", -99° 45' 47.95560"	29.7165665, -99.7633210
Joint 3	Upstream	3242	29° 42' 55.49256", -99° 45' 47.91060"	29.7154146, -99.7633085
	Downstream	3243	29° 42' 55.48032", -99° 45' 47.91024"	29.7154112, -99.7633084
Joint 2	Upstream	3286	29° 42' 55.04292", -99° 45' 47.89944"	29.7152897, -99.7633054
	Downstream	3288	29° 42' 55.03032", -99° 45' 47.90016"	29.7152862, -99.7633056
Crack 1	Upstream	3444	29° 42' 53.50068", -99° 45' 47.86956"	29.7148613, -99.7632971

	Downstream	3445	29° 42' 53.48736", -99° 45' 47.86272"	29.7148576, -99.7632952
Joint 1	Upstream	3467	29° 42' 53.26956", -99° 45' 47.85696"	29.7147971, -99.7632936
	Downstream	3468	29° 42' 53.25408", -99° 45' 47.85444"	29.7147928, -99.7632929

Table 3 Joint Locations at RM 337 Eastbound Section

Location ID		DMI [ft]	GPS Coordinates	
			DMS	Decimal
Joint 1	Upstream	5	29° 43' 28.82748", -99° 46' 16.87296"	29.7246743, -99.7713536
	Downstream	7	29° 43' 28.82676", -99° 46' 16.85712"	29.7246741, -99.7713492
Joint 2	Upstream	20	29° 43' 28.82388", -99° 46' 16.70124"	29.7246733, -99.7713059
	Downstream	22	29° 43' 28.82568", -99° 46' 16.68756"	29.7246738, -99.7713021
Joint 3	Upstream	36	29° 43' 28.82352", -99° 46' 16.53240"	29.7246732, -99.7712590
	Downstream	37	29° 43' 28.83036", -99° 46' 16.52088"	29.7246751, -99.7712558
Joint 4	Upstream	50	29° 43' 28.83216", -99° 46' 16.36752"	29.7246756, -99.7712132
	Downstream	52	29° 43' 28.83144", -99° 46' 16.35240"	29.7246754, -99.7712090
Joint 5	Upstream	66	29° 43' 28.82964", -99° 46' 16.19760"	29.7246749, -99.7711660
	Downstream	67	29° 43' 28.83036", -99° 46' 16.18284"	29.7246751, -99.7711619

Figure 14 FWD testing at downstream of the joint of RCC pavement

Load transfer efficiency (LTE) at joints or cracks provides an indication of the effectiveness of the joints or cracks to transfer loading from one slab to the adjacent slab across the discontinuity. The most widely used LTE equation is expressed as:

$$LTE_{\delta} = \frac{\delta_U}{\delta_L} \times 100\%$$

Here, δ_U and δ_L are deflections at the unloaded and loaded side, respectively. The configuration of sensors during the LTE testing for upstream and downstream are shown in Figure 15. At a joint or a crack, 2 LTE values are evaluated – one for upstream and the other for downstream. For upstream LTE, W_1 becomes δ_L and W_2 is δ_U, while for downstream LTE, W_1 becomes δ_L and W_8 is δ_U,

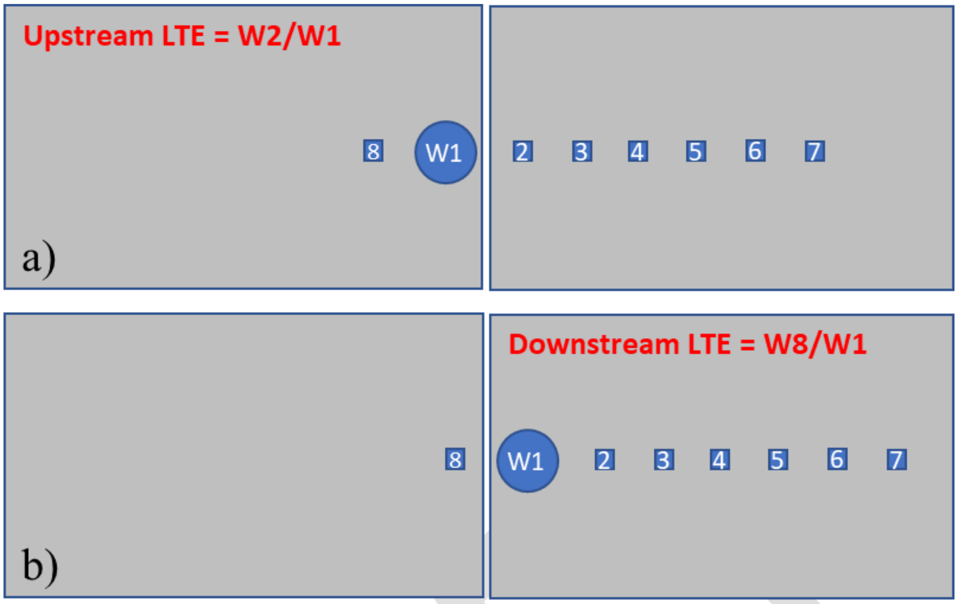

Figure 15 FWD sensor configurations for (a) upstream and (b) downstream LTE determination

Figures 16, 17 and 18 present LTE values evaluated at US 83 northbound, southbound and RM 337 eastbound, respectively. It is observed that LTE values are quite low and these values fall within the LTE ranges reported by Pittman (1996). Low LTE values imply that almost all the joints popped and aggregate interlock at joints and cracks that exists in this RCC pavement is not sufficient enough to provide adequate load transfer for highway main lane pavement. Another observation is that there is a large difference between upstream and downstream LTEs at certain joints, especially on US 83 southbound. This indicates the cracks under the sawcut are deviating from a straight line, as well as low level of aggregate interlock. Compared with US 83, LTE values on RM 337 are quite low. It is not known for this low LTE values, since joint spacing is the same for both projects and the only difference in pavement structure is slab thickness – 8.0-in for US 93 vs 7.5-in for RM 337. This small difference in slab thickness should not result in this large discrepancy in LTE levels. There might be other causes.

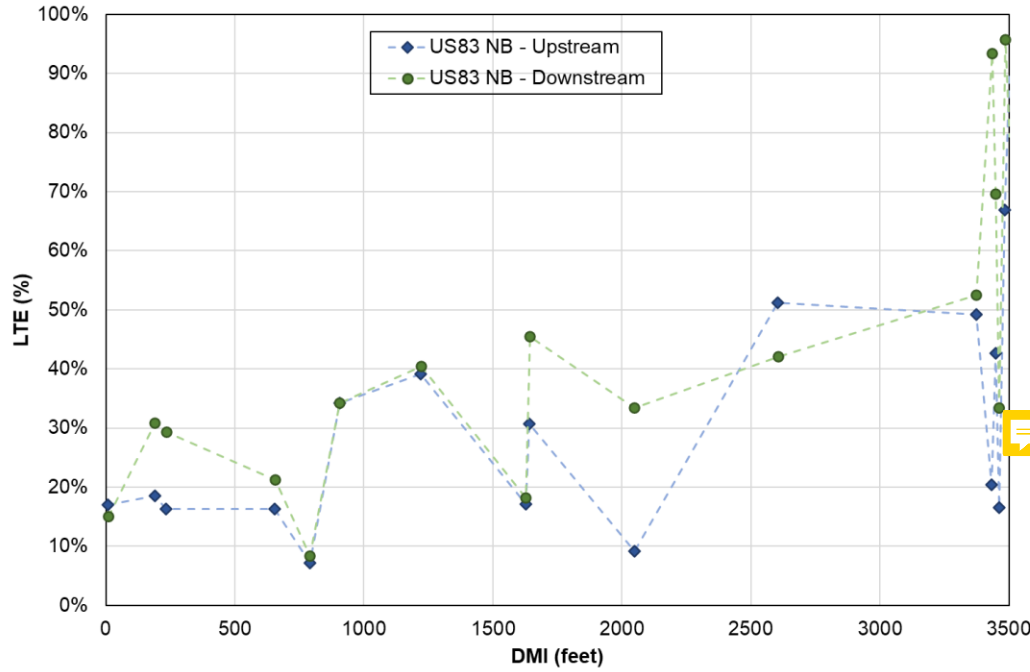

Figure 16 Joint LTE at US 83 northbound

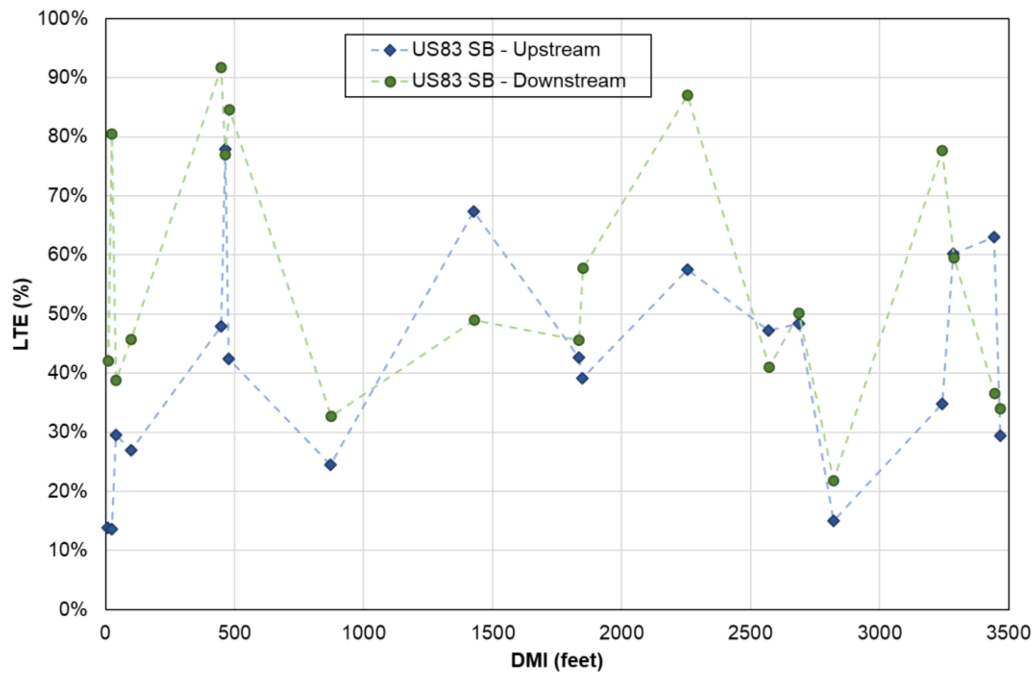

Figure 17 Joint LTE at US 83 southbound

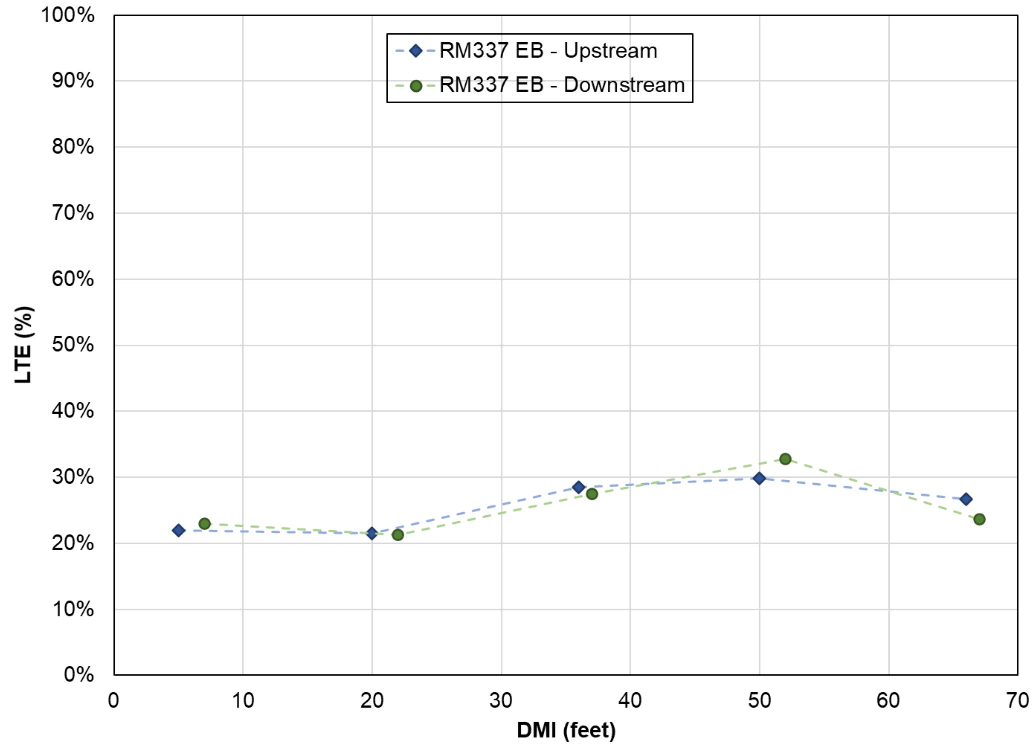

Figure 18 Joint LTE at RM 337 eastbound

Figure 19 illustrates the variations in LTE over the last 4 years at US 83 northbound. It shows general decrease in LTE over time, even though there are other factors than age or traffic that have effect on LTE, such as testing season, time of the day of testing, so forth. Since detailed information on the day and time of the FWD testing conducted in 2017 and 2019, an accurate assessment is not feasible on the LTE trend over time; however, it appears that LTE values were quite low even after one year of construction, and they stayed low. Figure 20 illustrates the variations in LTE over the last 4 years at US 83 southbound. Since the locations of the joints selected for LTE evaluations in 2017 and 2019 are not the same as those selected for this investigation, direct comparisons in changes of LTE over time are not feasible. Rather, it shows large variations in LTE values along the project, and not much changes are observed in average LTE values over time.

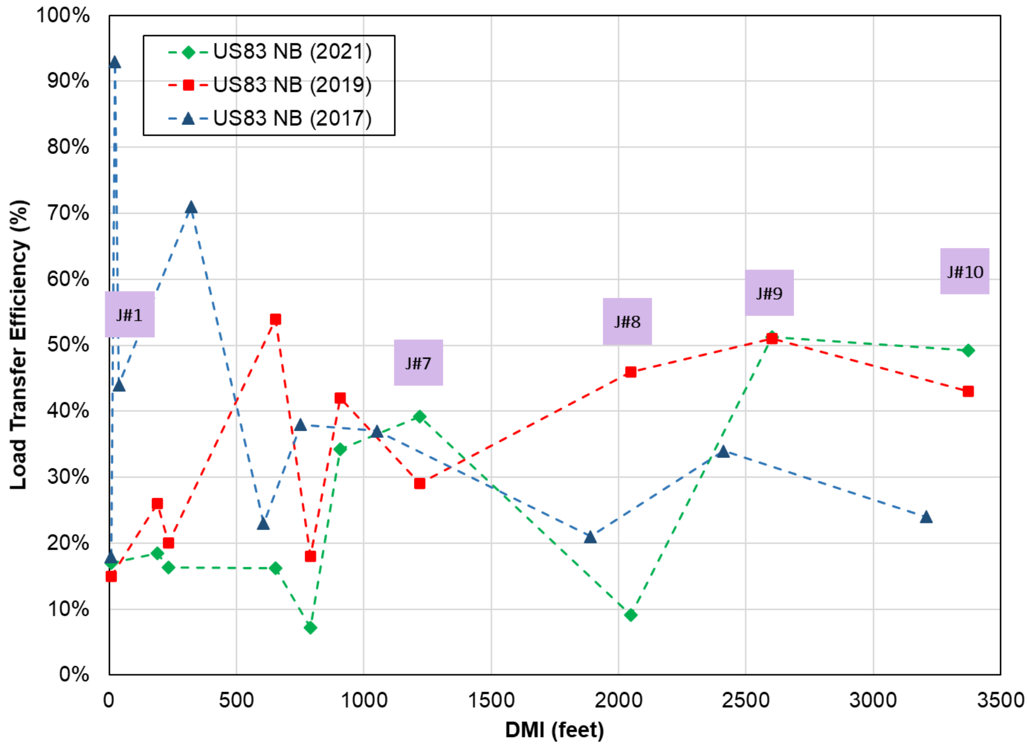

Figure 19 Joint LTE comparison at US 83 northbound in 2017, 2019 and 2021

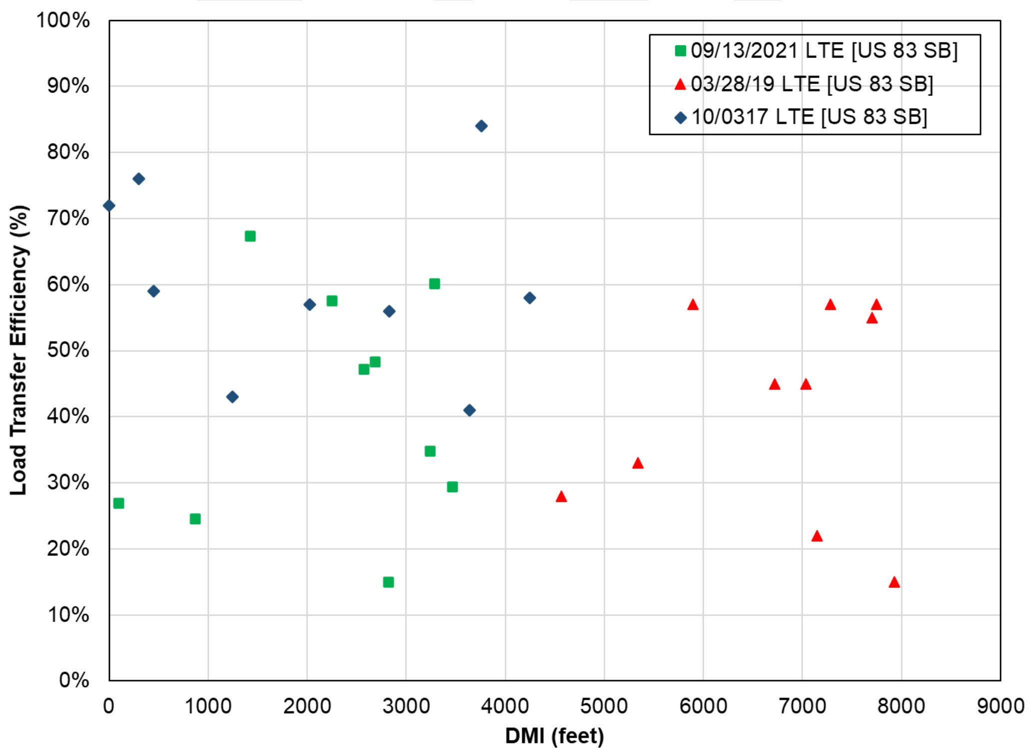

Figure 20 Joint LTE comparison at US 83 southbound in 2017, 2019 and 2021

Summary

The condition of the roller-compacted concrete pavement in US 83 and RM 337 in Leakey, in the San Angelo District, was evaluated. This project is the first and only RCC pavement in main lanes in highways managed by TxDOT. The scope of the evaluation was limited to visual condition survey and structural condition evaluations by deflection testing. Visual condition survey indicated cracks in few places; however, faulting was the major distress that was causing noise and detrimental effects on ride quality. The faulting was not measured; however, it was large enough to cause noise and deteriorated ride, which was not acceptable for main lanes of highways.

Structural evaluations with FWD indicated two interesting responses: (1) deflections at mid-slab were relatively small while (2) deflections at joints were large and LTE values at joints and cracks were quite low, which clearly indicates the lack of or low level of aggregate interlock at transverse contraction joints and cracks. One of the advantages of RCCP that has been stated is a good aggregate interlock at joints thanks to smaller crack widths from low drying shrinkage of the PCC mixtures. Even though this pavement is only 5 years old, and traffic on both highways is quite low, LTE values are unacceptably low for highway pavements.

Low compaction and resulting large voids of RCC at the lower part of the slab would reduce concrete strength substantially, causing lower fatigue life. In the RCCP projects evaluated in this investigation, fatigue failures of the pavement due to lower strength might not occur soon, primarily due to quite a low traffic level; otherwise, fatigue failures might have already developed or develop soon.

~~Based on the limited evaluations~~ of RCC pavements conducted in this investigation, the following recommendations are made:

1) Until new design/construction methods are incorporated that will ensure good load transfer at transverse joints, the use of RCCP in main lane highways is not recommended.
2) Construction and/or materials specifications need to be developed that will facilitate the compaction of RCC materials in RCCP.

References

ACI Guide (2015) American Concrete Institute. Guide to Roller Compacted Concrete Pavements. ACI 327R-14. Farmington Hills, MI, 2015.

Delatte, N. and C. Storey. (2005). Effects of density and mixture proportions on freeze-thaw durability of roller-compacted concrete pavements. Transportation Research Record: Journal of the Transportation Research Board 1914: 45–52.

Federal Highway Administration. (2016). TechBrief (June 2016 – Roller-Compacted Concrete Pavement, Vol. FHWA-HIF-16-003). Washington, DC: U.S. Department of Transportation Federal Highway Administration

Harrington, D., Abdo, F., Adaska, W., Hazaree, C.V., Ceylan, H., Bektas, F. (2010) Guide for roller-compacted concrete pavements. National Concrete Pavement Technology Center, Institute of Technology Iowa University, Portland Cement Association.

LaHucik, J., Dahal, S., Roesler, J., and Amirkhanian, A.N. (2017) Mechanical properties of roller-compacted concrete with macro-fibers Constr. Build. Mater., 135, pp. 440-446, DOI: 10.1016/j.conbuildmat.2016.12.212

Nanni, A., and A. Johari. (1989). RCC Pavement Reinforced with Steel Fibers. Concrete International, Vol. 11, No. 3, March 1989, pp. 64–69.

Nanni, A., D. A. Ludwig, and J. E. Shoenberger. (1996). Physico-Mechanical Properties and Load Transfer Efficiency of RCC Pavement. ACI Materials Journal, Vol. 93, 1996, pp. 356–361.

Piggott, R.W., (1999). Roller compacted concrete pavements: A study of long-term performance. PCA R&D Serial No. 2261. Skokie, III: Portland Cement Association.

Pittman, D. W. (1989) The Effects of the Construction Process on Selected Fresh and Hardened Properties of Roller-Compacted (RCC) Pavements. Miscellaneous Paper GL-89-22. U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Sept. 1989.

Pittman, D. W. (1996) Factors Affecting Joint Efficiency of Roller-Compacted Concrete Pavement Joints and Cracks. In Transportation Research Record 1525, TRB, National Research Council, Washington, D.C., 1996, pp. 10–20.

Pittman, D. W., and McCullough, B. F. (1997). Development of a Roller-Compacted Concrete Pavement Crack and Joint Spacing Model. In Transportation Research Record 1568, TRB, National Research Council, Washington, D.C., 1997, pp. 52–64.

Pittman, D.W., and Anderton, G. (2012) Characteristics of Roller Compacted Concrete Pavements in the United States. Presentation at the 7th International Conference on Maintenance and Rehabilitation of Pavements and Technological Control (MAIRE PAV 7), Auckland, New Zealand.

Shihata, S. A. (2000) Strength and Density of Laboratory-Prepared RCC Specimens: Effect of Compaction Procedure. Cement, Concrete, and Aggregates, Vol. 22, No. 1, 2000, pp. 1–9. http://dx.doi.org/10.1520/CCA10456J.

Sok, T., Hong, S.J., Kim, Y.K. and Lee, S.W., (2018). Evaluation of load transfer characteristics in roller-compacted concrete pavement, International Journal of Pavement Engineering, DOI: 10.1080/10298436.2018.1511782