State of Florida
Department of Transportation

Performance Evaluation of Limerock, Cemented Coquina, and Shell Rock as a Base Material

FDOT Office
State Materials Office

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Experimental Project Location
County Section 94030-3506
State Road 70
St. Lucie County

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EXECUTIVE SUMMARY

An experimental project located in the westbound travel lane of SR 70 in St. Lucie County just a few miles west of the Turnpike and I-75 in St. Lucie County was constructed in 1996 to evaluate the performance of a limerock, cemented coquina, and shell rock as base materials. Prior to resurfacing in 2008, an average of 1.9 million ESALs were applied to the experimental roadway. Plate bearing tests and laboratory measurements during construction showed that the limerock was the stiffest base material and the shell rock the least stiff. Falling Weight Deflectomer data collected during the life of the project showed that the limerock base was slightly stiffer than the cemented coquina and shell rock base materials. The section with the shell rock base rutted the greatest but the overall rut depths of all three sections were still satisfactory prior to resurfacing. Cracking was the most predominant distress of all three sections. The first cracks observed were in the shell rock section five years after construction and included primarily longitudinal cracks located in the wheel paths. Cracks were first observed in the other sections two years later.

INTRODUCTION

Background

In 1996, an experimental project was constructed on State Road (SR) 70 in St. Lucie County to investigate the relative performance of limerock, cemented coquina, and shell rock as a base material. The Florida Department of Transportation's (FDOT) current flexible pavement design method is based on the 1993 AASHTO method and is based on the structural number (SN) and layer coefficient concept (*1*). The Structural Number (SN) is used to indicate the required combined structural capacity of the pavement layers above the embankment. The required SN is a function of reliability, serviceability, embankment resilient modulus, and the expected traffic. The SN of the designed pavement must be greater than the required SN to ensure adequate pavement performance. The required SN is calculated as follows:

$$SN = (a_1 \text{ x } D_1) + (a_2 \text{ x } D_2) + (a_3 \text{ x } D_3) + \ldots + (a_n \text{ x } D_n)$$

Where:

$a_{1\text{-}n}$ = layer coefficient of the 1st to nth layer
$D_{1\text{-}n}$ = layer thickness of 1st to nth layer

Over the years, FDOT has constructed several experimental field projects to study thickness and type of pavement layers (*2*). Layer coefficients shown in TABLE 1 have been developed which represent the relative strength of different pavement materials in Florida. Structural coefficients must be developed or revised as new materials or research is introduced.

TABLE 1 Structural Coefficients for Various Florida Pavement Layers

Group	Layer	Layer Coefficient
Friction Courses	FC-5	0.00
	FC-12.5, FC-9.5	0.44
Structural Courses	SP-9.5, SP-12.5, SP-19.0	0.44
Base Courses (General Use)	Limerock (LBR 100)	0.18
	Cemented Coquina (LBR 100)	0.18
	Shell Rock (LBR 100)	0.18
	Bank Run Shell (LBR 100)	0.18
	Graded Aggregate (LBR 100)	0.15
	Type B-12.5 (Asphalt Base)	0.30
Base Courses (Limited Use)	Limerock Stabilization (LBR 70)	0.12
	Shell Stabilization (LBR 70)	0.10
	Sand Clay (LBR 75)	0.12
	Soil Cement (500 psi)	0.20
	Soil Cement (300 psi)	0.15
Stabilization	Type B Stabilization (LBR 40)	0.08
	Type B Stabilization (LBR 30)	0.06
	Type C Stabilization	0.06
Subgrade	Cement Treated	0.12
	Lime Treated	0.08

Research Objectives

The objective of this study is to compare the relative long-term performance of limerock, cemented coquina, and shell rock as base materials. Performance was evaluated in terms of deflection, ride quality/roughness, rutting, and cracking. This report covers the period from the initial reconstruction in 1996 until resurfacing 2008.

PROJECT DESCRIPTION

The experimental project is in the westbound travel lane of SR 70 just a few miles west of the Turnpike and I-75 in St. Lucie County. The roadway in this area consists of four 12-foot lanes divided by a grassy median. The three 1,500-foot-long test sections were constructed as a segment of County Section 94030-3506. The experimental sections consisted of 4 inches of hot-mix asphalt (HMA), 10 inches of base, and 12 inches of subgrade. The locations of each section are described in TABLE 2. The experimental sections were initially constructed in 1996 and resurfaced in 2008. Traffic monitoring sites are located at MP 16.310 and MP 20.200. The site located at MP 20.200 is less than 0.5 miles from the Turnpike access ramp and has recorded increased traffic levels. FIGURE 1 summarizes the ESALs for the project. Prior to resurfacing in 2008, 1.5 million ESALs were recorded at MP 16.310 and 2.3 million ESALs were recorded at MP 20.200. Rainfall data from 1999 to 2008 is presented in FIGURE 2.

TABLE 2 SR 70 Experimental Section Layout

	Section 1	Section 2	Section 3
Base Type	Limerock	Cemented Coquina	Shell Rock
Pit Number	87-145	94-209	93-406
Station, feet	215+00 to 230+00	270+00 to 285+00	290+00 to 305+00
Milepost	17.466 to 17.750	18.509 to 18.790	18.866 to 19.171

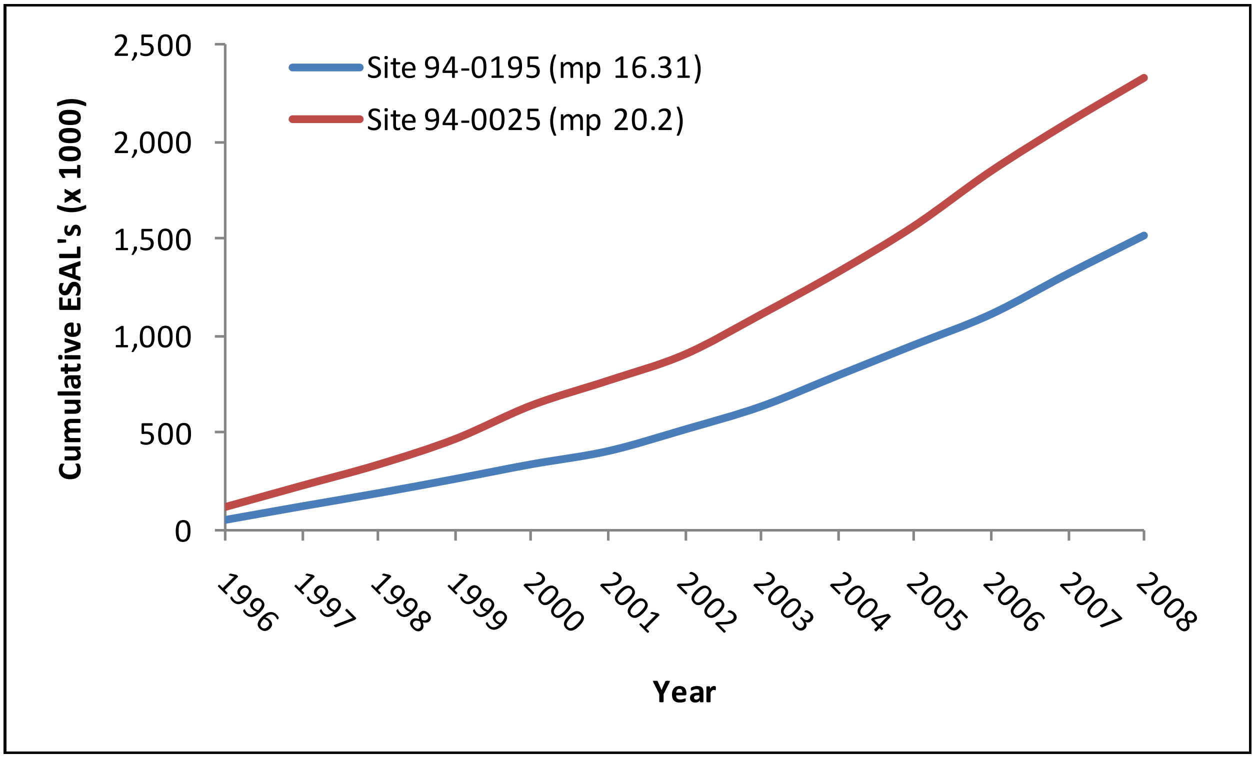


FIGURE 1 SR Traffic summary.

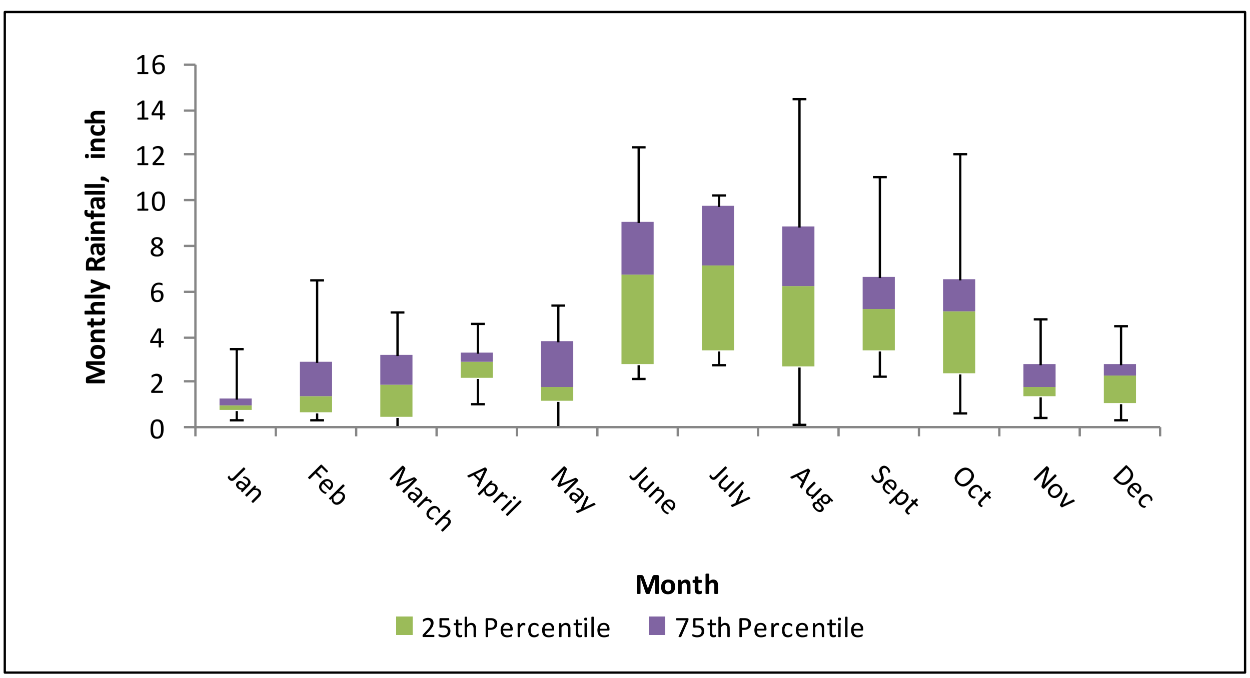


FIGURE 2 Rainfall history from 1999 to 2008.

SR 70 CONSTRUCTION MATERIAL INFORMATION

Roadway construction complied with FDOT's Standard Specifications for Road and Bridge Construction, 1991 Edition. The embankment material was classified as an A-3 according to the AASHTO classification system. The subgrade material was stabilized using coquina to achieve a limerock bearing ratio (LBR) of 40 or greater. In addition to standard quality control measurements, the following tests were conducted at selected locations:

1. Non-repetitive static plate load tests of soils (FM 5-527/AASHTO T-222)
2. Moisture-density relations for embankment soils (FM 5-525/AASHTO T99)
3. Moisture-density relations for subgrade and base materials (FM5-521/AASHTO T180)
4. Particle size analysis (FM 1-T 088/AASHTO T88)
5. Limerock Bearing Ratio (FM 5-515)
6. Percent carbonates (FM 5-514)

The gradation and percent carbonates for the three base materials are summarized in TABLE 3. Plate bearing tests and laboratory test results conducted as each pavement layer was constructed are summarized in TABLE 4. A trench was cut after the first lift of asphalt was placed to perform additional plate bearing tests. The intent of the additional trench tests was to determine if the stiffness or moisture content of the pavement layers had changed since construction. In general, the difference in stiffness is within the typical testing variability for the base and embankment. An increase in stiffness was observed for the subgrade. These properties are summarized in TABLE 5. Overall, there is no significant difference in subgrade or embankment stiffness of each of the three sections. The limerock base was found to be the stiffest and the shell rock base was found to be the least stiff during both test series.

Ground penetrating radar (GPR) data was collected during 2008 to confirm the asphalt and base thickness. TABLE 6 summarizes the pavement layer thickness estimated by GPR and suggests that the base layer was slightly under constructed.

TABLE 3 Base Material Gradation and Percent Carbonates

Section	Type	Percent Passing Sieve			Percent Carbonates
		3.5 inch	#4	#200	
1	Limerock	100	60	16.1	75
2	Coquina	100	67	9.2	56
3	Shell Rock	100	68	10.9	40

TABLE 4 Field and Laboratory Measurements

Layer	Section/Base Type	Plate Bearing Modulus, psi	Field Density, pcf	Field Moisture, %	Lab Max Density, pcf	Optimum Moisture, %	% Max Density	LBR Value
Base	1 / Limerock	37,770	133.0	5.6	133.1	6.8	99.9	161
	2 / Coquina	26,710	127.8	6.2	132.6	6.6	96.2	169
	3 / Shell Rock	25,350	128.1	7.3	129.2	8.0	99.1	157
Subgrade[1]	1 / Limerock	9,330	114.2	8.9	116.3	10.1	98.2	39
	2 / Coquina	8,240	119.3	8.4	117.9	9.0	101.2	46
	3 / Shell Rock	5,180	118.2	6.2	119.0	9.3	99.3	50
Embankment	1 / Limerock	16,570	114.1	4.4	109.9	12.3	103.8	20
	2 / Coquina	16,860	115.0	6.9	108.9	12.2	105.5	35
	3 / Shell Rock	11,940	110.4	7.7	107.2	12.8	102.9	44

Note 1: Subgrade plate bearing values are outside of expected range.

TABLE 5 Trench plate bearing data.

Section	Type	Station	Base		Subgrade		Embankment	
			Plate Bearing Modulus, psi	Field Moisture (Oven), %	Plate Bearing Modulus, psi	Field Moisture (Oven), %	Plate Bearing Modulus, psi	Field Moisture (Oven), %
1	Limerock	244+15	44,570	5.5	15,270	6.8	16,030	8.5
2	Coquina	273+15	30,550	5.1	20,280	9.5	17,090	12.9
3	Shell Rock	300+15	20,300	6.4	18,660	8.0	20,660	7.0

TABLE 6 Pavement Layer Thickness Estimated by GPR

	Section 1	Section 2	Section 3
Base Type	Limerock	Cemented Coquina	Shell Rock
Avg. HMA Thickness, inch	3.9	4.1	4.0
Avg. Base Thickness, inch	9.3	8.4	9.4

PAVEMENT PERFORMANCE MONITORING

The SMO conducts annual pavement performance monitoring of experimental projects during the same time of the year to better control variability due to seasonal changes (e.g., temperature, rainfall, etc.). The experimental project on SR 70 is typically evaluated in October. The focus of this report is to document the performance of the experimental sections from 1996 until resurfacing in 2008. The primary parameters used to evaluate the performance of the experimental sections include:

1. Pavement deflection/stiffness
2. Pavement smoothness
3. Asphalt surface permanent deformation (rutting)
4. Pavement cracking

Falling Weight Deflectometer (FWD)

Annual deflection measurements are made in the outside wheel path of each lane. A photograph of a FWD is shown in FIGURE 3. A typical testing configuration includes the following:

- A 12-inch load plate
- A 9-kip load
- Deflection sensors placed at 0, 8, 12, 18, 24, 36, and 60 inches from the load plate

FIGURE 3 Falling Weight Deflectometer.

The stiffness of a pavement system may be inferred through the analysis of deflection basins or by estimation of layer moduli through backcalculation or forward calculation. Backcalculation is a process used to estimate pavement layer moduli by matching estimated deflection basins with measured basins through an iterative process. The backcalculated embankment modulus during the first year of the experimental project is shown in FIGURE 4. The embankment modulus for each section is similar, ranging between 18 and 20 ksi, and should not influence the results of the study. The initial deflection basins measured in 1996 and 2007 are shown in FIGURE 5. The basins show that the initial stiffness of the three sections is similar. In 2007, the section with the limerock base is slightly stiffer than the Coquina section and the section constructed with a Shell Rock base was slightly weaker than the other sections. A complete FWD survey was not possible in 2008 due to the resurfacing effort. FIGURE 6 shows the variability of the backcalculated base modulus measured since 1996. Overall, the limerock base was found to be slightly stiffer than the other two sections.

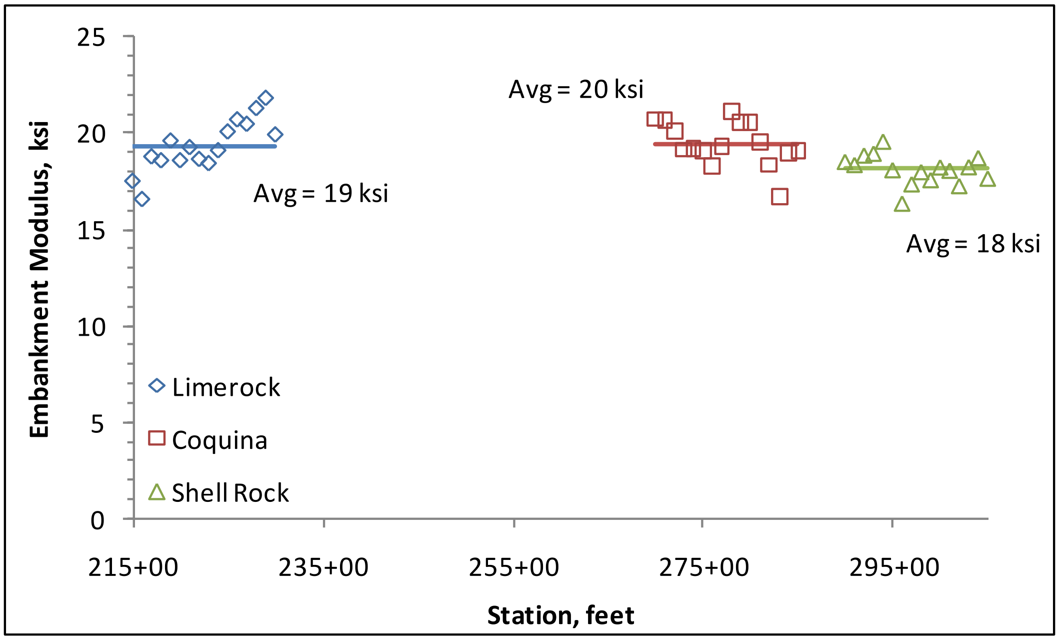


FIGURE 4 Embankment modulus measured during first survey year (1996).

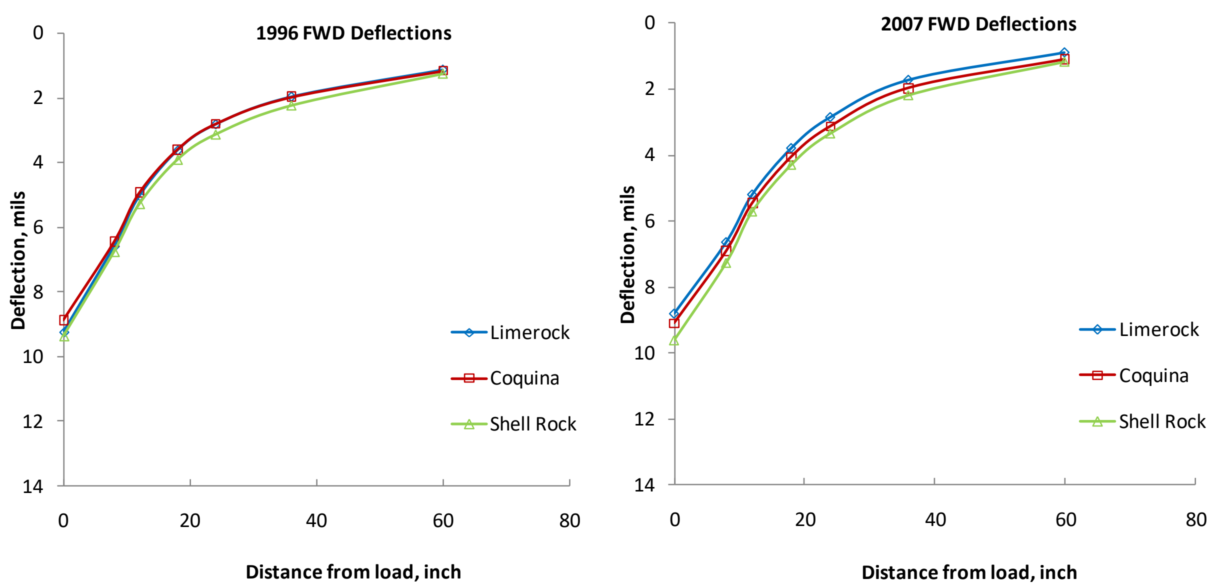


FIGURE 5 Deflection basins.

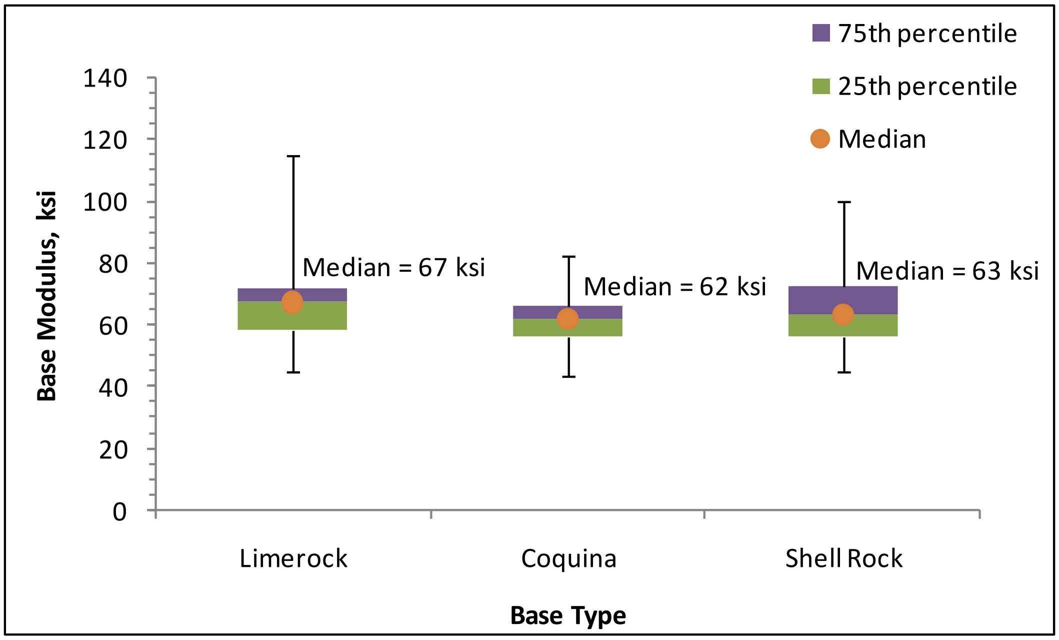


FIGURE 6 Variability of SR 70 base modulus since 1996.

Permanent Deformation

Permanent deformation, or rut depth, is measured manually. The rut depths measured during 2008 for the sections with the limerock, coquina, and shell rock bases were 0.19 inches, 0.20 inches, and 0.24 inches, respectively. While the shell rock base has a slightly greater rut depth on average, as indicated in FIGURE 7, the overall rut depths of the sections are acceptable.

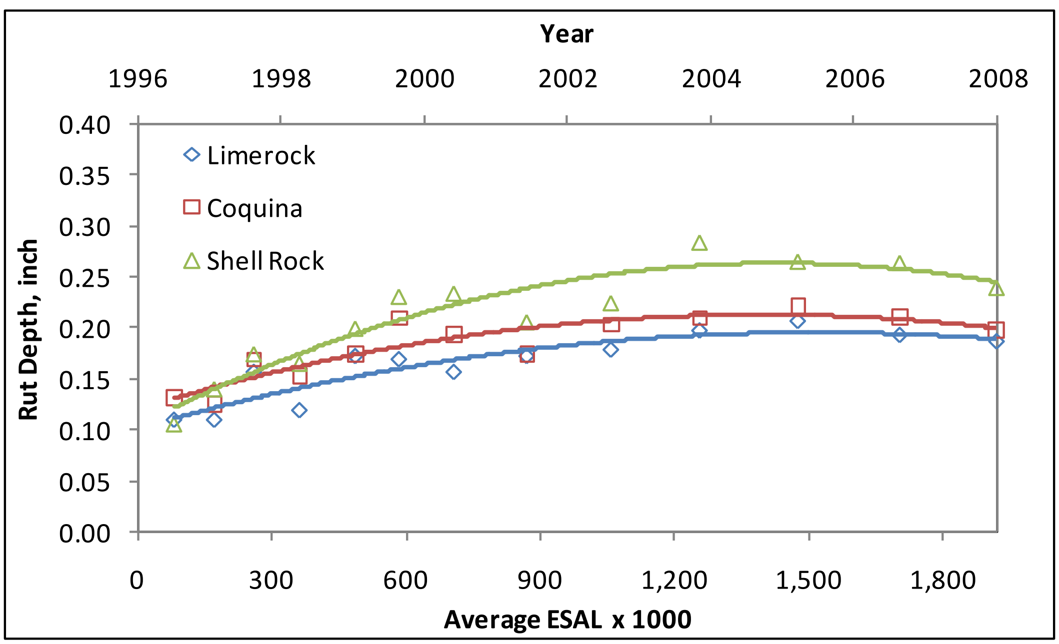
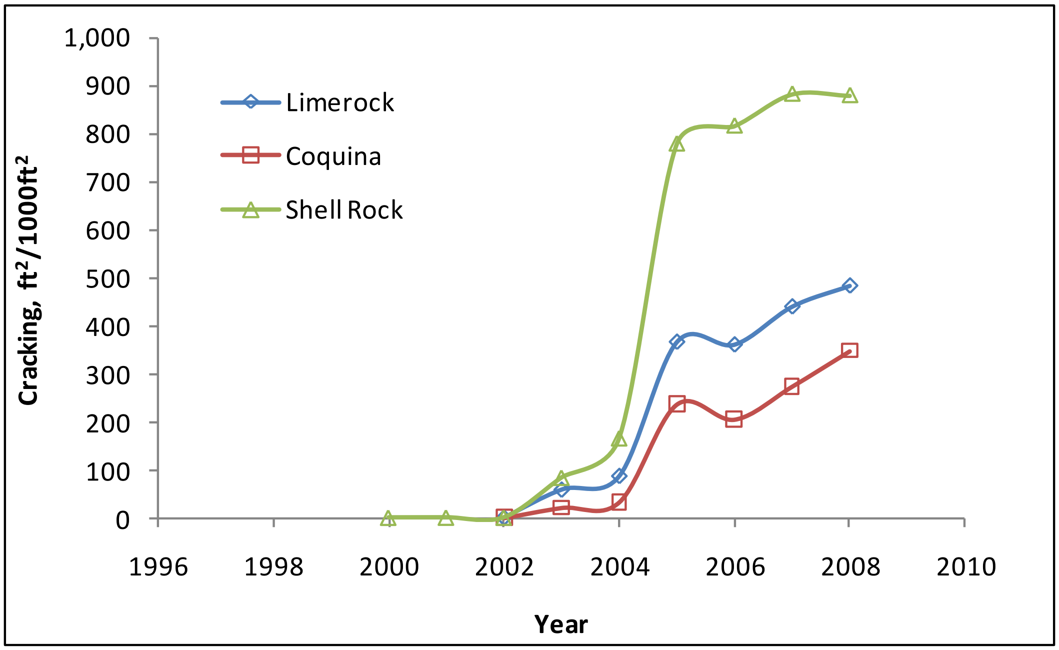


FIGURE 7 Rut depth measurements.

Cracking

The first cracks were observed in the Shell rock section five years after construction. Cracks were first observed in the other sections two years later. The primary crack type initially observed included longitudinal cracks located in the wheel paths. Later, transverse cracks were observed extending beyond the wheel paths. Block cracking was also evident between the wheel paths. Transverse cracks near the wheel path may be associated with fatigue due to traffic. Block cracking is usually caused by shrinkage of the asphalt due to daily temperature cycling. The presence of block cracking is an indication the asphalt has hardened significantly. Photographs of the crack patterns are shown in FIGURE 8. The shell rock section had nearly twice as much cracking as the other two sections in 2008. FIGURE 9 summarizes the crack measurements prior to resurfacing in 2008.

FIGURE 8 Pavement cracking trends: longitudinal crack in the wheel path initially.



Section	Years Until First Crack	2008 Crack Extent, $ft^2/1000ft^2$
Limerock	7	484
Coquina	7	349
Shell Rock	5	878

FIGURE 9 Summary of cracking on the SR 70 experimental project.

Roughness

The Ride Number (RN) (ASTM E 1489) is a profile index that rates the ride quality of a road using a scale that corresponds to a user's perception of pavement roughness. A RN of 5.0 represents a perfect ride quality while an RN of 0 corresponds to a virtually impassable surface. Prior to rehabilitation in 2008, the RN of each section remained above 4.0, for most of the pavement life representing a good ride quality. During the year prior to resurfacing, the RN for the Coquina and Shell Rock dropped below 4.0. FIGURE 10 shows the gradual decline in RN for each survey year.

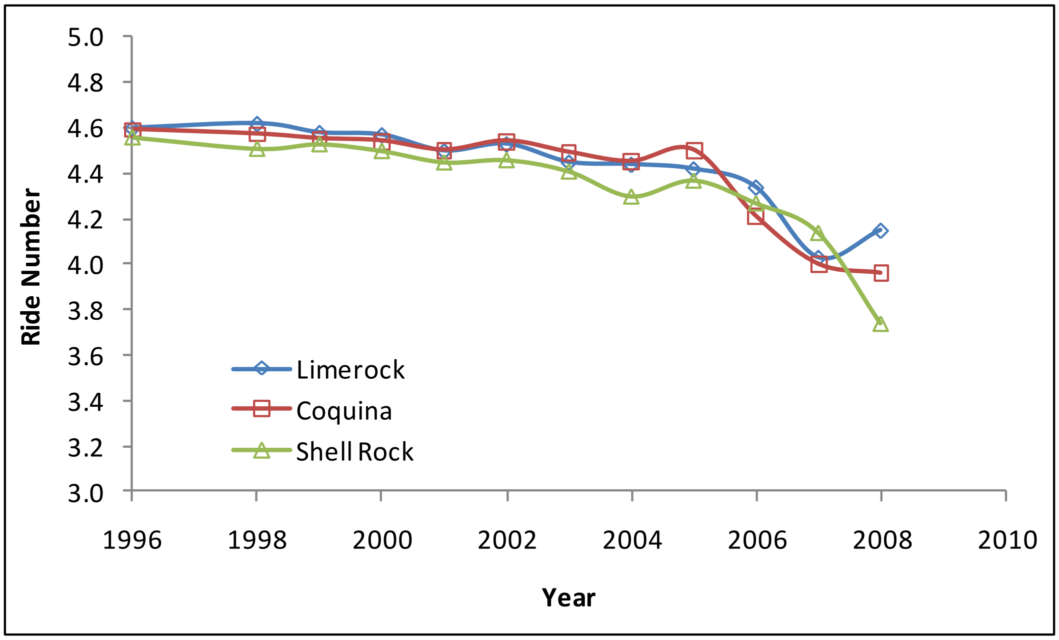


FIGURE 10 SR 70 Ride Numbers.

SUMMARY AND CONCLUSIONS

Plate bearing tests and laboratory measurements made during construction showed that the limerock base was the stiffest material and the shell rock base the least stiff. Plate bearing tests performed just after construction through trenches showed that the limerock base was more than 2.2 times stiffer than the shell rock base and 1.5 times stiffer than the coquina. However, FWD data collected during the life of the project showed that the limerock base was just slightly stiffer than either other base type (median limerock base modulus of 67 ksi compared to 62 to 63 ksi for coquina and shell rock, respectively).

The section with the shell rock base rutted the greatest but the overall rut depths of all three sections were still satisfactory prior to resurfacing. Cracking was the most predominant distress of all three sections. The first cracks observed were in the shell rock section five years after construction and included primarily longitudinal cracks in the wheel paths. Cracks were first observed in the other sections two years later. Load and non-load associated crack types were observed. The shell rock section had nearly twice as much cracking as the other two sections in 2008. Since only the extent of total cracking was measured, it is not clear how much load associated cracks were present in each section. As indicated previously, the shell rock base was found to be somewhat weaker than the other sections. Relatively speaking, pavements with weaker support systems result in greater damage than those with stronger systems.

REFERENCES

1. Florida Department of Transportation. Flexible Pavement Design Manual. Document No. 625-010-002-g, Tallahassee, FL, 2008.
2. Smith, L., and W. Lofroos. Pavement Design Coefficients A Re-Evaluation of Florida Base Materials. Florida Department of Transportation, Gainesville, FL, 1981.