Florida Department of
TRANSPORTATION

Performance Evaluation of Graded Aggregate as a Base Material

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1   EXECUTIVE SUMMARY

In 1994, the Florida Department of Transportation (FDOT) initiated an experimental project on a portion of the westbound travel lane of State Road (SR) 30A between Panama City Beach and the Walton County line. The main objective of the experimental project was to compare the relative long-term performance of a graded aggregate limestone base material (GAB) to the performance of a standard limerock base. The project included two 1,500 foot test sections which were resurfaced in May 2005 after approximately 2 million equivalent single axle loads (ESALs) were applied to the roadway. Pavement performance was evaluated in terms of deflection, rut depth and ride quality. Prior to resurfacing, the overall rut depth of both sections was approximately the same, 0.23 inch for the GAB section and 0.22 inch for the limerock base section, respectively. However, the 2012 survey showed the limerock section had a relatively higher rut depth of 0.28 inch compared to 0.19 inch for the GAB section. Cracks were first observed six years following construction in the GAB section and after eight years in the limerock section. There was no significant difference in ride quality between the GAB and the limerock base sections.

2 INTRODUCTION

2.1 Background

In a flexible pavement system, a base layer provides load distribution and contributes to strength, drainage and frost protection. The properties and characteristics of a base layer play a significant role in pavement performance, especially for traditional fatigue cracking (i.e. a load-associated bottom-up cracking), and are the primary source for resistance to bending. It is generally agreed that the type, quality, and mechanical properties of base materials are important factors that affect the performance of asphalt pavements.

The FDOT's current flexible pavement design method relies on the 1993 AASHTO Method and is based on the structural number (SN) and layer coefficient concept *(1)*. The 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:

$$SN = (a_1 \times D_1) + (a_2 \times D_2) + (a_3 \times D_3) + \ldots + (a_n \times D_n) \tag{1}$$

Where, $a_{1\text{-}n}$ = layer coefficient of the 1st to n^{th} layer and $D_{1\text{-}n}$ = layer thickness of 1st to n^{th} layer.

Over the years, FDOT constructed several experimental field projects to study the effect of layer thickness and type on pavement performance. The layer coefficients shown in TABLE 1 represent the relative strength of different pavement materials used in Florida. However, the

structural layer coefficients should be developed or revised as new materials are introduced and field performance is supported by research.

TABLE 1. Structural Layer Coefficients for Different Pavement Layers in Florida

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

Previous research conducted by McNamara and Miley concluded that the GAB and limerock base materials may perform similarly based on the results of preliminary testing conducted immediately after construction *(2)*. However, the in-place pavement performance should be monitored to evaluate how the GAB and limerock base sections truly perform over time. Therefore, in 1994, an experimental project was constructed on State Road (SR) 30A in Bay County to investigate the relative long-term performance of limerock and GAB as a base material.

2.2   Objectives and Scope

The objective of this study is to compare the relative long-term performance of limerock and GAB as base materials. Pavement performance was evaluated in terms of deflection, ride quality/roughness, rutting, and cracking.

3   PROJECT DESCRIPTION

The experimental project was constructed in 1994 along the westbound travel lane of SR-30A, between Panama City Beach and the Walton County line. This four-lane divided highway consists of 12 foot lanes separated by a grassy median and four foot asphalt shoulders. The two test sections are 1500 feet long. The test sections were resurfaced in May of 2005. TABLE 2 describes the limits of the test sections, base material sources and the installed price for the GAB and limerock base. A photograph of the beginning of section 1 is shown in FIGURE 1. Also, FIGURE 2 summarizes the traffic loading that has been applied to the test sections.

TABLE 2. SR-30A Experimental Project

	Section 1	Section 2
Base Type	Graded Aggregate Limestone	Limerock
Source	Vulcan Materials Birmingham, AL	Piney Grove Mine (No. 61391) Washington County
Station (feet)	340+00 to 355+00	374+00 to 389+00
Milepost	4.577 to 4.862	5.221 to 5.505
Installed Price ($/in/SY)	1.50 to 2.00	1.00 to 1.50

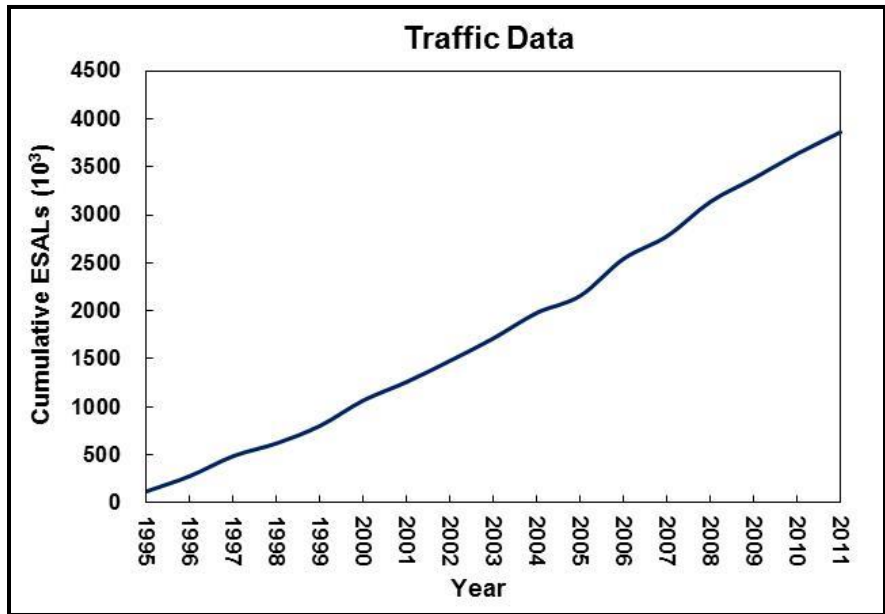**FIGURE 1. SR-30A at milepost 4.577**

FIGURE 2. SR-30A traffic information

4 SR-30A CONSTRUCTION MATERIAL INFORMATION

Roadway construction complied with the FDOT's Standard Specifications for Road and Bridge Construction, 1991 Edition. The embankment material was classified as A-3 and A-2-4 material according to the AASHTO Classification System. The subgrade consisted of embankment mixed with sand clay borrow material 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:

- Non-repetitive static plate load tests of soils (FM 5-527/AASHTO T 222)
- Moisture-density relations for embankment soils (FM 5-525/AASHTO T 99)
- Moisture-density relations for subgrade and base materials (FM 5-521/AASHTO T 180)
- Particle size analysis (FM 1-T 088/AASHTO T 88)
- Limerock Bearing Ratio (FM 5-515)
- Percent carbonates (FM 5-514)

The GAB gradation is summarized in TABLE 3. Also, TABLE 4 shows the average of three individual plate bearing, density, moisture, and LBR tests performed during construction. Results from field tests showed the embankment stiffness and density were similar for both sections. Notes from construction records indicated that it was difficult to construct a uniform subgrade using clay stabilized subgrade because it was slightly plastic and was not mixed well leading to a highly variable material depending on moisture content. Some areas where the moisture was low coincided with higher stiffness while wetter areas had a lower stiffness. While

specific subgrade LBR and plate bearing tests showed variability as indicated in TABLE 4, the average LBR results were well above the required value of 40 when all tests throughout the depth of the cross-section were considered. Both base materials passed the 100 LBR requirements. Construction records also indicated that the average as-built pavement structure for each section consisted of 4.5 inches of hot-mix asphalt (HMA), 10 inches of base material, and 16 inches of stabilized subgrade (4 inches thicker than designed).

TABLE 3. Graded Aggregate Base Gradation

Sieve Size	Section 204 Specification[1], % Passing	Average % Passing[2]
50 mm (2 inch)	100	100
19 mm (3/4 inch)	95 to 100	100
12.5 mm (1/2 inch)	65 to 90	80
9.5 mm (3/8 inch)	45 to 75	63
4.75 mm (#4)	35 to 60	51
2.0 mm (#10)	25 to 45	37
0.3 mm (#50)	5 to 25	11
0.075 mm (#200)	0 to 10	5

Note 1: The 2010 specification includes the same gradation requirements.
Note 2: Average of 3 samples. All samples met the specification.

TABLE 4. Field and Laboratory Measurements

Layer	Section / Base Type	Thickness (inch)	Plate Bearing Modulus (psi)	Field Density (pcf)	Field Moisture (%)	Lab Max Density (pcf)	Optimum Moisture (%)	% Max Density	LBR Value
Base	1 / GAB	10	12,690	138.3	2.6	142.9	5.9	96.7	169
	2 / Limerock	10	13,150	122.2	10.7	124.1	10.0	98.5	114
Subgrade	1 / GAB	16	18,500	115.8	5.3	116.0	10.5	99.7	45
	2 / Limerock	16	8,472	109.8	7.0	108.9	10.3	100.8	28
Embankment	1 / GAB	N/A	13,650	106.0	3.9	106.5	12.9	99.6	28
	2 / Limerock	N/A	10,520	104.6	5.4	104.5	12.9	100.2	31

5   PAVEMENT PERFORMANCE MONITORING

The SMO conducted annual pavement performance monitoring of the experimental sections during the same time of the year (typically in January) to better control variability due to seasonal changes (e.g. temperature, rainfall, etc.). The primary parameters used to evaluate the performance of the experimental sections include:

- Pavement deflection/stiffness
- Pavement smoothness
- Asphalt surface permanent deformation (rutting), and
- Pavement cracking

5.1   Falling Weight Deflectometer (FWD)

Annual deflection measurements were conducted in the outside of wheel path for 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 applied load
- Deflection sensors placed at 0, 8, 12, 18, 24, 36, and 60 inches from the load plate

FIGURE 3. Falling Weight Deflectometer

The relative stiffness of a pavement system may be inferred through the determination of several different deflection basin parameters. The peak deflection measured directly below the load plate is representative of the overall pavement stiffness and elastic compression of the entire pavement structure. Research has shown that 95 % of the deflections measured on the pavement surface originate at a depth in the pavement corresponding to 34° from the horizontal *(3)*. This concept applied to the pavement structure of SR-30A is illustrated in FIGURE 4. Also, according to FIGURE 4, the following deflection basin parameters may be used to evaluate the relative performance of the base, subgrade, and embankment:

$$\text{Base Response} = D\,(8) - D\,(18) \qquad\qquad (2)$$

$$\text{Subgrade Response} = D\,(24) - D\,(36) \qquad\qquad (3)$$

$$\text{Embankment Response} = D\,(60) \qquad\qquad (4)$$

Where, D (i) = deflection measured i inches from the load in mils. It is noted that a smaller deflection response corresponds to a stiffer layer (i.e. greater layer modulus).
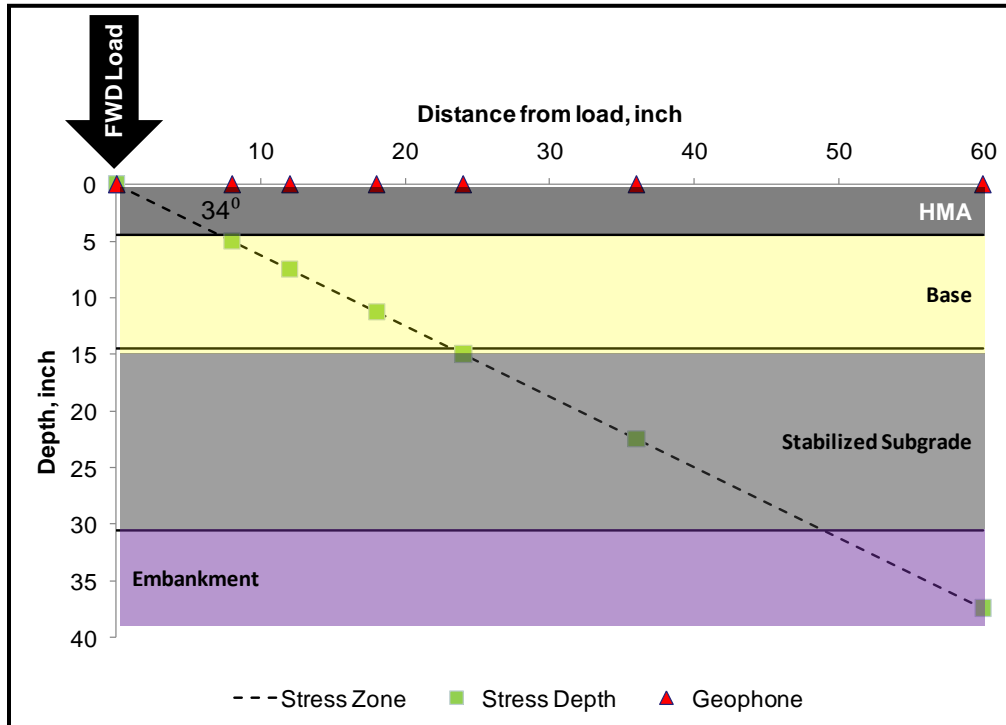

FIGURE 4. Stress distribution below FWD load

FIGURE 5 shows the base response to FWD loading for both GAB and limerock base sections over time. Prior to the 2005 resurfacing, the GAB shows a relatively stiffer response than the limerock base. After resurfacing, the difference in base stiffness between both sections became more negligible. Also, based on the FWD historical data, the overall limerock base response was greater than the GAB over time, which indicates a potentially lower initial stiffness of limerock base compared to GAB.
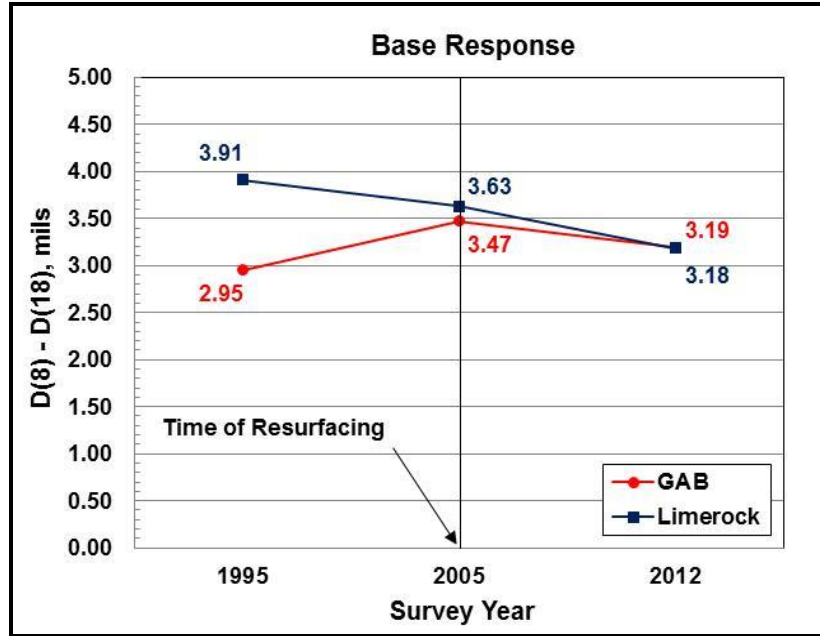
FIGURE 5. Base response from FWD loading

The deflection measured directly below the load plate indicates the stiffness of the overall pavement system. The overall pavement modulus may be calculated using the center deflection according to the following equation:

$$E_0 = \frac{1.5a\sigma_0}{d_0} \tag{5}$$

Where, E_0 = modulus of the entire pavement system beneath the load plate in psi, a = radius of load plate in inch, σ_0 = peak pressure of the FWD load plate in psi, d_0 = peak center FWD deflection in mils. FIGURE 6 shows the modulus for the overall pavement structure over time. Overall, the pavement modulus for the section with GAB was slightly higher than that of the section with a limerock base prior to the time of resurfacing in 2005 with the difference decreasing over time. Similar to the base response in FIGURE 5, the difference in pavement modulus between the two sections becomes negligible after the 2005 resurfacing. Also, up until 2005, the pavement stiffness for the GAB section decreased slightly from 69 to 61 ksi, while the

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stiffness of the limerock base section remained approximately the same at 59 ksi. However, after the 2005 resurfacing, the overall pavement modulus for both the GAB and limerock base sections increased at a similar rate over time reaching similar modulus value by 2012.
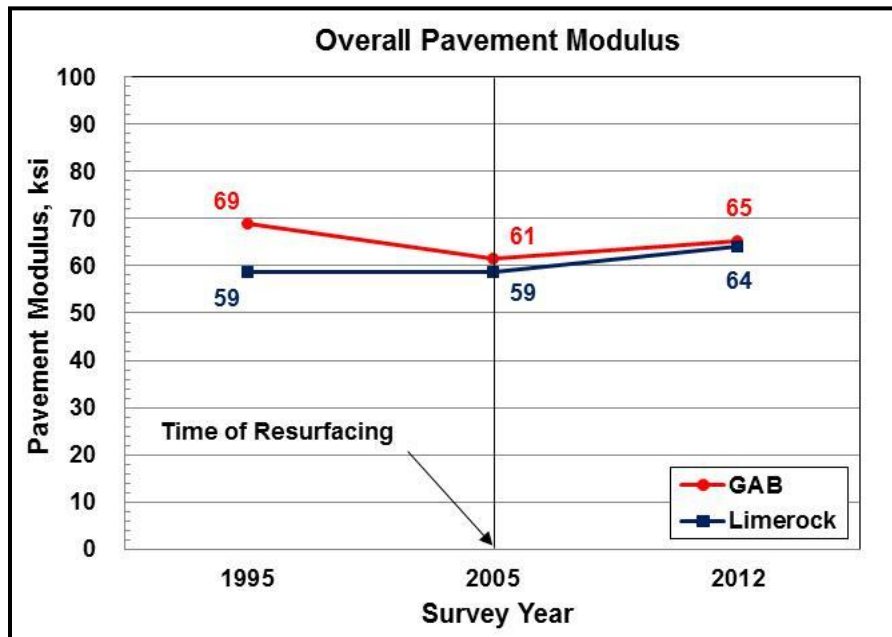

FIGURE 6. Overall pavement stiffness

5.2   Permanent Deformation (Rutting)

Permanent deformation, or rut depth, was measured manually in the right wheelpath using a 6-foot straightedge and measuring gauge. As shown in FIGURE 7, the overall rut depths of the GAB and limerock base sections were similar until the 2005 resurfacing. Prior to resurfacing, the average maximum rut depth was 0.23 inch for the GAB section and 0.22 inch for the limerock section. Both sections exhibited similar rate of change in rut depth over time. However, a significant difference in rut depth between the two sections was observed during the last four

surveys with a maximum average rut depth of 0.28 inch and 0.19 inch for limerock and GAB section, respectively. In particular, between 2008 and 2012, the limerock section increasingly showed a significant higher rate of change in rut depth than the GAB section. As indicated in FIGURE 7, the limerock base section exhibited a relatively higher rate of rutting than the GAB section over time throughout the entire project section.
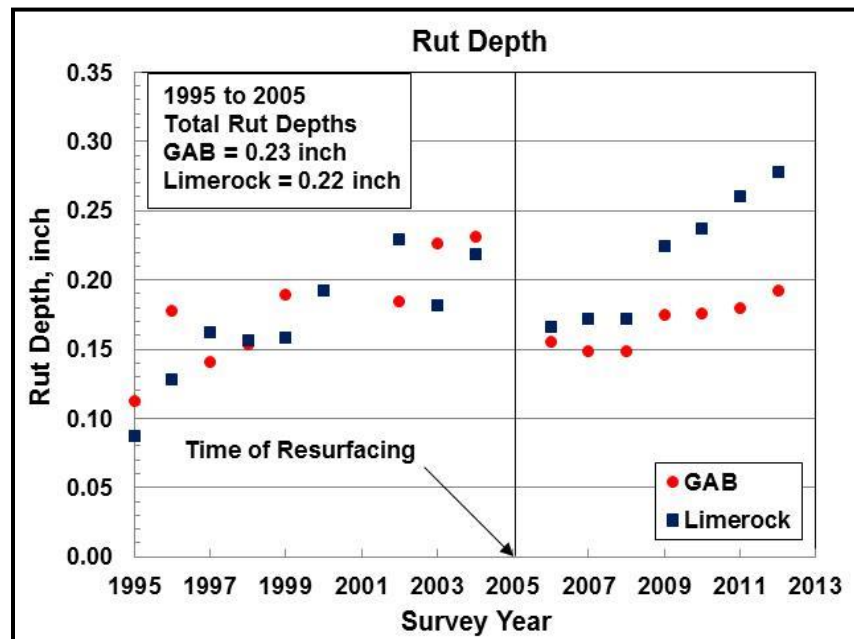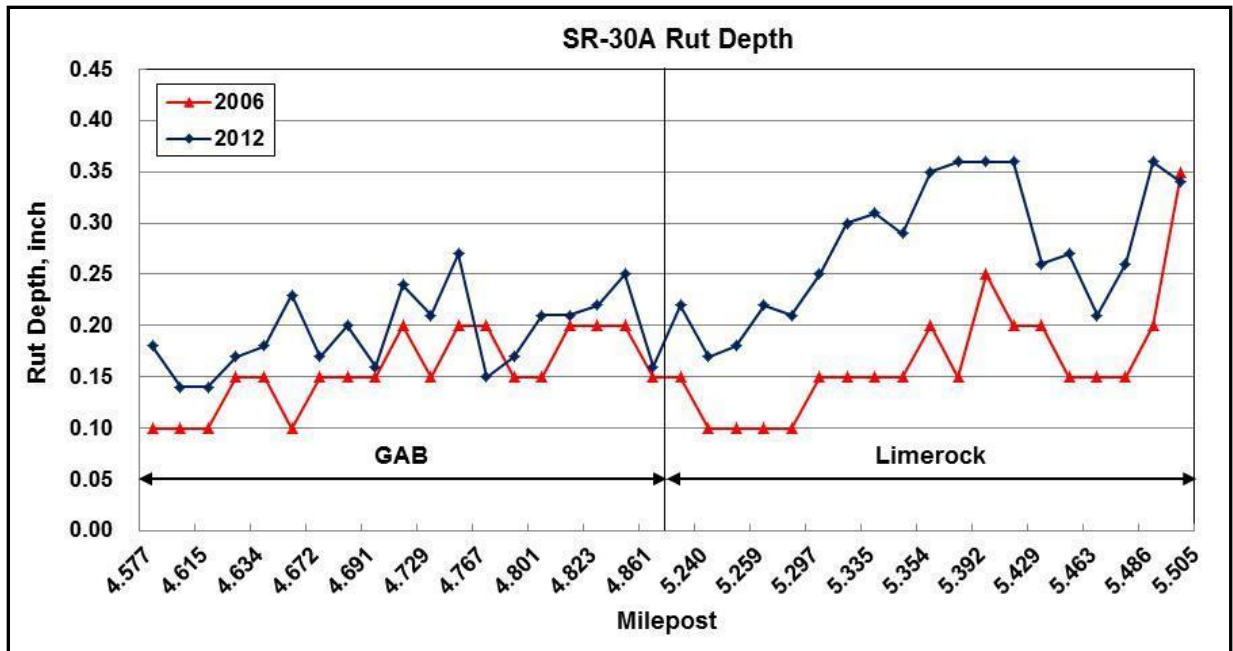

FIGURE 7. Rut depth measurements

FIGURE 8 shows the rut depth measurements in 2006 shortly after resurfacing and again in 2012 for the entire length of the experimental project section. Layer moduli were backcalculated using FWD data to determine if the increase in rut depth may be due to a deficiency in a specific layer.

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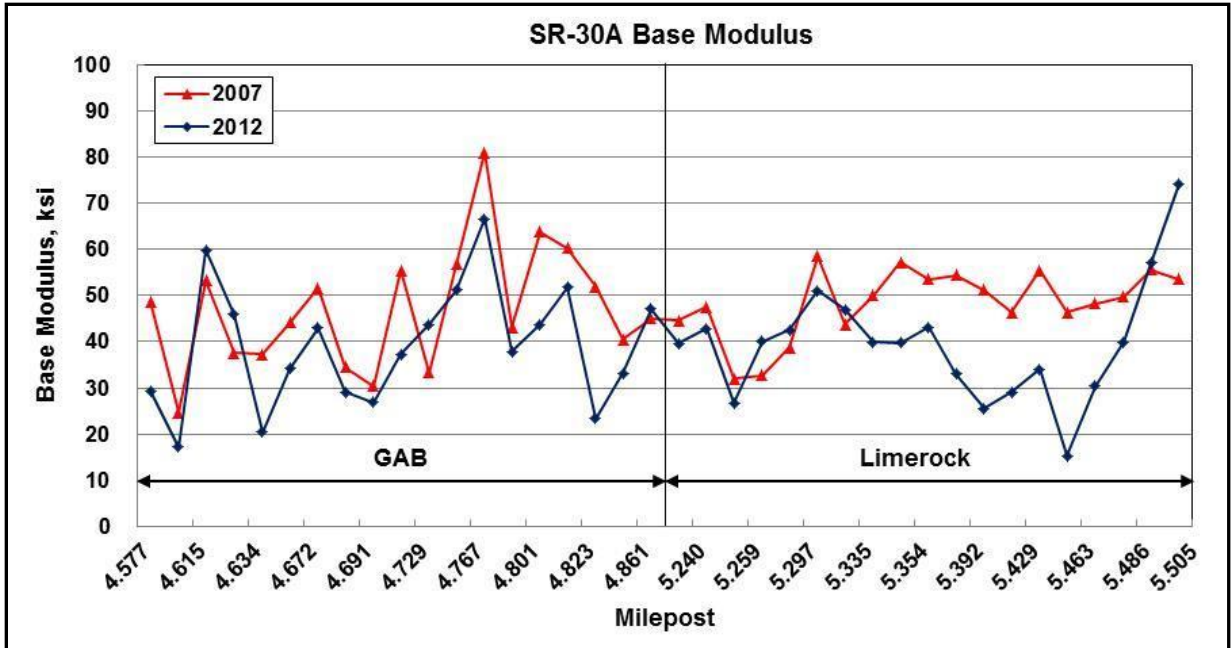| Survey Year	2006		2012	
Base Type	GAB	Limerock	GAB	Limerock
Min.	0.10	0.10	0.14	0.17
Max.	0.20	0.35	0.27	0.36
Average	0.16	0.17	0.19	0.28
Std.	0.04	0.06	0.04	0.07
%CV	23.8	36.3	19.7	23.4

FIGURE 8. Change in rut depth

FIGURES 9 through 11 represent the change of backcalculated layer moduli over time for asphalt concrete (AC), base, and subgrade/embankment. The subgrade and embankment layers were combined to simplify the backcalculation process. Based on the results in FIGURES 9 through 11, the limerock base modulus appears to be uniform just after resurfacing, but seems to have weakened and is much more variable in 2012 indicating potential damage. However, ground truth data is required to determine the exact contribution of the different layers to rutting.

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It is recommended that dynamic cone penetrometer (DCP) tests be conducted and/or trench cuts be made in both sections to better assess the condition of the different pavement layers.
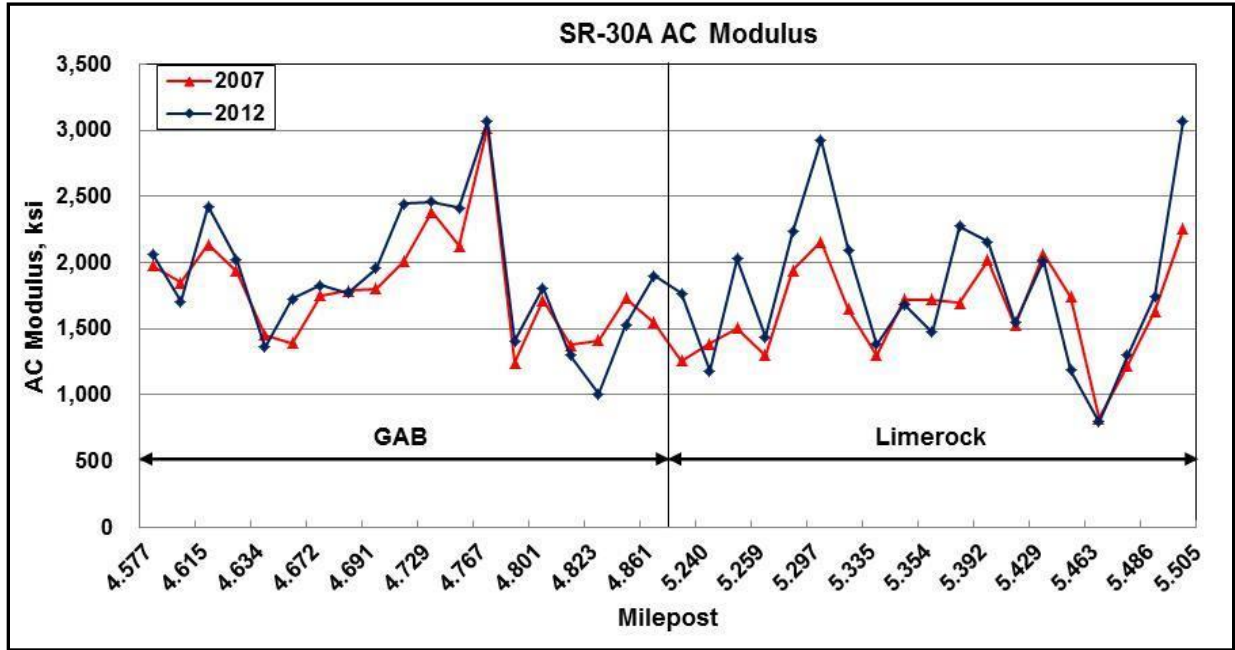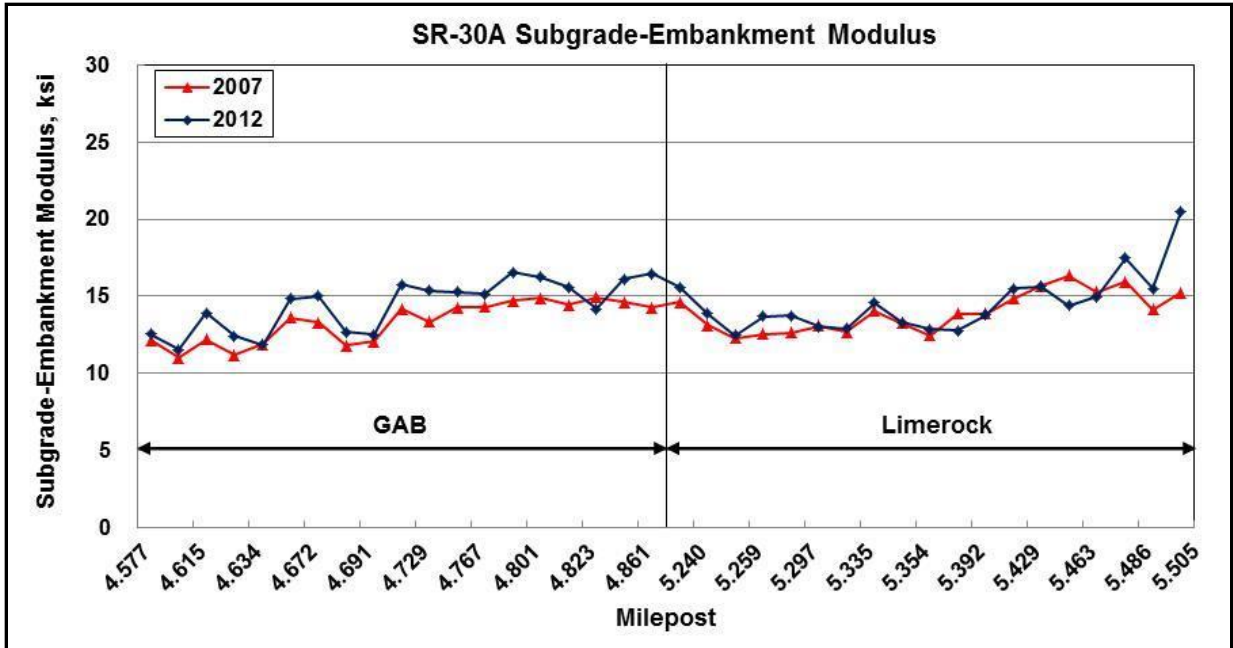

Survey Year	2007		2012	
Base Type	GAB	Limerock	GAB	Limerock
Min.	1243.53	821.05	1005.16	796.31
Max.	3020.94	2255.40	3067.40	3070.94
Average	1823.56	1627.28	1905.59	1805.68
Std.	417.65	365.56	499.04	583.51
%CV	22.9	22.5	26.2	32.3

FIGURE 9. Change in AC modulus

Survey Year	2007		2012	
Base Type	GAB	Limerock	GAB	Limerock
Min.	24.60	32.00	17.20	15.20
Max.	81.00	58.50	66.60	74.30
Average	47.04	48.42	39.04	39.53
Std.	13.42	7.59	13.23	12.82
%CV	28.5	15.7	33.9	32.4

FIGURE 10. Change in base modulus

SR-30A Subgrade-Embankment Modulus

Survey Year	2007		2012	
Base Type	GAB	Limerock	GAB	Limerock
Min.	11.03	12.26	11.52	12.45
Max.	14.93	16.35	16.56	20.50
Average	13.33	13.99	14.42	14.55
Std.	1.33	1.28	1.67	1.94
%CV	10.0	9.2	11.6	13.3

FIGURE 11. Change in subgrade-embankment modulus

5.3   Cracking

Crack measurements are conducted according to the procedure outlined in Florida's Flexible Pavement Condition Survey Handbook *(4)*. FIGURE 12 shows a summary of crack measurements. It was found that cracks were first observed in the GAB section 5 years after the initial construction and 2 years later in the limerock base section. The rate of change in cracking

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was similar for both sections. After the resurfacing in 2005, a small amount of cracking was observed in both sections for the first time during the 2011 survey.
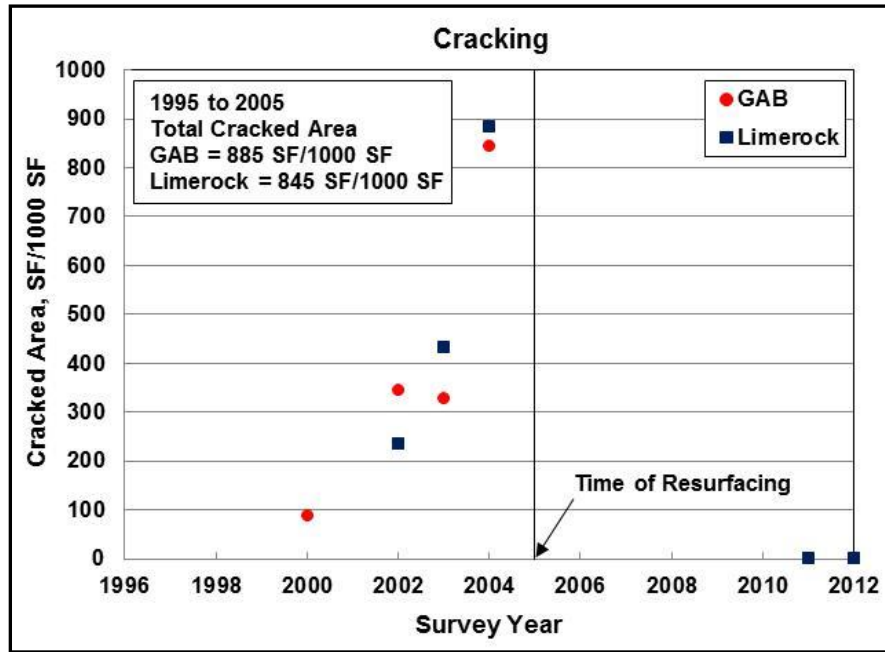
	1995 to 2005		After 2005 Resurfacing	
Section	Years Until First Crack	2005 Cracked Area, ft^2/1000 ft^2	Years Until First Crack	2012 Cracked Area ft^2/1000 ft^2
GAB	5	885	6	3
Limerock	7	845	6	2

FIGURE 12. Summary of cracking measurements

5.4   Roughness

Roughness is reported in terms of Mean Roughness Index (MRI) as shown in FIGURE 13 according to ASTM E 1926. The IRI is a mathematical process of the longitudinal pavement

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profile and is presented in units of inches per mile, where the rating scale starts from 0 representing a perfectly smooth surface and increases as the surface becomes rougher. Based on FIGURE 13, there is practically no significant difference in ride quality between the GAB and limerock base sections. Both sections exhibited an improvement in ride quality as indicated by a reduction in MRI in the years following resurfacing and the pavement smoothness has remained stable since.
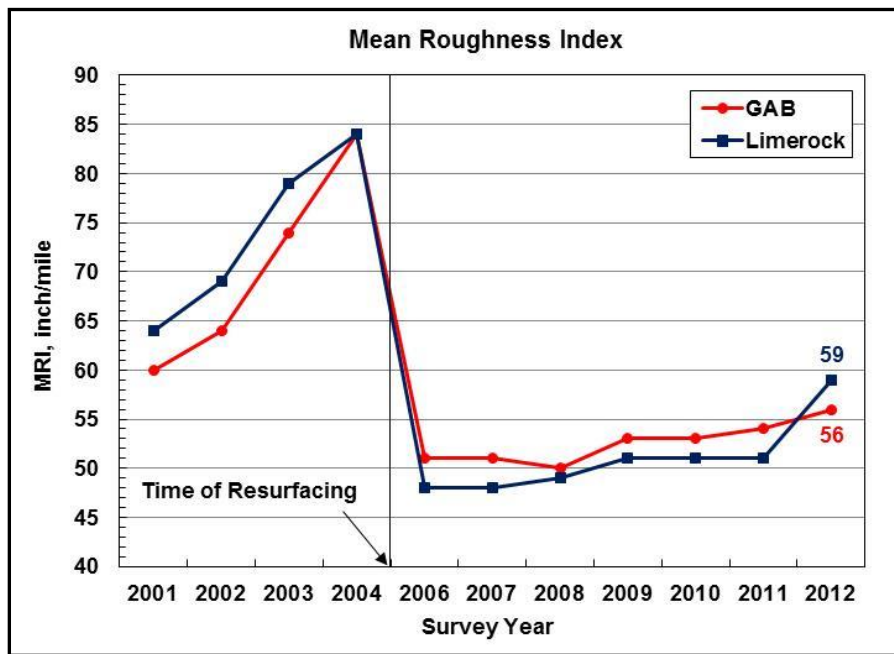

FIGURE 13. SR-30A MRI

6 SUMMARY AND CONCLUSIONS

In this study, the relative long-term pavement performance of GAB and limerock base was evaluated. Pavement performance was expressed in terms of deflection, rutting, cracking, and ride quality. A summary of findings and conclusions is presented as follows:

- Historical deflection data suggested that the limerock base exhibits slightly lower stiffness than the GAB.

- There was no significant difference in rutting performance between the GAB section and the limerock base section in the years prior to the 2005 resurfacing. However, the limerock base section had a greater rate of rutting than the GAB section since 2009. FWD data indicated possible damage to the limerock base, but further investigation is required to determine the specific cause.

- Before resurfacing in 2005, the extent and rate of cracking for both sections were similar. Six years after resurfacing, cracking is minimal and similar for both sections.

- There is no practical difference in ride quality between GAB section and limerock base section based on MRI results.

- Overall, there is little difference in performance for both sections after the initial life cycle. However, further investigation is warranted to explain the significant increase in rutting of the limerock base section since 2009.

7   ACKNOWLEDGEMENTS

8 REFERENCES

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3. Irwin, L. H. User's Guide to Modcomp2, Version 2.1. Local Roads Program, Cornell University, Ithaca, NY, 1983.

4. FDOT, *Flexible Pavement Condition Survey Handbook*. Florida Department of Transportation, Tallahassee, FL, 2009.