Florida Department of
TRANSPORTATION

Long-Term Evaluation of Geosynthetic Reinforcement of a Flexible Pavement Constructed over a Thick Organic Soil Deposit

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EXECUTIVE SUMMARY

Many regions throughout Florida have thick deposits of organic soil. Roadways built over these deposits often exhibit differential settlement, significant rut depths, and extensive cracks in a relatively short period of time. The Florida Department of Transportation (FDOT) constructed an experimental project on a realigned portion of State Road 15 on the southeastern shore of Lake Okeechobee in Palm Beach County to evaluate the effect of geosynthetic reinforcement on pavement performance. This roadway traverses farmlands with deep layers of organic material just beneath the surface and had a history of poor performance. The experimental project included four 500 feet sections with different combinations of geogrids and geotextiles placed below the base and above the organic material. A fifth section of similar length was constructed with no geosynthetic reinforcement and served as a control. In addition to geosynthetic reinforcement, the alignment was surcharged prior to construction. The investigation showed that surcharging alone significantly improved the pavement performance based on historical pavement condition and rehabilitation records. Use of geosynthetic reinforcement doubled the ESALs allowed on the unreinforced section. A rigid geogrid or woven geotextile placed below the base appeared to provide a slightly stiffer and better performing pavement than a flexible geogrid. This paper documents the research program and provides details on the improvement due to surcharging and use of geosynthetic reinforcement.

INTRODUCTION

There are many methods and techniques available to mitigate organic material or soft soils encountered in roadway construction. A recent survey of all Department of Transportation agencies found that more than 40% of the respondents completely or partially removed unsuitable soils, 20% surcharged soft soils, and nearly 15% favored deep mixing with some form of chemical stabilizers. Nearly 70% of the respondents have used a geosynthetic at the base/subgrade interface while approximately 10% have used a geosynthetic to reinforce a new asphalt overlay (*1*).

Geogrids and geotextiles are the most commonly used geosynthetic in pavement applications (*2*). Geogrids are composed of polypropylene, polyethylene, polyester, or coated polyester. Polyester and coated polyester geogrids are typically woven or knitted and are flexible. Polypropylene and polyethylene geogrids are rigid, and are either extruded or punched sheet drawn (*3*). The most common material used for geotextiles is polypropylene followed by polyester, polyethylene, and polyamide. Geotextiles are manufactured by standard weaving practices or through bonding of filaments or fibers by needle punching or melting (*3*).The primary function of geogrids and geotextiles is to stabilize the pavement support layers. In this role, stabilization refers to the reinforcement and/or separation of granular layers. In general, geogrids serve best to reinforce pavement layers while geotextiles act better as separators (*4*). However, both types have been used for either function. Geosynthetics provide a level of reinforcement through the following methods (*2, 3, 4*).

1) Lateral restraint through friction and interlock of the base, subgrade, and geosynthetic.
2) Increased bearing capacity of the system by shifting the failure envelope from the weaker subgrade to the relatively stiffer base.
3) Improved vertical stress distribution resulting from tensile stress in the deformed geosynthetic (tensioned membrane effect).

FDOT Experience with Pavements Built Over Organic Soils

Many regions throughout Florida have thick deposits of organic soils. While the largest deposit of organic soil is found in the Everglades region in southern Florida, many smaller deposits can be found throughout the state as illustrated in FIGURE 1 (*5*). Roadways built over these deposits often exhibit differential settlement, significant rut depths, and extensive cracks in a relatively short period of time. The Florida Department of Transportation (FDOT) has applied a variety of treatments to mitigate the effect of thick organic deposits on pavement performance. A brief description of some of these strategies is included in the following paragraphs.

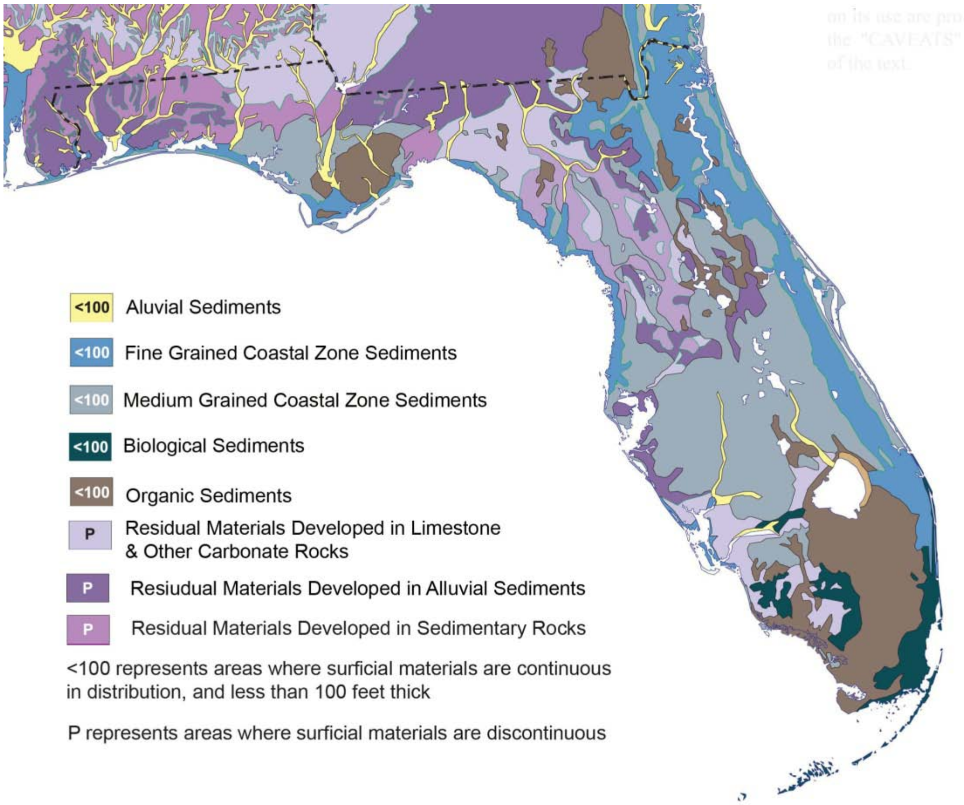


FIGURE 1 Major surface deposits in Florida

In the early 1950s, many of the roadways connecting the small communities on the southeastern shore of Lake Okeechobee were originally constructed by placing three to four feet of native rock material directly on the organic soil. Several of these roadways experienced excessive differential settlement and performed poorly. In 1973, an experimental project was initiated on State Road (SR) 80 near Belle Glade to investigate the use of different stabilization techniques including cement treatment and the use of a geotextile to separate the organic material from the pavement support layers. Based on the promising results from this study, two non-woven and one woven geotextiles were selected to be used as a separator on a 13 mile project on SR 80 in1980. The first phase of this project consisted of excavating a canal parallel to the roadway site and surcharging the organic material. In 1994, another experimental project was conducted on SR 80 to investigate a geogrid placed at the bottom of the base layer. The pavement was evaluated annually until it was rehabilitated in 2005. While the overall pavement surface deflections were of similar magnitude, the reinforced section appeared to have stiffer granular support layers. Furthermore, the geogrid reinforced section experienced approximately 40% less rutting than a control section without a geogrid (*6*).

Rehabilitation of existing pavements with unsuitable deposits is more difficult since the removal of organic material may be impractical depending on its depth and extent. Four test sections on SR 15 north of Pahokee in Palm Beach County were constructed in 2009 to study the effect of three paving fabrics and an asphalt rubber membrane interlayer (ARMI) on pavement performance. After five years of service, pavement performance is highly variable with no clear improvement due to any of the treatments compared to an untreated control section.

As indicated earlier, the Everglades area is not the only region where pavement performance is affected by the presence organic soils. One such example is a section of SR 100 in Putnam County located in northeast Florida. After several rehabilitation efforts due to excessive settlement, cement stabilized columns were constructed to support traffic load and control settlement. The grout columns were 6 inches in diameter through the pavement support layers and organic soil and then transitioned to a diameter of 10 inches within the existing asphalt pavement. A leveling course and heavy duty geocomposite stress absorbing membrane was placed above the columns followed by a 3.5 inch asphalt overlay. The geocomposite was used to transfer traffic loads to the columns. The grout columns were augured until firm end bearing was located. The columns were spaced 3 feet and each row was offset 1.5 feet to form a staggered pattern. The grout columns were found to increase the pavement stiffness, but differential settlement around the columns resulted in protrusions through the pavement surface as shown in FIGURE 2. The control section and the section with grout columns still had good rutting performance after an 18 month evaluation.

FIGURE 2 Cement stabilized column protrusions in the asphalt surface on SR 100.

EXPERIMENTAL OBJECTIVE

The objective of this research was to evaluate the potential performance improvement of a flexible pavement constructed over a thick deposit of organic material following a combination of mitigating treatments. The roadway was first surcharged and geotextiles and geogrids were used to provide reinforcement and to separate select fill from the organic material. Pavement performance was measured annually from 1999 through 2011.

PROJECT DESCRIPTION

SR 15 is a rural principal arterial and one of only two routes that connect Pahokee with Belle Glade and other communities located in western Palm Beach County near the southeastern shore of Lake Okeechobee. In 1997, FDOT initiated a realignment of a portion of SR 15 since reconstruction of the existing roadway was not a feasible alternative. Prior to realignment, the existing flexible pavement was in very poor condition due to thick layers of organic material and heavy traffic loads associated with farming equipment and heavy trucks. A review of historical pavement condition ratings, summarized in TABLE 1, indicates the roadway was rehabilitated often due to issues with differential settlement which led to excessive rutting and cracking. The realigned roadway was opened to traffic in 1999 and was found deficient in 2011. A rehabilitation consisting of milling and resurfacing was completed in 2012.

TABLE 1 SR 15 Historical Pavement Condition and Rehabilitation Summary

Year Rehabilitated	Year Deficient	Years until Deficient	Years until Next Rehabilitation
1978	1979	1	5
1983	1987	4	13
1996	1997	1	3
1999	2011	12*	13
* Experimental project with geosynthetic reinforcement and/or surcharge			

According to the United States Department of Agriculture (USDA) soil survey, Torry Muck is the predominant soil type in the location of the experimental project (*7*). A geotechnical investigation prior to construction revealed 7 to 10 feet of organic soil (AASHTO classification A-8) above a sandy silt marl layer (AASHTO classification A-4/A-5). Organic contents ranged from 6% to 87% and moisture contents ranged from 26% to 510%. Undrained shear strength values averaged 500 psf above the water table and 300 psf below the water table.

The experimental portion of the realignment included five 500 foot test sections. Four test sections were constructed with different combinations of geogrids and geotextiles within the pavement support layers. A fifth test section was constructed with no reinforcement and was used as the control. Prior to construction of the reinforced and control sections, the organic soil was surcharged with 5 feet of fill material with a dry density of 110 lb/ft^3 for 180 days. Settlements of 6 inches to 2 feet were recorded from 24 settlement plates. The pavement consisted of 3.5 inches of hot-mix asphalt (HMA), 10 inches of shell rock base, and 12 inches of granular subgrade consisting of a mixture of shell rock, and select fill material placed at depths of 1.5 to 4 feet depending on the elevation of the organic soil. Geosynthetics were placed below the shell rock base and also at the interface of the organic material and fill. TABLE 2 shows the location and types of geotextiles and geogrids placed within each experimental section. TABLE 3 shows manufacturer reported properties of the geosynthetics used in this study.

TABLE 2 Type and Location of Geosynthetics

Section	Base/Subgrade Interface	Select Fill/Peat Interface
1	Rigid Extruded Geogrid	Rigid Extruded Geogrid
2	Rigid Extruded Geogrid	Woven Geotextile
3	Woven Geotextile	Woven Geotextile
4	Flexible Woven Geogrid	Woven Geotextile
5	No Reinforcement	No Reinforcement

TABLE 3 Manufacturer Reported Geosynthetic Properties

Ultimate Tensile Strength	Test Standard	Rigid Extruded Geogrid	Woven Geotextile	Flexible Woven Geogrid
Machine-direction (lb/ft)	ASTM D 6637	850	2,400	1,159
Cross-direction (lb/ft)	ASTM D 6206	1,300	2,400	1,641

A traffic monitoring site was added in 2006 just south of the experimental project. Since installation, this traffic monitoring site has recorded approximately 600,000 ESALs in both travel directions. A second traffic monitoring site located approximately 5 miles north of the project has been in operation since the beginning of the project. Using traffic data from both sites, nearly 1.8 million equivalent single axel loads (ESALs) were applied in each direction during the life of the experimental project

PAVEMENT PERFORMANCE MONITORING

Pavement performance monitoring of SR 15 was conducted annually from 1999 to 2011. Pavement condition surveys were performed during October of each year to minimize variability due to seasonal changes (e.g., temperature, rainfall, etc.). The primary parameters used to evaluate performance of the experimental sections included pavement deflection, rutting, cracking, and ride quality.

Pavement Deflection

Deflection measurements were made in the outside wheel path of each lane using a Falling Weight Deflectometer (FWD). The FWD is a non-destructive pavement evaluation tool that applies an impulse load to the pavement surface. The magnitude and duration of the impulse load is similar to that of a 9 kip truck tire travelling at typical highway speeds. Geophones placed on the pavement surface measure the deflection due to the impulse load.

The relative stiffness of a pavement system may be inferred through the determination of various deflection basin parameters (DBP). Research has shown that 95 percent of the deflections measured on the pavement surface originate at a depth in the pavement corresponding to 34^o from the horizontal (*8*). This concept applied to the SR 15 pavement structure is illustrated in FIGURE 3. According to this figure, the following DBPs may be used to evaluate the relative performance of the base and stabilized subgrade:

$$\text{Base DBP, mils} = D(8) - D(18) \qquad (1)$$

$$\text{Subgrade DBP, mils} = D(24) - D(36) \quad (2)$$

Where,

D(*i*)= deflection measured *i* inches from the load, mils

Note: A smaller deflection represents a stiffer layer

Deflections were adjusted for variations in load and to a reference asphalt temperature of 68°F (*9*). The BELLS equation was used to estimate the mid-depth asphalt temperature and correction factors were used to adjust the deflections.

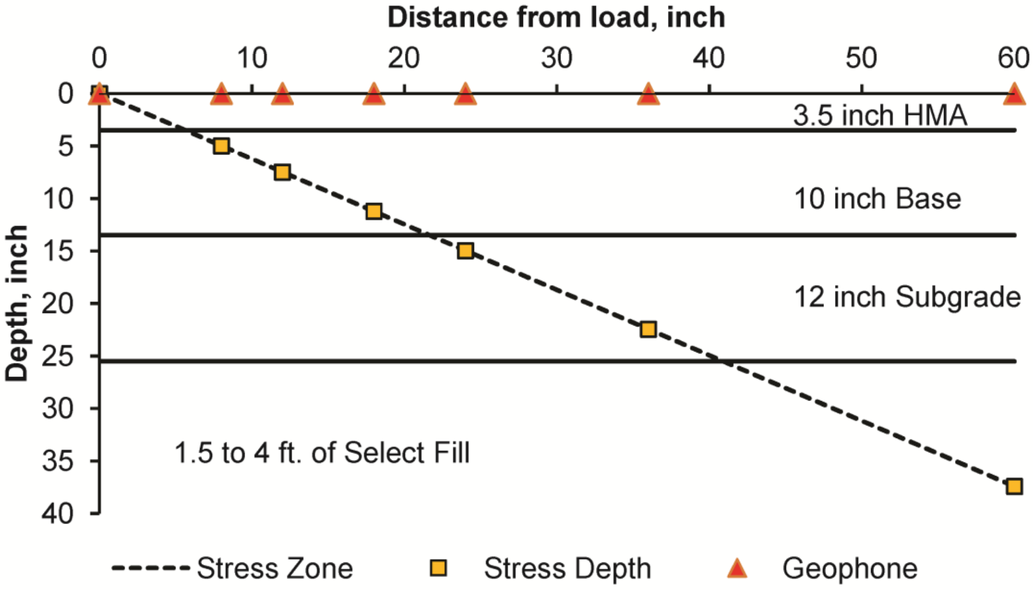
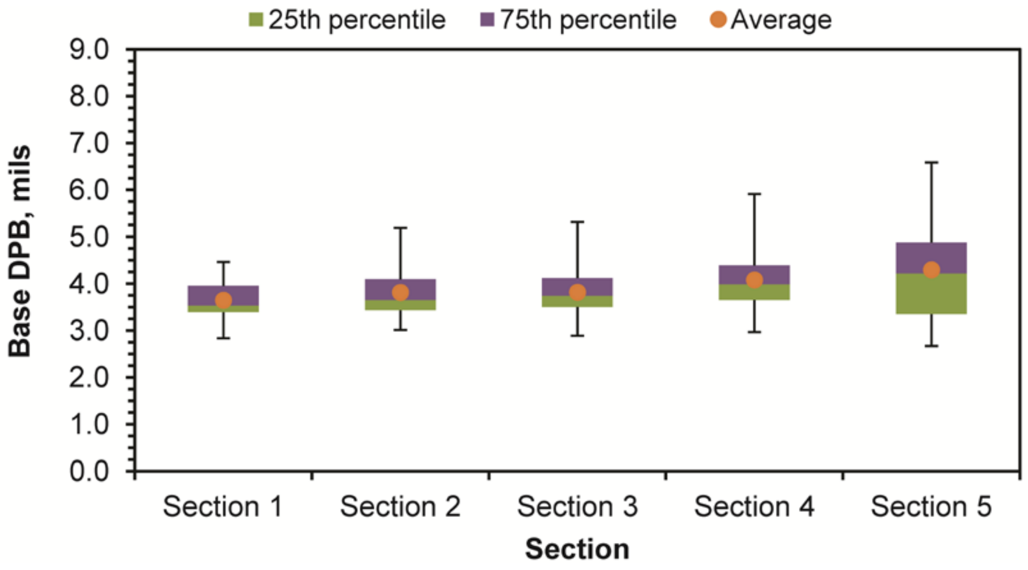
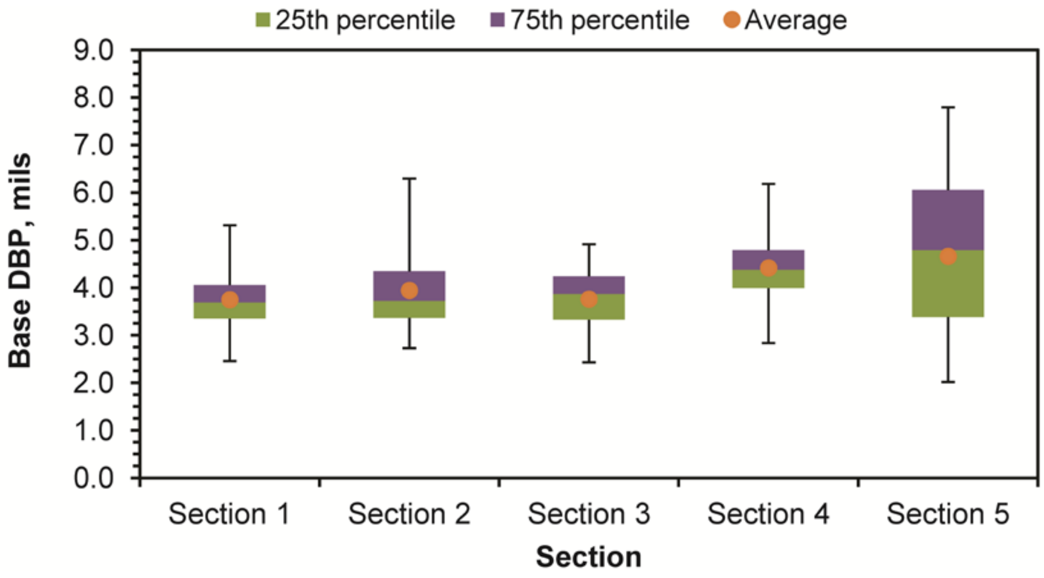


FIGURE 3 Stress distribution below a FWD load.

Despite adjustments for temperature, some variability from year to year was found due to environmental changes not accounted for such as moisture conditions. However, general trends were still evident. In order to account for the variability, the individual DBPs of the first and last three years were grouped and averaged for analysis. FIGURE 4 and FIGURE 5 show the base and subgrade DBPs, respectively. The Tukey Method was used to compare the average DBP of each section over the grouped years. The Tukey Method is a single-step multiple comparison procedure used in conjunction with an analysis of variance (ANOVA) to find means that are significantly different from each other. The statistical analysis is summarized in TABLE 4.

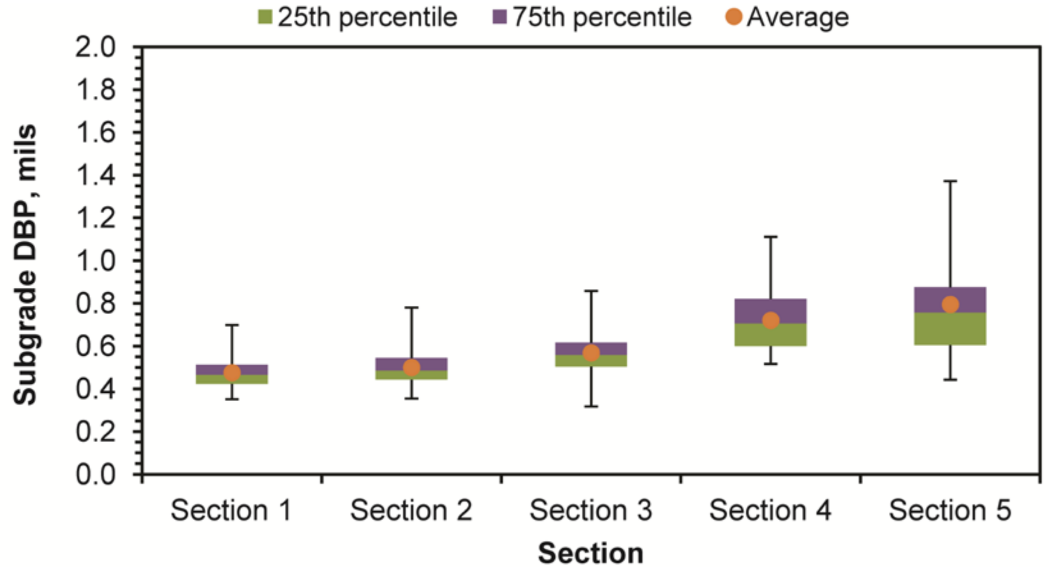


(a) Average of first three years.

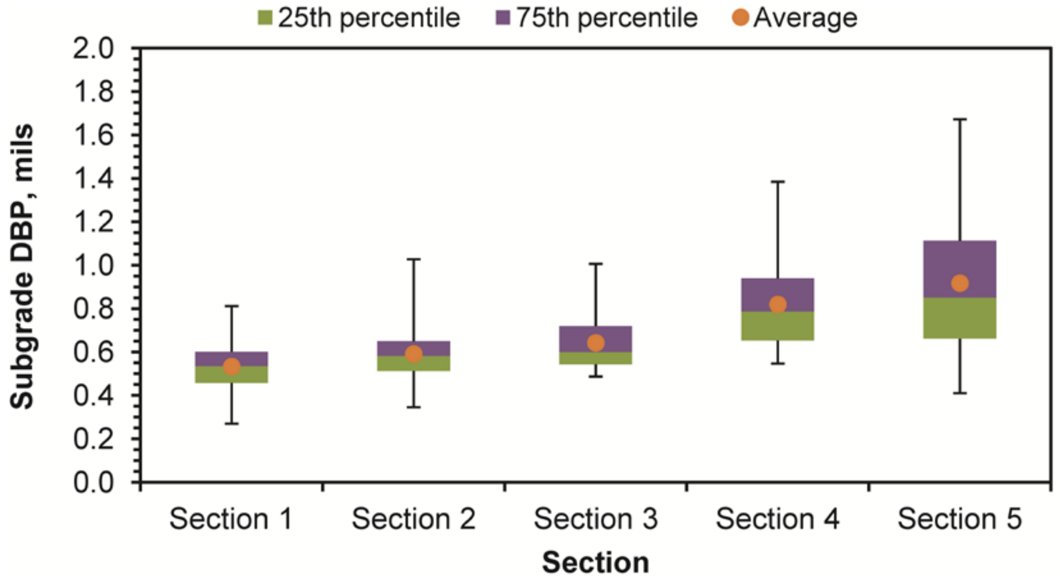


(b) Average of last three years.

FIGURE 4 Base Deflection Basin Parameter.



(a) Average of first three years.



(b) Average of last three years.

FIGURE 5 Subgrade Deflection Basin Parameter.

TABLE 4 Comparison of Deflection Responses using the Tukey Method (α = 0.05)

Section	First Three Years				
	1	2	3	4	5
Base Response, mils	3.7	3.8	3.8	4.1	4.3
	Sections 1 to 3 Equivalent				
		Sections 2 to 4 Equivalent			
				Sections 4 & 5 Equivalent	
Subgrade Response, mils	0.48	0.50	0.57	0.72	0.80
	Sections 1 & 2 Equivalent				
		Sections 2 & 3 Equivalent			
				Sections 4 & 5 Equivalent	
Section	Last Three Years				
	1	2	3	4	5
Base Response, mils	3.7	3.9	3.8	4.4	4.7
	Sections 1 to 3 Equivalent				
		Sections 2 to 4 Equivalent			
				Sections 4 & 5 Equivalent	
Subgrade Response, mils	0.53	0.59	0.64	0.82	0.92
	Sections 1 to 3 Equivalent				
				Sections 4 & 5 Equivalent	

When considering the overall deflection responses and trends, it appears that the combinations of rigid extruded geogrid and/or woven geotextile installed at the base/subgrade interface with a rigid extruded geogrid or woven geotextile placed above the organic material layer (Sections 1, 2, and 3) increased the pavement stiffness while the flexible woven geogrid and geotextile placed at the same locations (Section 4) did not significantly increase the pavement stiffness when compared to the unreinforced section (Section 5). A greater rate of increase in base and subgrade DBP suggests a faster degradation of support layers in Sections 4 and 5 and possibly intermixing of layers in Section 5. The following can be derived from FIGURE 4 and FIGURE 5 and the statistical analysis summarized in TABLE 4.

1. The base DBPs of Sections 1, 2, and 3 are statistically equivalent at the beginning and end of the pavement life. Section 1 has the stiffest base DBP at the beginning and end of the pavement life.
2. The base DBPs of Sections 2, 3, and 4 are statistically equivalent at the beginning and end of the pavement life.
3. The base DBPs of Sections 4 and 5 are statistically equivalent at the beginning and end of the pavement life. Section 5 has the weakest base DBP at the beginning and end of the pavement life.

4. The subgrade DBPs of Sections 1, 2, and 3 are equivalent at the end of the pavement life, but the Section 3 subgrade DBP is weaker at the beginning of the pavement life.
5. The subgrade DBP of sections 4 and 5 are statistically equivalent at the beginning and end of the pavement life. Section 5 has the weakest subgrade DBP at the beginning and end of the pavement life.
6. The rate of increase in base DBP from the beginning to the end of the project was at least 2.5 times greater for sections 4 and 5 as compared to sections 1, 2, and 3.
7. The rate of increase in subgrade DBP from beginning to the end of the project was slightly greater for section 5 as compared to the other sections.

Rutting

Permanent deformation, or total rut depth, was measured manually in 50 foot increments in the right wheelpath using a 6 foot aluminum straightedge and digital depth gauge. The total rut depth was measured as the distance from the top of the upheaval to the deepest point in the trough. The rut depth summary for the combined travel directions is shown in FIGURE 6. Rut depths in Section 5 have consistently been greater than all other sections. Sections 1, 2, and 3 have shown the best rut performance on average. In general, a power function can be used to describe the rut depth profiles throughout the life of the project. However, a linear portion of the rut depth history was used to project the remaining number of ESALs required to generate a critical rut depth value of 0.5 inches. For all practical purposes, Section 5 had reached the critical rut depth during the last survey in 2011.

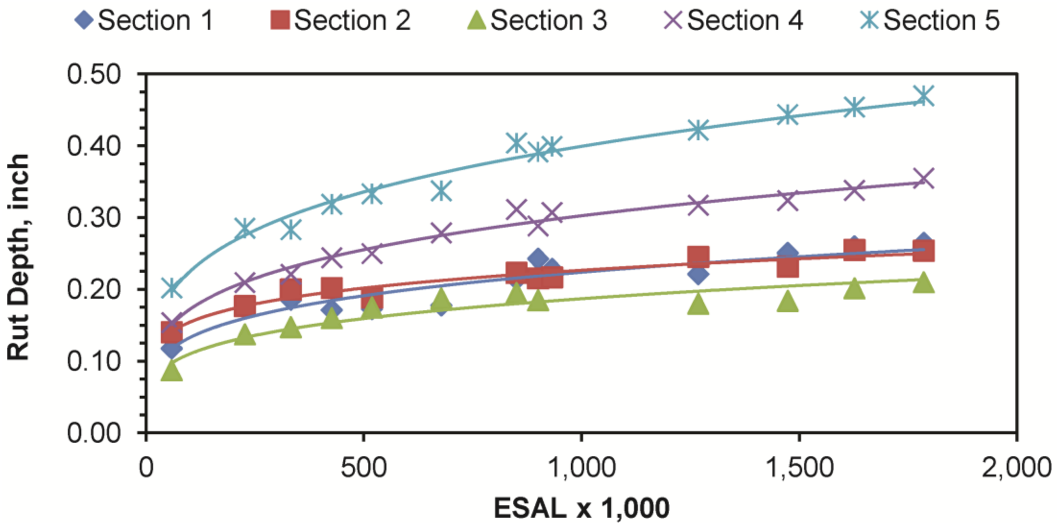


FIGURE 6 Rut depth measurements and projections.

A Traffic Benefit Ratio (TBR) was determined to demonstrate the additional traffic that could be allowed on the reinforced sections as shown in TABLE 5. A TBR of 1.8 or greater was found for the reinforced sections. However, Sections 1 through 3 had TBR values of 2.4 or greater, indicating that a rigid extruded geogrid or a woven geotextile placed at the base/subgrade interface provided greater rut resistance than the flexible woven geogrid used in Section 4. The TBR is calculated as follows:

$$TBR = N_R / N_U \tag{3}$$

Where,

N_R = ESALs applied to reinforced section required to reach failure

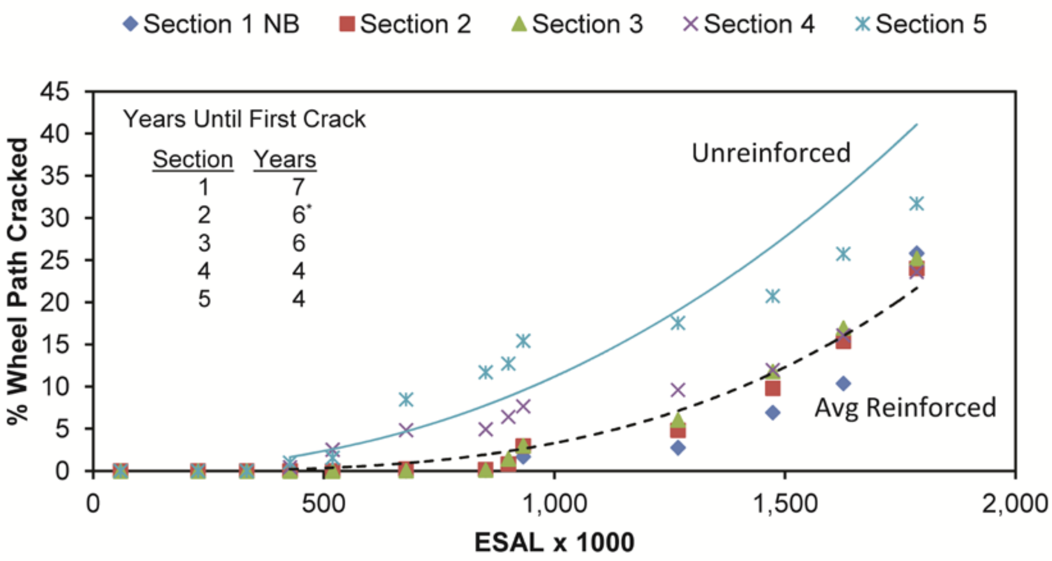
N_U = ESALs applied to unreinforced section required to reach failure

TABLE 5 Rut Depth Summary and Traffic Benefit Ratio

Section	2011 Rut Depth, in.	Remaining ESALs (x 1000) Projected for 0.5 in. Rut Depth	Rut TBR
1	0.27	2,760	2.4
2	0.25	4,950	3.6
3	0.21	6,340	4.4
4	0.36	1,490	1.8
5	0.47	80	1.0

Cracking

Crack measurements consisted of the combined length of cracks measured within each section. Cracks were first observed in Sections 2 (southbound), 4, and 5 after four years of service. However, it should be noted that a total crack length of 1 foot or less was found in the southbound direction of Section 2 in years four and five compared to 30 to 60 feet in Sections 4 and 5, respectively. Cracks were first identified in Sections 1, 2 (northbound), and 3 after six and seven years of service. The majority of cracks were found in the wheelpaths along the longitudinal direction. Overall, more cracks have been found in Section 5 than in any other section.
FIGURE 7 shows the crack initiation and development over the pavement life.



* Cracks 1 ft or less found in year 4 and 5 in the southbound direction only

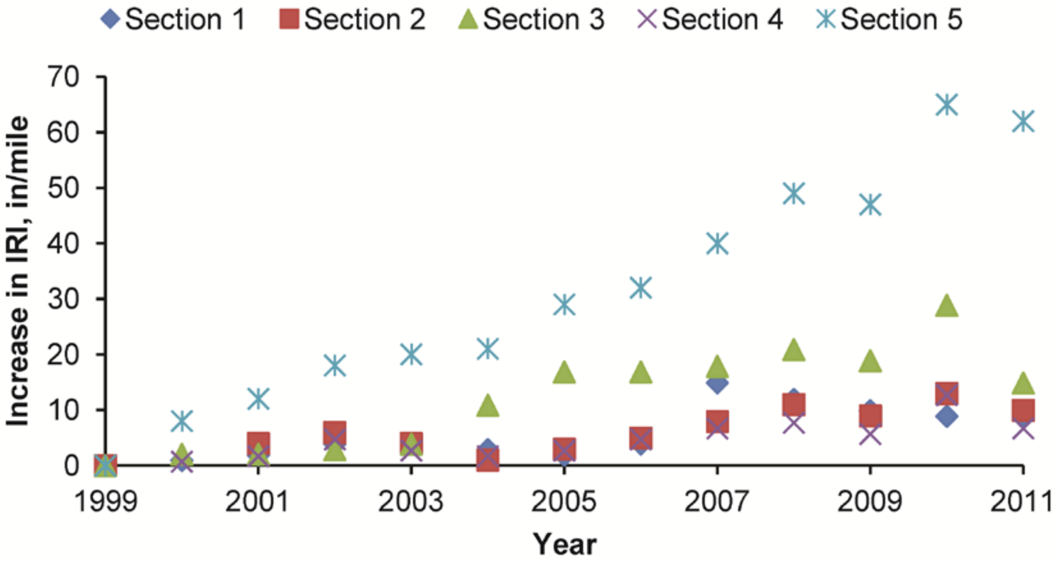
FIGURE 7 Crack measurements.

According to the 2011 crack survey, approximately 25% of the wheelpath area in Sections 1 through 4 (excluding the southbound direction of Section 1) was found to have cracks. The southbound direction of Section 1 experienced more accelerated cracking than the northbound direction, with an average total cracking similar to that of Section 5. Based on the same survey, approximately 32% of the wheelpath area in Section 5 was found to have cracks. According to FDOT pavement condition rating methods, the extent and severity of cracking located inside and outside the wheelpaths is considered when determining if a pavement is deficient. When considering only traffic related cracking, a flexible pavement is considered deficient when cracks make up more than 25% of the wheelpath. Based on this criterion, the reinforced sections had a TBR of 1.3 using equation 3.

Four cracks in each section were cored to measure crack depth, verify HMA thickness, and to possibly determine if cracking originated from the bottom or top of the HMA. The average core thickness was 3.7 inches and the average crack depth was 1.7 inches. Full depth cracks were observed in seven of the 20 cores, while the remaining 13 cores indicated that cracks originated from the HMA surface. Top-down cracking cannot be explained by the same mechanisms attributed to traditional bottom-up fatigue cracking. Research has suggested that regardless of the crack origination, pavements with weaker support systems tend to have a greater amount of damage. Factors thought to be responsible for top-down cracking include aging and healing of the asphalt mixture, pavement thickness and stiffness, and thermal cycles (*10*).

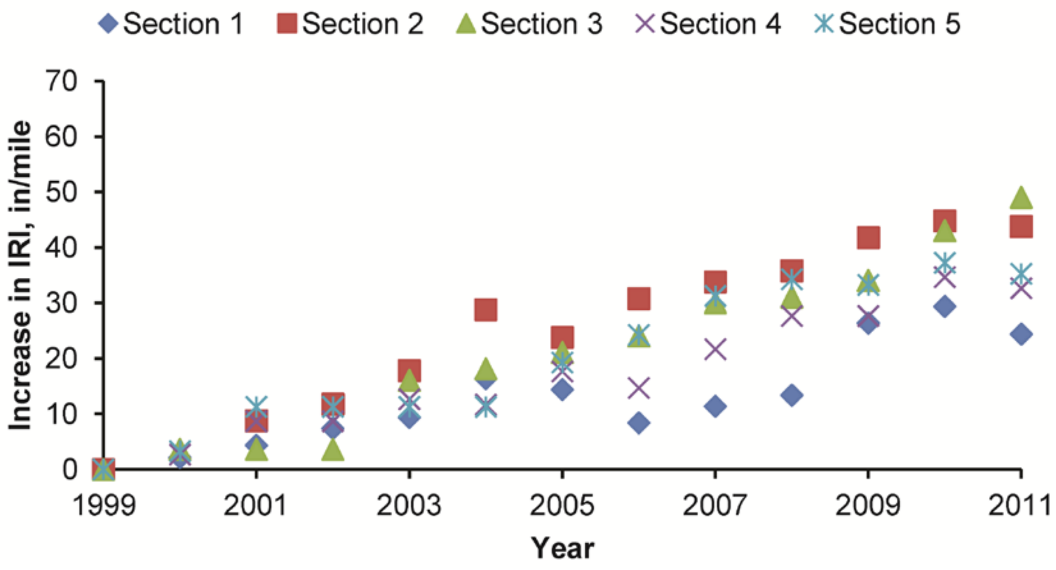
Ride Quality

Longitudinal profiles were measured annually using a high-speed inertial profiler and pavement smoothness was determined in terms of the average International Roughness Index (IRI) of both wheelpaths. The results in FIGURE 8 show the northbound lane was initially smoother than the southbound lane and remained smoother after 12 years of service. FIGURE 8 also shows that roughness in the southbound lane increased at a greater rate than roughness in the northbound lane except for Section 5. This supports previous research findings that pavements built smooth tend to remain relatively smooth over time. Except for Section 2 in the southbound lane, all reinforced sections provided a considerably smoother ride quality compared to the non-reinforced section. In general, the deterioration in ride quality was within typical expectations for all sections when compared to statewide historical data.



IRI	Section 1	Section 2	Section 3	Section 4	Section 5
1999	62	82	81	49	67
2011	71	92	96	56	129
Slope	1.6	2.6	2.8	1.8	4.3

(a) Northbound Lane



IRI	Section 1	Section 2	Section 3	Section 4	Section 5
1999	81	90	72	77	93
2011	105	134	121	110	128
Slope	2.2	4.2	3.6	2.9	3.5

(b) Southbound Lane

FIGURE 8 Pavement smoothness.

ECONOMIC ANALYSIS

A life cycle cost analysis (LCCA) was conducted to quantify the potential savings attributed to geosynthetic reinforcement and surcharging. The analysis was conducted over a 40 year period using 3.5% discount rate for a 12 ft wide lane over a one mile segment. Shoulders, additional lanes, inflation, and user costs were not included in the analysis. A standard mill and overlay was assumed to take place at the end of the service life. Historical construction costs were obtained from FDOT's Specifications and Estimates Office.

Pavement service lives were estimated based on rutting and cracking performance. Rehabilitation of rural roads that have become deficient due to cracking may sometimes be delayed due to budget limitations. However, rutting deficiencies may pose a safety hazard and will not likely be postponed. Therefore, two service lives were used for the reinforced sections to reflect the better rutting performance. A four-year service life based on historical performance prior to reconstruction was used for comparison. TABLE 6 summarizes the LCCA of each section. In general, all of the experimental sections were found to have a minimum reduction in present worth of 27% when compared to the historical performance. Up to 3% additional reduction was possible when reinforcement was included and rutting only was considered.

TABLE 6 Life Cycle Cost Analysis

Experimental Section	Description	Service Life	Initial Cost per Lane Mile	Present Worth per Lane Mile	Difference in Present Worth per Lane Mile	% Difference in Present Worth per Lane Mile
1 to 4	Reinforcement & Surcharge	20 yrs (rutting)	$395,000	$481,841	$215,808	31
		14 yrs (cracking)	$395,000	$506,062	$191,588	27
5	Surcharge Only	11 yrs	$340,000	$500,471	$197,178	28
Historical Performance	Typical Rehab (no mitigation)	4 yrs	$115,000	$697,649	Reference	Reference

SUMMARY AND CONCLUSIONS

The pavement performance of a rural arterial constructed over a thick layer of organic soil was evaluated from 1999 to 2011. During this time, approximately 1.8 million ESALs were applied to the pavement. Historically, the previous alignment became deficient within one to four years after rehabilitation. The investigation showed that the life of the experimental sections was increased by 11 years due to surcharging alone and the reinforced sections were projected to approximately double the allowable ESALs before failure occurred based on rutting criterion. Specific conclusions are listed below.

1. A rigid extruded geogrid or woven geotextile placed at the base/subgrade interface was found to produce a stiffer pavement when compared to a section constructed with a flexible woven geogrid and an unreinforced section. Sections with a rigid extruded geogrid or woven geotextile at the bottom of the base were projected to allow at least 2.4 times more ESALs than an unreinforced section.
2. A flexible geogrid placed at the bottom of the base did not significantly increase the pavement stiffness but could potentially allow 1.8 to 4.4 times more ESALs compared to the unreinforced section.
3. Slightly more wheelpath cracking was found in the unreinforced section compared to any of the reinforced sections.
4. The reinforced sections provided a smoother pavement than the unreinforced section and the smoothness deteriorated less quickly. All sections were within expected smoothness ranges when compared to statewide historical data.
5. When applied to other projects, the level of improvement found in this study would in all likelihood diminish as the stiffness of the in-situ soil increases. However, surcharging

organic soils and the use of a rigid extruded geogrid or woven geotextile at the bottom of the base layer combined with a woven geotextile placed above the organic material layer is recommended for new roadways constructed over thick deposits of organic material and when reconstruction is a viable alternative.

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