State of Florida

Department of Transportation

A 30-Year Performance Evaluation of a Two-Layer Concrete Pavement System

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EXECUTIVE SUMMARY

In 1978, an experimental two-layer concrete pavement was opened to traffic on State Road 45 near Fort Myers, Florida. The experimental pavement included a series of two-layer pavement sections with various design features placed over either a granular or cement treated subbase. These sections consisted of a 3 inch (7.5 cm) Portland cement concrete (PCC) surface over a 9 inch (23 cm) lean concrete (commonly referred to as econocrete) layer. The control section consisted of a standard 9 inch (23 cm) thick PCC with joints spaced 20 feet (6 m) on a cement-treated subbase. After 30 years of service, the sections constructed over a granular base performed better than those placed over a cement treated subbase. The distresses on the two-layer concrete pavement sections built on a granular subbase were minimal irrespective of their slab lengths. In contrast, the control section experienced greater cracking and moderate to severe spalling.

The findings validates several features of Florida's current design policies such as limiting joint spacing to 15 feet and prohibiting cement treated subbases below concrete pavements. Furthermore, this experimental project has also demonstrated that a two-layer concrete system consisting of a relatively thin higher quality PCC surface over a lower quality econocrete layer and a granular subbase can be a sustainable and long lasting pavement design alternative.

INTRODUCTION

Background

High quality, durable aggregates used to construct pavements are becoming scarce and costly. In return, the construction industry has begun to focus on methods that promote sustainability while preserving quality. Sustainable construction methods balance economic, environmental, and social benefits. One such method is the two-layer concrete pavement system. This system consists of a thin high quality concrete surface placed on top of a thick econocrete layer of lower quality. This type of composite pavement allows the designer the ability to balance the need for high quality aggregates for surface friction and durability with economic and environmental concerns.

Two-layer concrete pavements are not a new concept in the United States (U.S.). In fact, the first U.S. concrete pavement constructed in 1891 in Bellefontaine, Ohio consisted of a two-layer system (*1*). In the past, many states specified two-layer concrete pavements to facilitate the placement of steel-reinforcing mesh. However, in the early 1970's the concrete paving industry began to move away from a jointed reinforced concrete pavement design and the use of mesh has been largely discontinued (*2, 3*). Since then, two-layer concrete pavements have been primarily used in experimental or demonstration projects. The focus of these pavements has shifted to promote the use of recycled materials and maximize local aggregate sources (*4*).

In March of 1978, an experimental two-layer concrete pavement was opened to traffic on State Road (SR) 45 in Charlotte County, Florida. The experimental sections consisted of a 3 inch (7.5 cm) Portland cement concrete (PCC) surface with a 9 inch (23 cm) lean concrete (commonly referred to as econocrete) layer. Sections were constructed over either 6 inch (15.0 cm) cement-treated or 6 inch (15 cm) granular subbases. The control section was a standard 9 inch PCC on top of a 6 inch (15 cm) cement-treated subbase. The econocrete layer aggregate source was located four miles from the project and consisted of a mixture of rock, sand, and shell produced from the dredging of canals. In the past, this aggregate source had been used as fill material since it did not meet FDOT's concrete or base aggregate specifications. The aggregate was crushed on-site at the dredging location to produce a uniform aggregate supply and then transported to the batch plant site and stockpiled. FIGURE 1 shows photographs of the dredged canal, raw material, processing, and crushing of the econocrete aggregate source.

FIGURE 1 Econocrete aggregate source.

Two-Layer Concrete State of the Art

Two-layer concrete paving is a wet-on-wet process. That is, the surface layer is placed over the lower layer while the concrete is still wet. The placement of the surface lift is timed so that the bottom lift is stiff enough to resist mixing of the materials and still sufficiently wet to promote a good bond so that a monolithic concrete structure is formed. The time between placing lifts is often no more than 30 minutes (*5*). The surface layers have primarily consisted of jointed plain concrete but continuous reinforced concrete pavement have also been used (*2, 5*). Dowels for load transfer have been placed with baskets and dowel bar inserters. Perhaps the two greatest challenges involved in the construction of two-layer concrete systems include the additional costs and logistics required for two plants to produce different concrete mixtures and the additional costs for two slip-form paving machines (*4, 6*). A single slipform paver was once commonly used in Europe, but Austria and Germany have concluded that two machines offer more flexibility and control than the current single paver technology (*6*).

Limited research on the structural and functional performance of this composite pavement has been performed in the U.S. since interest diminished in the 1970's. However, federal initiatives have attempted to leverage the European experience. Some of these initiatives include the 1992 U.S. tour of European concrete highways and the 2006 International Technology Scanning Program tour of long-life concrete pavements in Europe and Canada (*2, 5*). European countries with significant two-layer concrete paving experience include Germany, Austria, and France (*2, 5, 6*). These countries use more durable aggregates in the thin surface layer and less expensive and often recycled materials in the lower layer. Typical layer thicknesses for highway applications range from 1.5 to 2 inches (4 to 5 cm) for the surface layer and 7.5 to 10 inches (19 to 25 cm) for the lower layer. A two-layer concrete pavement at the

Munich Airport was constructed with a surface layer thickness of 5.5 inches (14 cm) and a lower layer thickness of 9.5 inches (24 cm) (*4*).

Research Objectives

The primary objective of this investigation was to determine the structural and functional performance of a two-layer concrete pavement experimental project after 30 years of service. A secondary objective was to document historical design and construction information to aid current researchers and designers develop practical, long-life alternative concrete pavements.

PROJECT DESCRIPTION

The experimental project is located in the southbound lanes of SR 45 in Charlotte County, between Fort Myers and Punta Gorda. This roadway consists of four 12 foot lanes divided by a grassy median. The Fort Myers area has an average low temperature of 65° F (18°C) and an average high temperature of 84°F (29°C). Annual rainfall in excess of 50 inches (127 cm) is typical (*8*). Truck traffic decreased initially after the nearby I-75 was opened to traffic in 1981 but has gradually increased as travelers began to use SR 45 as an alternate route to bypass the growing Fort Myers area. More than 5.1 million trucks (6.2 million 18 kip equivalent single axle loads (ESALs)) have utilized this route since the construction was completed. FIGURE 2 shows the truck traffic trends over the past 30 years.

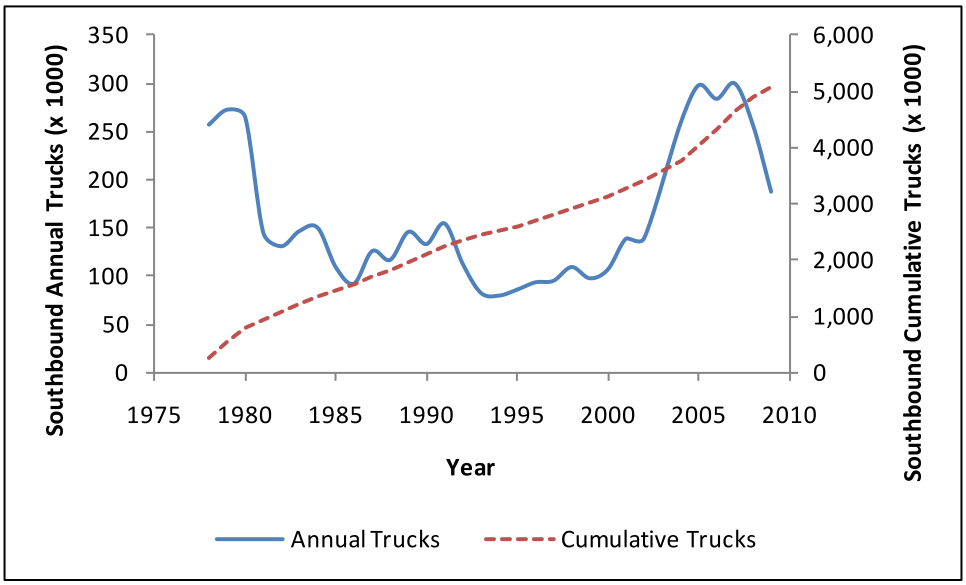


FIGURE 2 Truck traffic over the last 30 years.

CONSTRUCTION NOTES AND MEASUREMENTS

The construction of the experimental project began in April 1976 and was opened to traffic in March 1978. The following features were considered in the initial design and construction of the various test sections:

1. PCC joint orientation (right angle and skewed)
2. PCC joint spacing
3. Load transfer methods
4. PCC reinforcement

5. PCC/Econocrete bond condition
6. Non-traditional joint concepts
7. Econocrete strength
8. Subbase type

TABLE 1 summarizes the important design parameters for each of the experimental sections. A standard concrete pavement design was included as a control section. A brief description of each section and related construction details are presented below. Further in-depth construction information can be found elsewhere (*9*, *10*). It should be noted that section names are different in these reports.

Asphalt Overlay

Asphalt overlays of a 10 inch (25 cm) limerock base and a 9 inch econocrete base were initially included in the experimental project. A 6 inch granular subgrade was constructed for these sections. The subsection with the limerock base received 3 inches (7.6 cm) of asphalt while the econocrete sections received 2 inches (5 cm). A 1 inch (2.5 cm) friction course was applied to all of the subsections. In addition to a tack coat, a tine texture was applied to the surface of the econocrete base to promote bonding with the asphalt. Due to the tine texture, the top 0.25 inches (0.64 cm) of the econocrete base was pulled up by the tracks of the asphalt paver in some locations.

An evaluation performed after 10 years found rut depths of 0.1 to 0.2 inches (2.5 to 5 mm) in the standard asphalt section. Reflective transverse cracks were observed in sections with the econocrete base at 30 to 40 foot (9 to 12 meter) intervals.

TABLE 1 Experimental Sections

Section	Asphalt Overlay			Control	1A	1B	1C	2A	2B	2C
Length, ft	2000	2000	2000	2000	2000	2900	1700	2000	2000	2000
Surface	4 inch HMA	3 inch HMA		9 inch PCC	3 inch PCC			3 inch PCC		
PCC reinforcement	NA			None	None			None		
Joint spacing/ orientation	NA			20 ft/90°	15 feet / skewed			15 feet / 90°		
Load transfer	NA			None	None			None		
PCC/base interface	NA			NA	Monolithic bond			Monolithic bond		
Base	10 inch LR	None								
Econocrete	None	9 inch Mix B	9 inch Mix C	None	9 inch Mix A	9 inch Mix B	9 inch Mix C	9 inch Mix A	9 inch Mix B	9 inch Mix C
Subgrade	12 in. granular	6 inch granular		6 in. cement treated	6 inch granular			6 inch cement treated		

Section	3A	3B	4A	4B	5A	5B	5C
Length, ft	2000	2000	2050	2100	1023	1023	900
Surface	3 inch PCC		3 inch reinforced concrete		2 inch fiber reinforced concrete (FRC)		3 inch FRC
PCC reinforcement	None		Various reinforcement designs (steel, wire fabric, and steel fiber)		None		
Joint width / orientation	20 feet / 90°		NA	Elastic joint at 8 to 20 feet	9 to 15 ft. joint inserted into base / 90°		50 to 75 ft. joint inserted into base / 90°
Load Transfer	1 inch diameter x 18 inch long dowels at 12 inch centers		None		None		
PCC/base interface	Monolithic bond		Unbonded	Bonded	Bonded		Unbonded
Econocrete	9 inch Mix A	9 inch Mix B	9 inch Mix B		9 inch Mix B		8 inch Mix B
Subgrade	6 inch granular		6 inch granular		6 inch granular		

1 inch = 2.54 mm
1 foot = 0.305 meter

Control Section

The PCC control section consists of a standard 9 inch (23 cm) jointed plain concrete pavement (JPCP) that utilized a No. 4 nominal size limestone aggregate. Transverse joints were cut at a 90^{o} orientation and spaced at 20 foot (6 m) long intervals. The foundation included a 6 inch (15 cm) cement-treated subbase. A review of construction notes and plans revealed no information on whether a bond breaker was used between the PCC and the cement treated subbase. Cores retrieved in 2007 suggest a weak bond or no bond was present at the time of coring. This design was considered to be a standard concrete pavement at the time of construction. TABLE 2 summarizes the PCC properties.

TABLE 2 Standard PCC Section Properties

Property	Test Value	Standard Deviation	n
Compressive Strength, psi	5,110	444	30
Modulus of Rupture, psi	735	52	27
Split Tensile Strength, psi	515	70	20
Modulus of Elasticity, ksi	5,750	270	30
Air Content, %	3.8	0.31	10
Slump, inch	1.6	0.77	10
Unit Weight, lb/ft^2	141	0.78	9
1 psi = 6.89 kPa 1 inch = 2.54 cm 1 lb/ft^2 = 16 kg/m^3 n = number of samples Note 1: Strength tests performed at 28 days Note 2: Tests conducted according to AASHTO test procedures			

Experimental Sections 1 to 3

The monolithic composite construction used in sections 1 to 3 required the simultaneous production of two mixtures. Differentiation of each mixture presented no problems at the roadway. Each delivery truck was provided a ticket designating the type of material and the concrete surface mix was flagged. Error was further minimized by maintaining the same trucks and drivers for each mix type. The 9 inch (23 cm) econocrete layer was placed with a modified spreader. The modifications included the addition of spud vibrators to consolidate the econocrete, a vibrating pan or screed for striking off excess material, and trailing forms. In essence, the modifications transformed the spreader into a crude slipform paver. The econocrete surface was scarified using two deep passes of a transverse tine texturing machine to enhance the bond with the top layer.

The PCC surface consisted of a 3 inch (7.5 cm) jointed plane concrete pavement (JPCP) of the same mix design and properties as the control section. The surface layer was constructed using a slipform paver approximately ½ to 1 hour after placing the econocrete. The result was a monolithic structure with the econocrete layer bonded to the surface layer. FIGURE 3 shows the paving process and TABLE 3 summarizes the properties of the econocrete mixture. Cement content was varied to achieve three different strength levels. FIGURE 4 shows the gradation and properties of the aggregates used for the econocrete.

FIGURE 3 Paving operation.

A transverse tine texture was applied to the final surface layer. Section 3 specified the placement of 18 inch (45.5 cm) long and 1 inch (2.5 cm) diameter dowel bars placed at 12 inch (30.5) centers at mid-depth of the total composite pavement structure. The dowel bars were secured to the subbase in prefabricated baskets prior to placement of the econocrete. A 30 inch (76 cm) #5 center-line tie bar was inserted into the fresh concrete at three foot intervals. Transverse joints were cut in a skewed orientation in section 1, and at right angle in sections 2 and 3. Joints were cut at 15 foot (4.5 m) intervals in sections 1 and 2, and 20 foot (6 m) intervals in section 3. Final transverse and longitudinal joint depths were specified to be 3.5 inches. A neoprene joint seal was specified for all transverse contraction joints. Hot-poured joint sealant was specified for all longitudinal joints. Joint details are shown in FIGURE 5.

TABLE 3 Econocrete Properties

Laboratory Test	Econocrete Mix A			Econocrete Mix B			Econocrete Mix C		
	Test Value	Std Dev	n	Test Value	Std Dev	n	Test Value	Std Dev	n
Compressive Strength[1,2], psi	1,955	87	9	1,280	220	24	675	190	15
Modulus of Rupture[1,2], psi	380	27	9	280	33	26	160	43	15
Split Tensile Strength[1,2], psi	250	14	6	175	31	16	93	29	11
Modulus of Elasticity[1,2], ksi	2,210	130	9	1,730	260	23	1,290	129	15
Air Content[2], %	4.1	0.81	3	3.8	0.43	8	4.1	0.54	5
Slump[2], inch	1.8	0.14	3	1.9	0.46	8	1.7	0.32	5
Design Cement content, lb/yd^3	340	Not Measured		290	Not Measured		220	Not Measured	
Unit Weight[2], lb/ft^2	132	0.7	3	133	1.3	8	132	1.8	4

1 psi = 6.89 kPa
1 inch = 2.54 cm
1 lb/yd^3 = 0.59 kg/m^3
1 lb/ft^2 = 16 kg/m^3
n = number of samples
Note 1: Strength tests performed at 28 days
Note 2: Tests conducted according to AASHTO test procedures

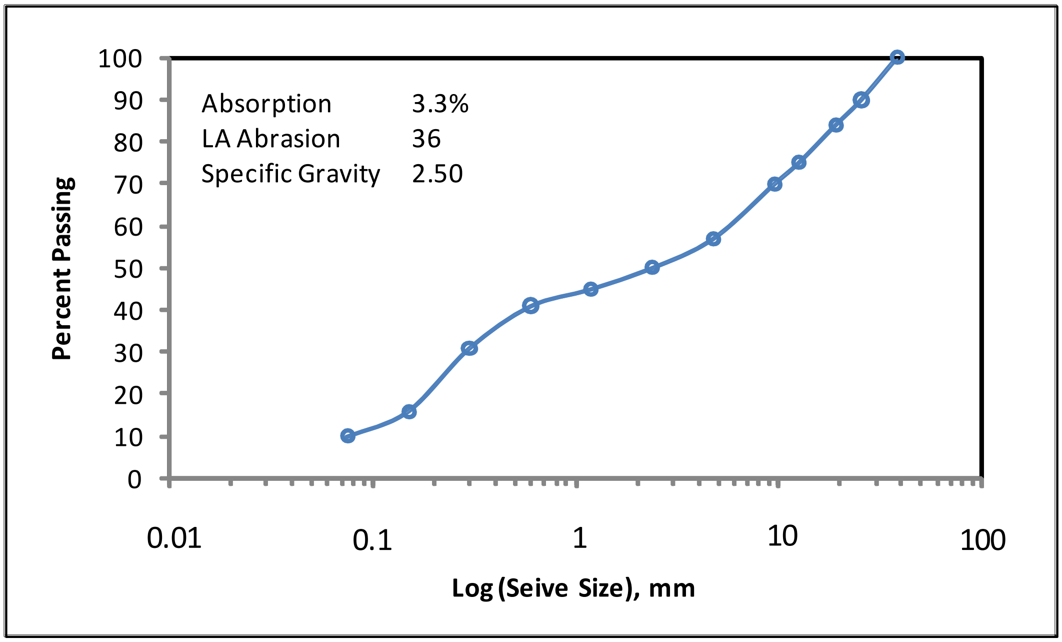
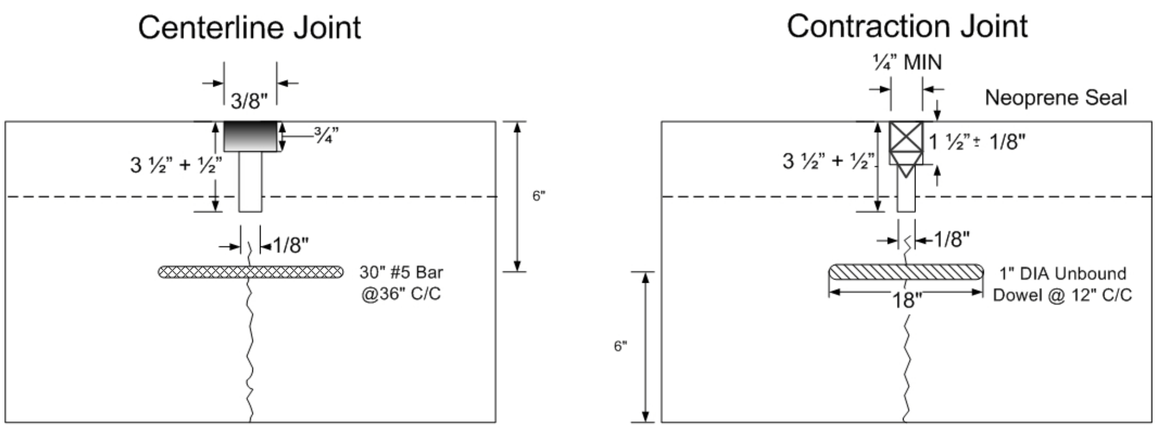


FIGURE 4 Econocrete aggregate properties.



Note: Dowel bars specified for sections 3A and 3B only.

FIGURE 5 Joint construction details.

Experimental Sections 4 and 5

Section 4 included a 3 inch (7.5 cm) concrete pavement reinforced with either steel or wire fabric. The concrete was placed a day after the econocrete was constructed. The wire fabric proved difficult to place uniformly due to undulations and deformations that were impossible to work out. The bond between the PCC and the econocrete was inhibited in one subsection.

Section 5 included subsections of 2 inches and 3 inches (5 and 7.5 cm) of steel fiber reinforced PCC. Workability of this mixture was poor and clumping of the steel fibers was widespread. Excessive bleeding and differential stiffness was typical which resulted in an inability to achieve a satisfactory surface texture. Several attempts to correct the problems were unsuccessful.

Some subsections of sections 4 and 5 included an experimental crack inducing concept. This concept required placement of a plastic crack inducing joint insert in the fresh surface of the econocrete. A debonding agent was applied at various distances before and after the insert as shown in FIGURE 6. The concept hypothesized that a hairline crack would reflect in the concrete surface layer at the locations corresponding to the joint insert and debonded interface. Over time, transverse cracks did appear at the approximate location of the joint insert. However, longitudinal cracks formed perpendicular to the transverse cracks in most of the sections. Heavy spalling occurred at some of the transverse joints. Crack sealant was not applied to all of the transverse cracks since they occurred after construction and at various time periods. Many of these sections failed and were removed within 1 year due to extensive cracking and the unsightly appearance that resulted from the construction problems noted above. Furthermore, it was concluded that a bond must be established for thin concrete overlays of rigid pavements. Current guidelines for traditional concrete overlays call for an unbonded overlay thickness of 4 to 11 (10 to 28 cm) inches and bonded overlay thickness of 2 to 5 inches (5 to 7 cm) depending on traffic and desired life (*11*). Some sections with the experimental joint concept were still in service after 10 years of service but their condition ranged from poor to fair.

FIGURE 6 Debonding material used for experimental joint concept (steel reinforcement shown).

Pavement Foundation

The finished pavement profile required approximately 3 feet (1 m) of fill built in 8 inch (20 cm) layers and compacted to 98 percent of the maximum density as measured by AASHTO T-99. The 6 inch (15 cm) granular subbase was a mixture of shell and limerock that was compacted to a density of not less than 100 percent and not more than 105 percent maximum density as measured by AASHTO T-180. The 6 inch (15 cm) cement treated subbase had a cement content that ranged from 9.0 to 11.5 percent by weight and was mixed in-place. The average properties measured during construction of the compacted fill, granular subbase, and cement treated subbase are summarized in TABLE 4.

TABLE 4 Average Foundation Layer Properties

Layer	Maximum Dry Density, pcf[2]	Optimum Moisture[2], %	30 inch Plate Bearing Modulus[3], psi	Limerock Bearing Ratio[3] (LBR)	Compressive Strength[1,2], psi	Modulus of Elasticity[1,2], ksi
Compacted Fill	109	12.2	27,800	Not measured	NA	Not Measured
Granular Subbase	122	9.3	24,800	145	NA	Not Measured
Cement Treated Subbase	115	9.6	48,800	Not Measured	880	1,440

LBR = 0.8 x California bearing ratio (CBR)
1 lb/ft^2 = 16 kg/m^3
1 psi = 6.89 kPa
1 ksi = 1000 psi
NA = not applicable
Note 1: Strength tests performed at 28 days
Note 2: Tests conducted according to governing AASHTO standards
Note 3: Tests conducted according to modified AASHTO test procedures

PERFORMANCE EVALUATION

The performance evaluation summarized herein focused on sections that were still in service (Control and sections 1 to 3) at the time of the subject investigation. The limited maintenance performed on these sections included two slab replacements in section 1B and minor patching. The slab replacements were made to correct extensive scaling due to over-finishing the concrete. The primary parameters used to evaluate the performance of the experimental sections include:

1. Pavement deflection/stiffness
2. Load transfer
3. Joint faulting
4. Pavement smoothness
5. Pavement cracking
6. Laboratory measurements on cores

Pavement Deflection/Stiffness

Deflection measurements are often used to infer a pavement system's structural adequacy. A comprehensive deflection measurement survey was performed in October 2007 to measure pavement stiffness, joint edge deflection, and loss of support at the slab corner. Daytime and nighttime falling weight deflectometer (FWD) tests were performed to account for the curling effect of the concrete slab due to thermal gradients. During testing, temperature measurements were made at the surface of the concrete and at the bottom of the econocrete. The temperature measurements are summarized in FIGURE 7. A polynomial curve appears to provide a good correlation during this time period. The maximum positive temperature gradient occurred between 1:00 and 2:00 PM while the largest negative temperature gradient occurred between 6:00 and 7:00 AM. FWD testing was performed between the daytime hours of 2:00 to 4:30 PM and the nighttime hours of 10:00 PM to 4:30 AM.

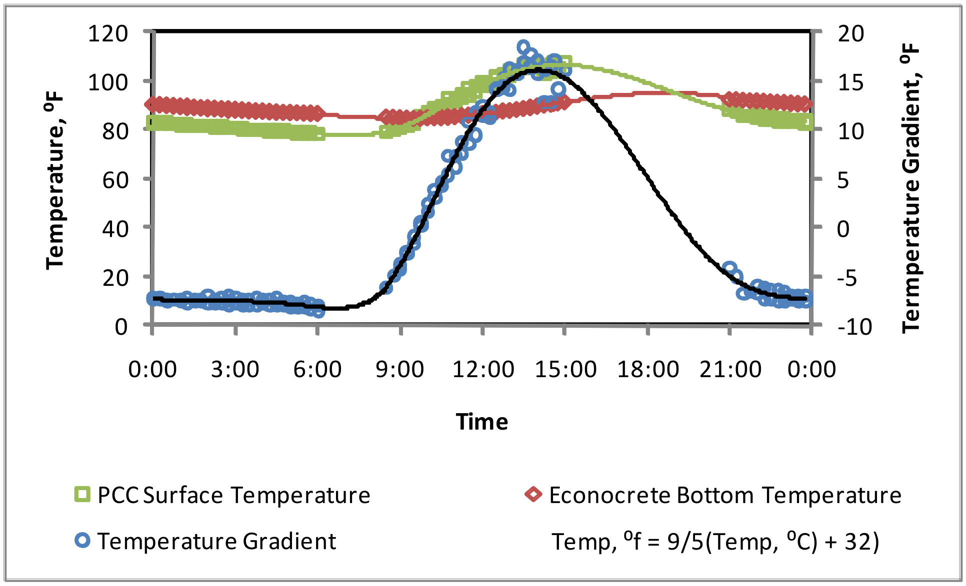


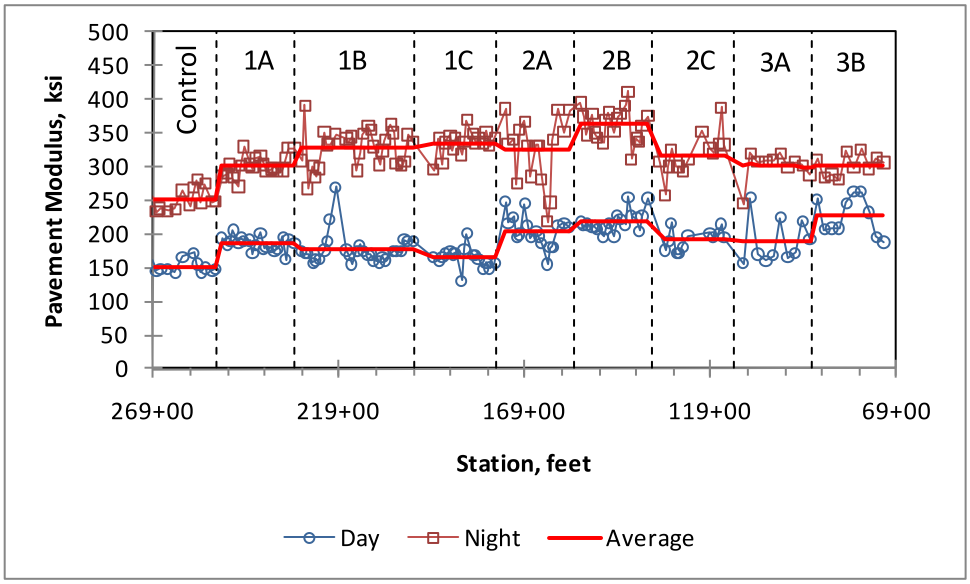
FIGURE 7 Concrete temperature measurements.

Deflection measurements were made in the outside lane at three locations: 1) center of the slab, 2) the outside corner, and 3) the joint edge along the outside wheel path. The modulus of the overall pavement system was calculated to determine which overall pavement structure was more substantial in terms of stiffness. The pavement modulus was determined by loading the center of each slab and utilizing the following equation (*12*):

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

Where, E_0 = composite modulus of the entire pavement system beneath the load plate
a = radius of load plate
σ_0 = peak pressure of the FWD load plate
d_0 = peak center FWD deflection

FIGURE 8 illustrates the temperature gradient effect on the pavement modulus. On average, the day time pavement stiffness was approximately 0.6 times the night time stiffness. Interestingly, the full-depth concrete pavement structure (control section) was found to be less stiff than the two-layer pavement structures.



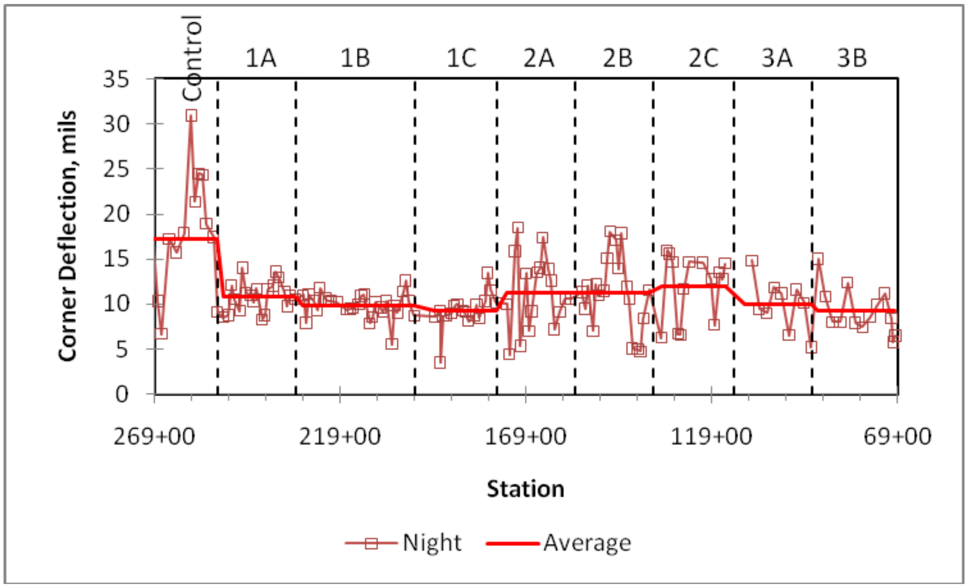
Pavement Modulus, ksi										
		Control	1A	1B	1C	2A	2B	2C	3A	3B
Day	Avg.	152	186	177	165	205	218	192	188	226
	Std. Dev.	9	10	23	15	23	15	15	33	28
Night	Avg.	251	302	329	336	326	364	318	301	303
	Std. Dev.	15	16	27	49	49	25	30	22	16
Day/Night Ratio		0.61	0.62	0.54	0.49	0.63	0.60	0.61	0.63	0.75
1 ksi = 6.9 MPa										

FIGURE 8 Pavement modulus.

Load Transfer and Corner Deflection

FIGURE 9 summarizes the day-time load transfer as well as center, edge, and corner deflections. Joint load transfer measurements were made along the outside wheel path joint. It should be noted that concrete temperatures measured during testing never dropped below 78º F (26ºC). The high joint edge load transfer efficiency (LTE) could be a result of the slabs locking up due to seasonal expansion and is not likely to be representative of the load transfer performance during the lifetime of the pavement. Moderate joint spalling was observed in sections 1 and 2 and moderate to severe joint spalling was found in the control section. Joint spalling may be an indication of excessive horizontal slab movement and/or infiltration of joint reservoirs with debris. The control section was constructed with 20 foot (6 m) long slabs which may have also promoted spalling and transverse cracking due to excessive stress from thermal expansion of the slab. FDOT currently specifies a joint spacing of no greater than 15 feet (4.5 m) long (*13*).

Deflection measurements were also made on the outside slab corners. Excessive deflections and pumping in the control section suggest a loss of support. Slightly higher deflections and pumping were also measured for section 2. It is interesting to note that these sections included a cement treated subbase.



Deflection and LTE Summary										
Experimental Section		Control	1A	1B	1C	2A	2B	2C	3A	3B
Center Def., mils	Day	4.8	3.9	4.1	4.5	3.6	3.3	3.8	4.0	3.3
	Night	2.9	2.4	2.2	2.2	2.3	2.0	2.3	2.4	2.4
Joint Edge Def., mils	Day	4.2	3.5	3.4	3.7	3.3	3.1	3.6	3.6	3.8
	Night	4.7	4.0	4.1	3.5	3.6	4.1	4.0	3.5	4.8
Corner Def., mils	Day	5.5	5.1	5.2	4.1	3.8	3.4	3.7	4.0	4.1
	Night	17.2	10.9	9.9	9.3	11.3	11.2	12.0	10.9	9.9
Edge LTE, %	Day	89	89	90	90	91	91	93	93	92
	Night	85	87	87	86	88	87	86	87	88
1 mil = 0.0254 mm										

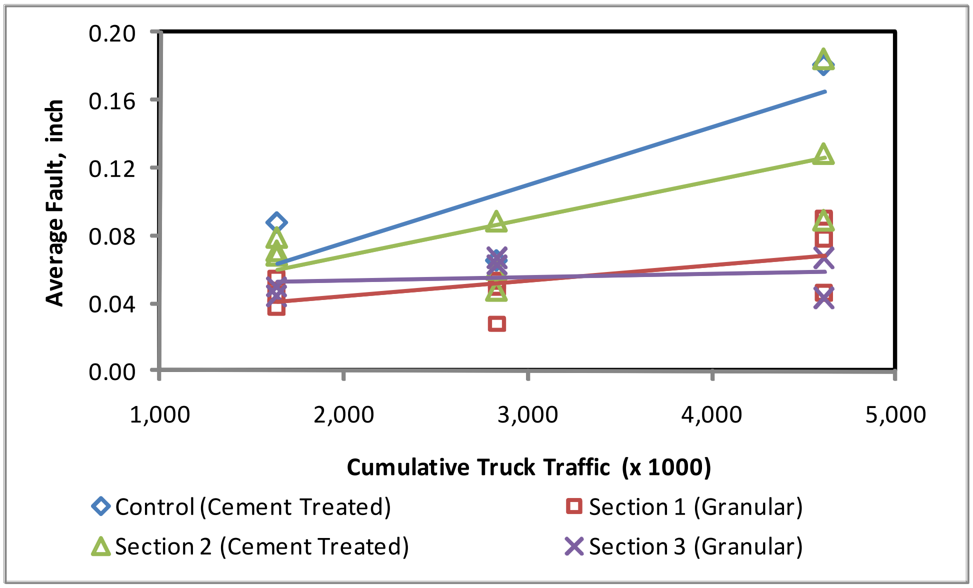
FIGURE 9 Summary of load transfer and deflection parameters.

Joint Faulting

Excessive faulting of joints and cracks is a common trigger for many rehabilitation strategies. Florida has adopted a fault depth of 0.10 inches (2.5 mm) as a trigger for grinding and other maintenance strategies (*13*, *14*). In general, four conditions must be present for faulting to take place. These conditions include (*13*):

1. Poor load transfer of joints and cracks
2. Erodible subbase material
3. Available free water at the slab/subbase interface
4. Traffic

Faulting measurements for 1986, 1996, and 2007 are shown in FIGURE 10. In general, the faulting rates for sections constructed with the cement treated subbase were found to be greater than that of the sections with the granular subbase. The greater faulting measurements and corner deflections suggest, as well as evidence of pumping indicate the cement treated subbase is more erodible than the granular subbase.



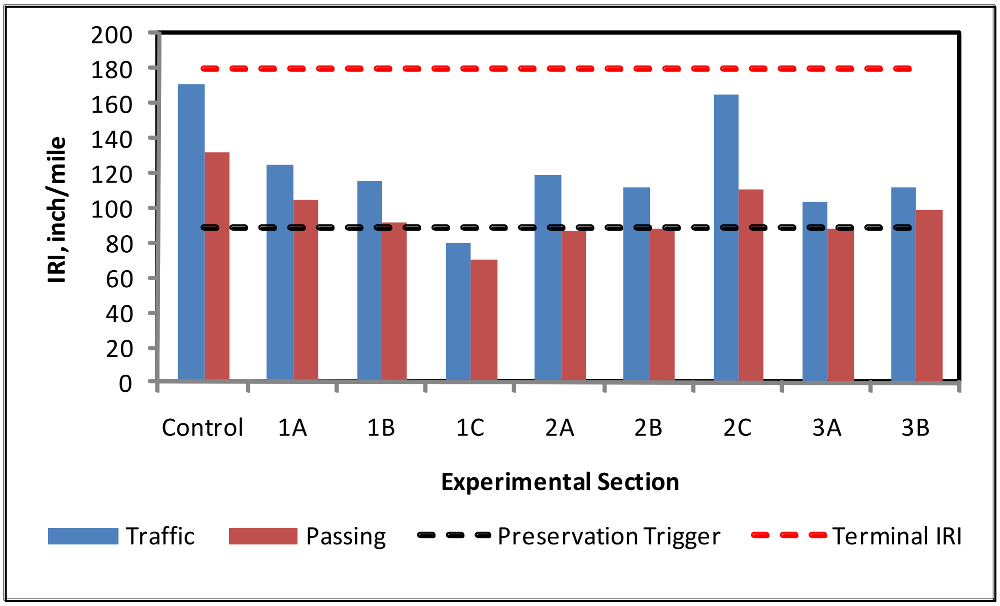
Average Fault Depths, inch									
Year	Control	1A	1B	1C	2A	2B	2C	3A	3B
1986	0.09	0.06	0.04	0.04	0.07	0.07	0.08	0.05	0.04
1996	0.06	0.05	0.05	0.03	0.06	0.05	0.09	0.07	0.06
2007	0.18	0.09	0.08	0.05	0.13	0.09	0.18	0.04	0.07
1 inch = 25.4 mm									

FIGURE 10 Fault depths.

Pavement Smoothness

The smoothness has been evaluated almost every year since construction, and reported in terms of Ride Number (RN) and International Roughness Index (IRI). The RN is a profile index that rates the rideability of a road using a 0 to 5 scale that corresponds to a user's perception of pavement roughness (*14*). A RN of 5.0 represents a perfect ride quality while an RN of 0 corresponds to a virtually impassable surface. In general, the RN has remained above 4.0 for approximately 25 years for all sections except for sections 1A, 2C, and the control whose RN remained above 4.0 for 15 years. After 30 years, the RN for section 2C and the control have begun to deteriorate at a faster rate and have just slipped below 3.

The International Roughness Index is a common index that is a function of qualities related to pavement surface roughness that impact vehicle response (*14, 15*). A typical IRI value published by the FHWA to trigger rehabilitation or preservation strategies for a low volume roadway such as SR45 is 89 inch/mile (1400 mm/km) (*16*). Many of these sections have just recently exceeded this threshold. Only the control and section 2C were approaching Florida's terminal IRI of 180 inch/mile (2,840 mm/km) as per the 2007 survey IRI results shown in FIGURE 11.



2007 Roughness Summary										
Index	Lane	Control	1A	1B	1C	2A	2B	2C	3A	3B
RN	Traffic	2.8	3.4	3.4	3.8	3.4	3.5	2.9	3.7	3.6
	Passing	3.3	3.6	3.7	3.9	3.8	3.8	3.6	3.9	3.8
IRI, in/mile	Traffic	172	126	116	81	119	113	166	104	112
	Passing	132	105	93	71	87	89	111	89	99
1 inch = 25.4 mm 1 mile = 1.61 km										

FIGURE 11 2007 Roughness summary.

Pavement Cracking

A crack evaluation was performed in 2007 to determine the type and number of cracks. TABLE 5 shows the result of this survey for the traffic lane. Longitudinal cracking was the most predominant crack type and was observed primarily in the control section and section 2. Longitudinal cracks were observed in the wheel paths, along the center of the slab, and near the longitudinal joint. A photograph of longitudinal cracking observed in section 2 is shown in FIGURE 12. Literature indicates that cement-treated subbases may promote uncontrolled cracking due to high friction, rigidity, and in some cases, bonding between the concrete slab and subbase (*17, 18*). In addition, the compressive strength of the cement-treated subbase was 880 psi (6 MPa) at the time of construction. The ACPA recommends a compressive strength of 300 psi (2 MPa) to 800 psi (5.5 MPa) to limit the stiffness and minimize curling and warping stresses (*19*). Florida no longer recommends the use of cement stabilized courses below concrete pavements due to their rigidity, lack of permeability, and the difficulty in achieving non-erodible properties (*13*).

TABLE 5 2007 Crack Summary

Percentage of Cracks									
Crack Type	Control	1A	1B	1C	2A	2B	2C	3A	3B
Corner	6	0	1	1	1	0	1	0	0
Transverse	12	0	1	1	0	0	5	0	3
Longitudinal	17	0	0	1	19	1	45	0	0

FIGURE 12 Longitudinal cracking in section 2.

Laboratory Data

More than 40 cores were collected in 2007 to verify the long-term structural properties of the PCC and econocrete. TABLE 6 summarizes the laboratory properties. In general, the properties of the PCC and econocrete A mixture have experienced little long term change while the econocrete B and C mixes have increased in strength and now approach those of the A mixture. It should be noted that, in general, there were less samples tested and more variability in the 2007 test data than the samples measured during construction. The modulus and strength values for the PCC remain more than double the econocrete. The considerable bond strength is evidence that the two-layer system is still acting as a monolithic structure. The slight difference in the PCC and econocrete coefficient of thermal expansion (COTE) (less than 5 percent) minimized the thermal stress on the bond. Thermal curling and loading stresses would have been greater and may potentially lead to more cracking had the bond degraded. FIGURE 13 shows photographs of cores taken from the control sections and section 3. The photographed core numbers represent a previous numbering system.

TABLE 6 PCC and Econocrete Laboratory Properties After 30 years of Service

Laboratory Test	PCC			Econocrete Mix A			Econocrete Mix B			Econocrete Mix C		
	Test Value	Std Dev	n	Test Value	Std Dev	n	Test Value	Std Dev	n	Test Value	Std Dev	n
Compressive Strength, psi	5,653	732	12	2,255	810	4	1,843	294	6	1,763	342	4
Split Tensile Strength, psi	420	64	24	260	32	8	247	23	9	226	34	5
Modulus of Elasticity, ksi	4,724	815	31	1,789	247	11	1,897	177	13	1,946	34	7
Bonded Interface Shear Strength, psi	NA	NA	NA	432	75	3	614	72	4	465	92	2
COTE, in/in/°F	5.25E-06	2.45E-07	4	5.36E-06	2.61E-07	12	5.45E-06	2.86E-07	13	5.04E-06	3.55 E-07	7

n = number of samples
Modulus of elasticity (ASTM C469)
Compressive strength (ASTM C39)
Split tensile strength (ASTM C496)
Bonded interface shear strength (AASHTO T323)
Coefficient of thermal expansion (AASHTO TP60)

1 psi = 6.89 kPa
1 inch = 2.54 cm
Temp, °F = 9/5(Temp, °C) + 32

FIGURE 13 Photographs of control and section 3 cores.

CONCLUSIONS

Several sections of a 30 year old experimental two-layer concrete pavement in Florida are still in service today with minimal maintenance. More than 5.1 million trucks (6.2 million 18 kip ESALs) have utilized this route since the construction was completed. The better performing two-layer concrete pavement section (section 3) included 20 foot (6 m) long slabs, 90° joints, dowels, and a granular subbase. The other section constructed with the same subbase (section 1) and 15 foot (4.5 m) long slabs and skewed joints experienced slightly greater corner deflections and faulting. The distress levels of these sections are still minimal and are just approaching the level at which good performance can be maintained with a well planned preservation strategy. Preservation strategies that could be utilized include diamond grinding, cleaning and sealing joints and cracks, and perhaps load transfer restoration. A more specific evaluation may be necessary to select the right preservation strategy. Section 2 and the control section constructed with a cement-treated subbase experienced greater faulting and excessive longitudinal cracking. Pumping and greater corner deflections were also observed in the control section. The control section, constructed with 20-foot (6 m) long slabs, experienced greater transverse cracking and moderate to severe spalling. These observations reinforce Florida's current recommendation against using cement treated subbases for concrete pavements and limiting joint spacing to 15 feet (4.5 m). A more extensive rehabilitation strategy will be required to extend the life of these sections. No significant performance differences were noted for subsections with differing econocrete strength.

This project has demonstrated that a two-layer concrete system consisting of a lower quality econocrete layer and higher quality PCC surface is a feasible sustainable pavement alternative with long-lasting service life.

REFERENCES

1. Snell, L.M. and B.G. Snell. "Oldest Concrete Street in the United States." Concrete International, March 2002, p. 72 -74. American Concrete Institute, 2002.
2. Hall, K. Long-life Concrete Pavements in Europe and Canada. Publication No. FHWA-PL-07-027. Federal Highway Administration, Washington D.C., 2007.
3. Shields-Cook, Sabrina and P.C. Taylor. "Working a Double Lift." Roads and Bridges, January 2009, p. 32-35.
4. Cable, J.K. and D.P. Frentress. Two-Lift Portland Cement Concrete Pavement to Meet Public Needs. Publication No DTF61-01-X-00042 (Project 8). Federal Highway Administration, Washington D.C., 2004.
5. 5Video Conference Agenda and Meeting Notes: Two-Lift Concrete Pavement Meeting Notes 27 June 2007. Retrieved from http://www.cptechcenter.org/projects/two-lift-paving/jun27-07-video-conf-minutes.pdf.
6. Tompkins, D, L. Khazanovich, M.I. Darter, and W. Fleisher. Design and Construction of Sustainable Pavements: Austrian and German Two-Layer Concrete Pavements. In *Transportation Research Record: Journal of the Transportation Research Board, No. 2098*, Transportation Research Board of the National Academies, Washington D.C., 2009, p.75-85.
7. Darter, M.I., Report on the 1992 U.S. Tour of European Concrete Highways. Publication No. FHWA-SA-93-012. Federal Highway Administration, Washington D.C., 1993.
8. Climatezone.com. Climate For Ft. Myers, FL. Retrieved from http://www.climate-zone.com/climate/united-states/florida/fort-myers/.
9. Page, G.C., and L.W. Harper. Florida Econocrete Test Road Post Construction and Materials Report. Florida Department of Transportation, Tallahassee, FL, November 1980.
10. Larsen, T.J. and J.M. Armaghani. Florida Econocrete Test Road A 10-Year Report. Florida Department of Transportation State Materials Office, Gainesville, FL, 1989.
11. Harrington, D., Guide to Concrete Overlays. National Concrete Pavement Technology Center. Ames, IA, September, 2008.
12. Stubstad, R.N., Y.J. Jiang, and E.O. Lukanen. Guidelines for Review and Evaluation of Backcalculation Results. Publication No. FHWA-RD-05_152. Federal Highway Administration, Washington D.C., February 2006.
13. Florida Department of Transportation. Rigid Pavement Design Manual. FDOT Report No. 625-010-006-e, Tallahassee, FL, January 2009.
14. Florida Department of Transportation. Rigid Pavement Condition Survey Handbook. Gainesville, FL. June 2009.

15. Sayers, M.W., and S.M. Karahimas. The Little Book of Profiling. University of Michigan Transportation Research Institute, 1998. Retrieved from http://www.umtri.umich.edu/content/LittleBook98R.pdf.
16. Smith, K.D., T.E. Hoerner, and D.G. Peshkin. Concrete Pavement Preservation Workshop. Federal Highway Administration, Washington D.C., February 2008.
17. Hall, J.W., J. Mallela, and K.L Smith. Stabilized and Drainable Base for Rigid Pavement. Report IPRF-01-G-002-021(G). Innovative Pavement Research Foundation, Skokie, IL, October 2005.
18. Voigt, G.F. Early Cracking of Concrete Pavement: Cause and Repair. American Concrete Pavement Association. Skokie, IL., May 2002.
19. Stabilized Subbases. Technical Series Document TS204.7P, American Concrete Pavement Association, Skokie, IL, 2008.